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Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures

Reported by ACI Committee 440

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Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures

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Several voting members of the committee contributed chapters or made other substantial contributions to the report. In addition, the committee would like to acknowledge the contribution of associate members T. Ivan Campbell and Luke A. Bisby.

Applications of fiber-reinforced polymer (FRP) composites as reinforcement for concrete structures have been growing rapidly in recent years. ACI Committee 440 has published design guidelines for internal FRP reinforcement, externally bonded FRP reinforcement for strengthening, prestressed FRP reinforcement, and test methods for FRP products. Although these guidelines exist, new products and applications continue to be developed. Thus, this report summarizes the current state of knowledge on these materials and their application to concrete and masonry structures. The purpose of this report is to act as an introduction to FRP materials in areas where ACI guides exist, and to provide information on the properties and behavior of concrete structures containing FRP in areas where guides are not currently available. If an ACI guide is available, the guide document supersedes information in this report, and the guide should always be followed for design and application purposes. ACI Committee 440 is also in the process of developing new guides and thus the current availability of guides should be checked by the reader. In addition to the material properties of the constituent materials (that is, resins and fibers) and products, current knowledge of FRP applications, such as internal reinforcement including prestressing, external strengthening of concrete and masonry structures, and structural systems, is discussed in detail. The document also addresses durability issues and the effects of extreme events, such as fire and blast. A summary of some examples of field applications is presented.

Keywords: aramid fibers; blast; bridges; buildings; carbon fibers; composite materials; corrosion; design; dowels; ductility; durability; external reinforcement; fatigue; fiber-reinforced polymer (FRP); fibers; fire; glass fiber; masonry; mechanical properties; polymer resin; prestressed concrete; seismic; stay-in-place forms; structural systems; test methods.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

The purpose of this report is to present the current state of knowledge with regard to applications of fiber-reinforced polymer (FRP) materials in concrete. This report summarizes the fundamental behavior, the most current research, design codes, and practical applications of concrete and masonry structures containing FRP. This document is intended to complement other reports (for example, standards and design guidelines) produced by ACI Committee 440, either by summarizing the research that supports those documents or by providing information on future developments of those documents. If an ACI guide is available, the guide document supersedes information in this report, and the guide should always be followed for design and application purposes. ACI Committee 440 is also in the process of developing new guides; thus, the current availability of guides should be checked by the reader.

FRP materials are composite materials that typically consist of strong fibers embedded in a resin matrix. The fibers provide strength and stiffness to the composite and generally carry most of the applied loads. The matrix acts to bond and protect the fibers and to provide for transfer of stress from fiber to fiber through shear stresses. The most common fibers are glass, carbon, and aramid. Matrixes are typically epoxies, polyesters, vinylesters, or phenolics.

1.2—Historical perspective of FRP composites

While the concept of composites has been in existence for several millennia (for example, bricks made from mud and straw), the incorporation of FRP composite technology into the industrial world is less than a century old. The age of plastics emerged just after 1900, with chemists and industrialists taking bold steps to have plastics (vinyl, polystyrene, and Plexiglas) mimic and outdo natural materials. Spurred on by the needs of electronics, defense, and eventually space technologies, researchers created materials with properties that seemed to defy known principles, such as bullet-stopping Kevlar. The first known FRP product was a boat hull manufactured in the mid-1930s as part of a manufacturing experiment using a fiberglass fabric and polyester resin laid in a foam mold (ACMA MDA 2006). From this modest beginning, FRP composite applications have revolutionized entire industries, including aerospace, marine, electrical, corrosion resistance, and transportation.

FRP composite materials date back to the early 1940s in the defense industry, particularly for use in aerospace and naval applications. The U.S. Air Force and Navy capitalized on FRP composites' high strength-weight ratio and inherent resistance to the corrosive effects of weather, salt air, and the sea. Soon the benefits of FRP composites, especially its corrosion resistance capabilities, were communicated to the public sector. Fiberglass pipe, for instance, was first introduced in 1948 (ACMA MDA 2006) for what has become one of its widest use areas within the corrosion market, the oil industry. FRP composites proved to be a worthy alternative to other traditional materials even in the high-pressure, large-diameter situations of chemical processing. Besides superior corrosion resistance, FRP pipe offered both durability and strength, thus eliminating the need for interior linings, exterior coatings, and cathodic protection. Since the early 1950s, FRP composites have been used extensively for equipment in the chemical processing, pulp and paper, power, waste treatment, metal refining, and other manufacturing industries (ACMA MDA 2006). Myriads of products and FRP installations help build a baseline of proven performance in the field.

The decades after the 1940s brought new, and often revolutionary, applications for FRP composites (ACMA MDA 2006). The same technology that produced the reinforced plastic hoops required for the Manhattan nuclear project in World War II spawned the development of high-performance composite materials for solid rocket motor cases and tanks in the 1960s and 1970s. In fact, fiberglass wall tanks were used on the Skylab orbiting laboratory to provide oxygen for the astronauts. In 1953, the first Chevrolet Corvette with fiberglass body panels rolled off the assembly line (ACMA MDA 2006). Now, high-performance race cars are the proving ground for technology transfer to passenger vehicles. In the 1960s, the British and U.S. Navies were simultaneously developing FRP-based minesweeper ships because FRP composites are not only superior to other materials in harsh marine environments, they are also nonmagnetic. It was also noticed at that time that one of the features of FRP is the ability of the materials to reduce the radar signature of the structure, such as a ship or an aircraft. High-performance composite materials have been demonstrated in advanced technology aircraft such as the F-117 Stealth Fighter and B-2 Bomber. Currently, FRP composites are being used for space applications and are involved in several NASA test initiatives (ACMA MDA 2006).

While the majority of the historical and durability data of FRP composite installations comes from the aerospace, marine, and corrosion-resistance industries (ACMA MDA 2006), FRP composites have been used as a construction material for several decades. FRP composite products were first demonstrated to reinforce concrete structures in the mid-1950s (ACMA MDA 2006). In the 1980s, a resurgence in interest arose when new developments were launched to apply FRP reinforcing bars in concrete that required special performance requirements such as nonmagnetic properties or in areas that were subjected to severe chemical attack.

Composites have evolved since the 1950s, starting with temporary structures and continuing with restoration of historic buildings and structural applications. Typical products developed were domes, shrouds, translucent sheet panels, and exterior building panels. A major development of FRP for civil engineering has been the application of externally bonded FRP for rehabilitation and strengthening of concrete structures.

During the late 1970s and early 1980s, many applications of composite reinforcing products were demonstrated in Europe and Asia. In 1986, the world's first highway bridge using composite reinforcing tendons was built in Germany. The first all-composite bridge deck was demonstrated in China. The first all-composite pedestrian bridge was installed in 1992 in Aberfeldy, Scotland. In the U.S., the first FRP-reinforced concrete bridge deck was built in 1996 at McKinleyville, West Virginia, followed by the first all-composite vehicular bridge deck (The No-Name Creek Bridge in 1996) in Russell, Kansas. Numerous composite pedestrian bridges have been installed in U.S. state and national parks in remote locations not accessible by heavy construction equipment, or for spanning over roadways and railways (ACMA MDA 2006).

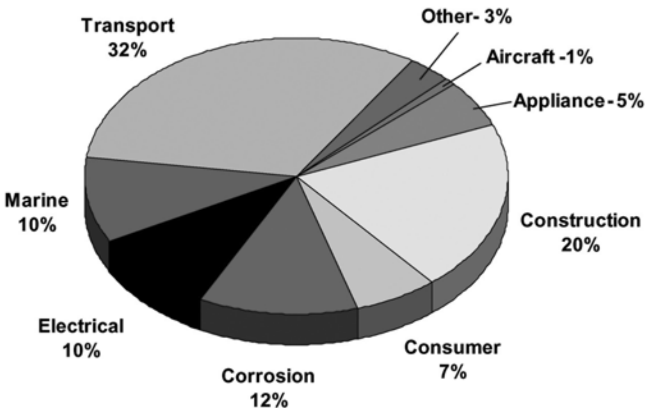


Fig. 1.1—U.S. composites shipments 2004 market share by end-use application (ACMA 2005). Total estimated volume: 1.8 billion kg (4.0 billion lb) (ACMA 2005 and PPG).

Composite fabricators and suppliers are actively developing products for the civil infrastructure, which is considered to be the largest potential market for FRP composites (ACMA MDA 2006). Concrete repair and reinforcement, bridge deck repair and new installation, composite-hybrid technology (the marriage of composites with concrete, wood, and steel), marine piling, and pier upgrade programs are just some of the areas that are currently being explored. This document describes all aspects of applications of FRP composites for concrete and masonry structures including internal reinforcement, strengthening, prestressing, and integrated stay-in-place forms.

1.2.1 *Industry statistics*—The composites industry associations and producers have traditionally tracked FRP market growth in several primary markets: aircraft/aerospace, appliance/business equipment, construction, consumer products, corrosion-resistant equipment, electrical, marine, transportation, and other applications. The American Composites Manufacturers Association (ACMA 2005) and PPG Industries reported that estimates of composites shipments in 2004 reached 1.8 billion kilograms (4.0 billion pounds). Figure 1.1 shows the distribution of FRP composites materials shipped in 2004. According to the *Composites News International* (ACMA 2005), the estimated size of the composites industry in North America is approximately $9 billion. The composites industry has shown considerable growth over the past 10 years, and is projected to increase as FRP composites are accepted in new markets.

1.2.2 *Product and benefits for construction applications*—FRP composites provide many solutions to the needs of the owner and civil engineer. FRP products for civil infrastructure/construction applications are more resistant to corrosion than reinforcing steel, thus the service life of the structure may be increased. FRP products have high strength-to-weight ratios and strength properties greater than those of steel. In repair and rehabilitation, the light weight and ease of application of the materials can result in saving in labor costs. The main drawback of the materials is their relatively high material cost.

Currently, many FRP products are available to build or repair civil engineering structures. These products have been extensively demonstrated and used around the world. Examples of FRP composite products include:

- FRP composite systems for repair, strengthening, and seismic retrofit for beams, columns, slabs, and walls;
- FRP reinforcing bars, grids, and tendons for concrete reinforcement;
- Bridge deck panels and pedestrian bridge systems;
- New structural shapes;
- Piling products and systems for marine waterfront structures;
- FRP dowel bars for durable long-term service in concrete highway pavements; and
- FRP tie connectors and FRP grid shear connectors for concrete sandwich wall construction.

Several examples of such applications are described in Chapter 13 of this report.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A_f = area of FRP reinforcement, mm^2 (in.^{2})
a = depth of equivalent rectangular stress block; length of shear span, mm (in.)
b = width of carbon FRP plate, mm (in.)
c = neutral axis depth, mm (in.)
D = diameter, mm (in.)
D_i = inner diameter of tube, mm (in.)
D_o = outer diameter of tube, mm (in.)
d = distance from extreme compression fiber to centroid of tensile reinforcement, mm (in.)
d_b = bar diameter, mm (in.)
E_c = modulus of concrete core, MPa (psi)
E_{FRP} = modulus of elasticity of FRP reinforcement, MPa (psi)
E_f = equivalent orthotropic elastic modulus of tube in hoop direction, MPa (psi)
E_s = elastic modulus of steel reinforcement, MPa (psi)
f'_c = specified concrete compressive strength, MPa (psi)
f'_{cc} = compressive strength of confined concrete, MPa (psi)
f_{fu} = ultimate strength of FRP material, MPa (psi)
f_{pi} = initial prestressing stress, MPa (psi)
h = depth of reinforced concrete beam, mm (in.)
I_{cr} = cracked moment of inertia of the section of a reinforced concrete beam, mm^4 (in.^{4})
I_e = effective moment of inertia, mm^4 (in.^{4})
I_{ey} = overall effective beam moment of inertia at yielding, mm^4 (in.^{4})
L = length of CFFT, mm (in.)
l_d = development length, mm (in.)
l_t = transfer length for prestressing tendons, mm (in.)
M_u = factored moment, kNm (lb-in.)
N = axial load, kN (lb)
R = radius of FRP tube, mm (in.)
T_g = glass transition temperature, °C (°F)
t = thickness of FRP plate, mm (in.)
V_c = nominal shear strength provided by concrete, kN (lb)
V_{exp} = experimentally obtained shear force, kN (lb)
Δ = deflection of beam subjected to load, mm (in.)
Δf_p = stress range in fatigue loading, MPa (psi)

ε_{cc} = axial strain to calculate confinement pressure in CFFT
ε_{FRP} = strain level in FRP reinforcement
$\varepsilon_{FRP\text{-}service}$ = strain level in FRP reinforcement in service
ε_{fe} = effective strain in FRP laminate
ε_{fu} = design rupture strain of FRP
ε_s = strain of steel reinforcement
ϕ = strength-reduction factor
ϕ_u = curvature of beam at failure, 1/mm (1/in.)
ϕ_y = curvature of beam at yielding of internal steel, 1/mm (1/in.)
θ = rotation of a section of a beam, radian or degree; or direction of fibers
κ_m = coefficient to calculate effective strain in FRP laminate
μ = ductility index
ρ = reinforcement ratio
ρ_{FRP} = FRP longitudinal reinforcement ratio
ρ_{fc} = fiber composite reinforcement ratio
σ_R = confinement pressure in circular CFFT, MPa (psi)
σ_x = hoop tensile strength, MPa (psi)
σ_y = axial compressive strength, MPa (psi)
τ_m = bond strength of FRP bar, MPa (psi)
ν_c = Poisson's ratio of concrete
ν_f = longitudinal Poisson's ratio of tube

2.2—Definitions

AFRP—aramid fiber-reinforced polymer.

aging—the process of exposing materials to an environment for an interval of time.

aramid fiber—highly oriented organic fiber derived from polyamide incorporating aromatic ring structure.

balanced FRP reinforcement ratio—the reinforcement ratio in a flexural member that causes the ultimate strain of FRP bars and the ultimate compressive strain of concrete (assumed to be 0.003) to be simultaneously attained.

bar—a composite material formed into a long, slender structural shape suitable for the internal reinforcement of concrete and consisting of primarily longitudinal unidirectional fibers bound and shaped by a rigid polymer resin material. The bar may have a cross section of variable shape (commonly circular or rectangular), and may have a deformed or roughened surface to enhance bonding with concrete.

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

BMC—bulk molding compound.

bond-critical applications—applications of FRP systems for strengthening structural members that rely on bond to the concrete substrate; flexural and shear strengthening of beams and slabs are examples of bond-critical applications.

braided string or rope—string or rope made by braiding continuous fibers or strands.

braiding—a process whereby two or more systems of yarns are wound together in the bias direction to form an integrated structure. Braided material differs from woven and knitted fabrics in the method of yarn introduction into the fabric and the manner by which the yarns are interlaced.

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.

carbon fiber—fiber produced by pyrolysis of organic precursor fibers. Used interchangeably with graphite. Types of carbon fibers include mesophase pitch carbon and pan carbon (polyacrylonitrile).

catalyst—organic peroxide used to activate the polymerization.

CET—coefficient of thermal expansion; change in linear dimension per unit length due to change in temperature.

CFFT—concrete-filled FRP tube.

CFRP—carbon fiber-reinforced polymer (includes graphite fiber-reinforced polymer).

composite—a combination of one or more materials differing in form or composition on a macroscale. Note: The constituents retain their identities; that is, they do not dissolve or merge completely into one another, although they act together. Normally, the components can be physically identified and exhibit an interface between one another.

concrete substrate—the original concrete or any cementitious repair material used to repair or replace the original concrete; the substrate can consist entirely of original concrete, entirely of repair materials, or of a combination of original concrete and repair materials; the substrate includes the surface to which the FRP system is adhered.

contact-critical applications—applications of FRP systems that rely on continuous intimate contact between the concrete substrate and the FRP system. In general, contact-critical applications consist of FRP systems that completely wrap around the perimeter of the section. For most contact-critical applications, the FRP system is bonded to the concrete to facilitate installation, but does not rely on that bond to perform as intended. Confinement of columns for seismic retrofit is an example of a contact-critical application.

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.

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

continuous filament tow—parallel filaments coated with sizing, drawn together into single or multiple strands, and wound into a cylindrical package.

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

coupling agent—part of a surface treatment or finish that is designed to provide a bonding link between the fiber surface and the laminating resin.

cps—centipoises, unit of viscosity. The standard unit is poise. For relatively low viscosities, the units are often referred to as centipoises (cps) or 0.01 poise. Water is the standard at 1 cps.

crimp—waviness of a fiber, a measure of the difference between the length of the unstraightened and straightened fibers.

cross-link—a chemical bond between polymer molecules. Note: An increased number of cross-links per polymer molecule increases strength and modulus at the expense of ductility.

cure of FRP systems—the process of causing the irreversible change in the properties of a thermosetting resin by chemical reaction. Cure is typically accomplished by addition of curing (cross-linking) agents or initiators, with or without heat and pressure. Full cure is the point at which a resin reaches the specified properties. Undercure is a condition where specified properties have not been reached.

curing agent—a catalytic or reactive agent that, when added to a resin, causes polymerization. Also called hardener or initiator.

deformability factor—the ratio of energy absorption (area under the moment-curvature curve) at ultimate strength of the section to the energy absorption at service level.

denier—measure of fiber diameter, taken as the weight in grams of 9000 m of the fiber.

durability—ability to resist cracking, oxidation, chemical degradation, delamination, wear, fatigue, and/or the effects of foreign object damage for a specified period of time, under the appropriate load conditions, and under specified environmental conditions.

E-glass—a family of glass with a calcium alumina borosilicate composition and a maximum alkali content of 2.0%.

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.

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.

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.

fiber content—the amount of fiber present in a composite. Note: This usually is expressed as a percentage volume fraction or weight fraction of the composite.

fiberglass—a composite material consisting of glass fibers in resin.

fiber-reinforced polymer (FRP)—composite material consisting of continuous fibers impregnated with a fiber-binding polymer then molded and hardened in the intended shape.

fiber volume fraction—the ratio of the volume of fibers to the volume of the composite.

fiber weight fraction—the ratio of the weight of fibers to the weight of the composite.

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 continuous rovings onto a rotating mandrel. The rovings may be dry or saturated with resin.

filler—a relatively inert substance added to a resin to alter its properties or to lower cost or density. Sometimes the term is used specifically to mean particulate additives.

fire retardant—chemicals that are used to reduce the tendency of a resin to burn; these can be added to the resin or coated on the surface of the FRP.

FRC—fiber-reinforced composite.

GFRP—glass fiber-reinforced polymer.

glass fiber—fiber drawn from an inorganic product of fusion that has cooled without crystallizing. Types of glass fiber include alkali-resistant (AR-glass); general purpose (E-glass); and high-strength (S-glass).

glass transition temperature—the midpoint of the temperature range over which an amphorous material (such as glass or a high polymer) changes from (or to) brittle, vitreous state to (or from) a rubbery state.

graphite fiber—fiber containing more than 99% crystalline carbon made from a precursor by oxidation.

grating—a planar FRP form. Gratings may be manufactured using molding methods or by mechanically assembling pultruded FRP elements (bars, I-shapes, or rods) together in two orthogonal directions to produce sheets.

grid—a planar FRP form in which continuous fibers are aligned in two orthogonal directions and combined with resin to produce a open mesh-like structure. Grids may be made using continuous manufacturing methods and supplied on rolls, or sheets made by molding methods and supplied in sheets.

GRP—glass-reinforced plastic.

hand lay-up—an open-mold manufacturing process in which resin is applied by hand, brush, or roller on to dry fiber reinforcements and exposed to the atmosphere for cure. This process can be done in a mold or performed on an object.

hardener—substance used to cure epoxy resins.

HFRP—hybrid FRP.

hybrid—a combination of two or more different fibers, such as carbon and glass or carbon and aramid, into a structure.

impregnation—saturation of voids and interstices of a reinforcement with a resin.

initiator—a substance, usually peroxide, that speeds up the curing of a resin.

interface—the boundary or surface between two different, physically distinguishable media. On fibers, the contact area between fibers and coating/sizing.

interlaminar shear—shearing force tending to produce a relative displacement between two laminae in a laminate along the plane of their interface.

isophthalic polyester resin—product of isophthalic acid, glycol, and maleic anhydride (Ashland Specialty Chemical 2006).

laminate—two or more layers of fibers, bound together in a resin matrix.

lay-up—the process of placing the FRP reinforcing material in position for molding.

LEED—Leadership in Energy and Environment Design—a certification program for sustainable building.

mat—a fibrous material for reinforced polymer, consisting of randomly oriented chopped filaments, short fibers (with or without a carrier fabric), or long random filaments loosely held together with a binder.

matrix—in the case of FRPs, the materials that serve to bind the fibers together, transfer load to the fibers, and protect them against environmental attack and damage due to handling.

MF-FRP—mechanically fastened FRP.

monomer—an organic molecule of relatively low molecular weight that creates a solid polymer by reacting with itself or other compounds of low molecular weight or both.

multifilament—yarn consisting of many continuous filaments.

NDI—nondestructive inspection.

NEFMAC—new fiber composite material for advanced concrete.

NSM—near-surface-mounted.

nylon—polyamide polymer that is thermoplastic in nature.

PAN—polyacrylonitrile, a precursor fiber used to make carbon fiber.

PAN carbon fiber—carbon fiber made from polyacrylonitrile (PAN) fiber.

phenolic resin—thermoset resin produced by condensation of aromatic alcohol.

pitch—a black residue from the distillation of petroleum.

pitch carbon fiber—carbon fiber made from petroleum pitch.

plastisol—a plastisol is a liquid dispersion of polyvinyl chloride resin in a plasticizer along with materials such as stabilizers, colorants, fillers, and other additives.

ply—a single layer of fabric or mat; multiple plies, when molded together, make up the laminate.

PMC—polymer matrix composite.

polyester—one of a large group of synthetic resins, mainly produced by reaction of dibasic acids with dihydroxy alcohols; commonly prepared for application by mixing with a vinyl-group monomer and free-radical catalysts at ambient temperatures and used as binders for resin mortars and concretes, fiber laminates (mainly glass), adhesives, and the like. Commonly referred to as unsaturated polyester.

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.

polymer—a high-molecular-weight organic compound, natural or synthetic, containing repeating units.

polymerization—the chemical reaction in which two or more molecules of the same substance combine to form a compound containing the same elements and in the same proportions but of higher molecular weight.

polyurethane—reaction product of an isocyanate with any of a wide variety of other compounds containing an active hydrogen group; used to formulate tough, abrasion-resistant coatings and matrixes.

postcuring, FRP—additional elevated-temperature curing that increases the level of polymer cross-linking; final properties of the laminate or polymer are enhanced.

pot life—time interval after preparation during which a liquid or plastic mixture is to be used.

precusor—the rayon, PAN, or pitch fibers from which carbon fibers are derived.

prepreg—semi-hardened fiber-matrix 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.

PVC—polyvinyl chloride.

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.

resin content—the amount of resin in a laminate, expressed as either a percentage of total mass or total volume.

resistance factor—factor applied to a specified material property or to the resistance of a member for the limit state under considerations, which takes into account the variability of dimensions, material properties, workmanship, type of failure, and uncertainty in the prediction of resistance.

roving—a number of yarns, strands, tows, or ends of fibers collected into a parallel bundle with little or no twist.

RTM—resin transfer molding.

SCRIMP—Seemans composite reinforcement infusion molding process—a vacuum process to combine resin and reinforcement in an open mold.

SFRP—steel FRP.

shape—construction made of continuous fibers in a shape other than used to reinforce concrete monoaxially, 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.

sheet, FRP—a dry, flexible ply used in wet lay-up FRP systems. Unidirectional FRP sheets consist of continuous fibers aligned in one direction and held together in-plane to create a ply of finite width and length. Fabrics are also referred to as sheets.

shelf life—the length of time packaged materials can be stored under specified conditions and remain usable.

SIP—Structurally integrated stay-in-place. Used to describe SIP form systems described in Chapter 9.

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; polyvinyl chloride (PVC); polyethylene (PE) (olefin).

textile—fabric, usually woven.

thermoplastic—polymer that is not cross-linked. Thermoplastic polymer generally can be remelted and recycled.

thermoset—resin that is formed by cross-linking polymer chains. A thermoset cannot be melted and recycled.

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.

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.

URM—unreinforced masonry.

UV—ultraviolet.

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

VARTM—vacuum resin transfer molding—a vacuum process to combine resin and reinforcement in an open mold.

volume fraction—the proportion from 0.0 to 1.0 of a component within the composite, measured on a volume basis, such as fiber-volume fraction.

weaving—a multidirectional arrangement of fibers. For example, polar weaves have reinforcement yarns in the circumferential, radial, and axial (longitudinal) directions; orthogonal weaves have reinforcement yarns arranged in the orthogonal (Cartesian) geometry, with all yarns intersecting at 90 degrees.

wet lay-up—a method of making a laminate product by applying the resin system as a liquid when the fabric or mat is put in place.

wet-out—the process of coating or impregnating roving, yarn, or fabric in which all voids between the strands and filaments are filled with resin; it is also the condition at which this state is achieved.

yarn—group of fibers held together to form a string or rope.

CHAPTER 3—CODES AND STANDARDS

This chapter provides an overview of available design documents addressing applications of FRP composite materials either as internal reinforcement or as an external repair material for concrete structures. For design guidance, the referenced design guidelines or codes should be consulted and take precedence over information in this report. With the exception of materials testing standards, the Canadian code documents (CSA S6 and S806), and the Egyptian code (Egyptian Ministry of Housing, Utilities, and Urban Development 2005), the available documents are intended to provide guidance for the use of FRP materials and do not carry the weight of design standards. In some cases, the issuing agency has chosen to term the documents Bulletins (fib 2001), Interim Guidance (IStructE 1999) or, in the case of ACI, Emerging Technology Documents (ACI 440.2R and 440.4R). Nonetheless, all are consensus documents and represent the current state of the practice. ACI removed the Emerging Technology designation from ACI 440.1R.

3.1—Materials

Most major standards-writing organizations have a number of available materials testing standards appropriate for determining a variety of FRP material properties and characteristics. Materials test standards, however, are always generic and are intended to set a standard of performance based on laboratory tests; therefore, they are not necessarily well suited to determine the in-place properties of interest to the concrete designer.

ACI Committee 440 has initiated the development of test methods specifically intended for FRP products and materials used with reinforced concrete. These ACI-developed standards are aimed at generating design values useful to the concrete practitioner. Once accepted by ACI, these test methods are submitted to ASTM Committee D30.05, "Composite Materials—Structural Test Methods" to be developed as formal ASTM standards. The first collection of ACI test methods is compiled in ACI 440.3R. ASTM D30.05 is beginning the process of revising and adopting these initial standards.

Most international standards organizations issue standards for determining basic physical properties of FRP materials. Many of these have been adapted by various industry and research interests for products with applications for concrete structures. Chapter 5 of this report discusses FRP materials testing more completely.

3.2—Internal FRP reinforcement

Fundamental design methodologies for FRP-reinforced concrete are similar to those of conventional steel-reinforced concrete. Cross-sectional equilibrium, strain compatibility, and constitutive material behavior form the basis of all code approaches to designing reinforced concrete, regardless of the reinforcing material. The nonductile and anisotropic natures of FRP reinforcing products, however, need to be addressed in design guidelines. In flexural design, for instance, the ultimate limit state may be defined either by FRP rupture or concrete crushing, provided that the strength and serviceability criteria are met. Because of the lack of ductility of such failure modes, a higher reserve of strength is required. Thus, strength-reduction factors or material-resistance factors are generally lower for FRP-reinforced concrete members than for steel-reinforced concrete members. In all cases, FRP design guidelines and codes are consistent with the applicable reinforced concrete design codes. No attempt is made to adjust load factors; thus, only strength-reduction factors or material-resistance factors are adjusted to reflect the use of FRP. Chapters 6 and 7 of this document discuss, in more detail, internally FRP-reinforced and FRP-prestressed concrete members, respectively.

ACI 440.1R uses a strength design approach for FRP-reinforced concrete members that is consistent with ACI

318. In this approach, a member is designed based on its required strength and checked for fatigue and creep rupture endurance, and serviceability criteria. In many cases, these latter criteria may control the design of members, particularly where lower stiffness aramid or glass reinforcement is used. ACI 440.1R, consistent with ACI 318, provides strength-reduction factors related to the ductility of the failure mode considered. Guidance, in the form of additional reduction factors, is also provided for all durability-related criteria with the exception of fire resistance. ACI 440.4R provides similar design guidance for prestressing structures with FRP tendons.

The CSA S806 represents the first stand-alone formalized design code (rather than guide) addressing FRP-reinforced concrete. Consistent with CSA A23.3, CSA S806 prescribes partial material-resistance factors rather than strength-reduction factors. Additionally, CSA S806 does not consider FRP rupture to be a valid failure, permitting only concrete crushing. The CSA S806 does not address durability directly, but rather references CSA S478 instead. CSA S806 requires the designer to consider fire resistance, but offers no provisions or guidance in this regard. CSA S806 also provides provisions for prestressing concrete members with aramid or carbon bars.

The CSA S6 also includes formalized provisions for the application of FRP reinforcement or prestressing. Material-resistance factors and stress limits associated with serviceability, pretensioning, and post-tensioning are provided. Durability is addressed through material-resistance factors for FRP type and application, and the effects of creep and fatigue are assumed to be included in the factors and limits provided.

The ISIS *Design Manual 3—Reinforcing Concrete Structures with Fibre Reinforced Polymers* (Rizkalla and Mufti 2001) takes an approach very similar to ACI 440.1R in permitting both FRP rupture and concrete crushing failures. *Design Manual 3* provides material-resistance factors, though it does not differentiate between expected failure modes. *Design Manual 3* discusses durability, but does not provide additional factors to account for environmental exposure. Creep and fatigue are addressed in the same manner as in CSA S6. Finally, *Design Manual 3* provides listings of product-specific material properties.

In Europe, the Federation Internationale de Beton (fib) Task Group 9.3, FRP Reinforcement for Concrete Structures, is preparing a technical bulletin addressing the use of internal FRP reinforcement. As of early 2007, this document is not yet available, but several European countries have developed their own guidelines for such applications.

In Great Britain, the *Interim Guidance on the Design of Reinforced Concrete Structures using Fibre Composite Reinforcement* (IStructE 1999) takes the form of suggested changes to the British design codes BS8110, "Structural Use of Concrete, Parts 1 and 2" (British Standards Institution [BSI] 1997) and BS5400, "Part 4 Code of Practice for the Design of Concrete Bridges" (BSI 1990). The approach taken by the IStructE document is consistent with that taken by CSA S6.

The Japanese "Recommendations for the Design and Construction of Concrete Structures using Continuous Fiber Reinforced Materials" (JSCE 1997) is intended to supplement the "JSCE Standard Specification for Concrete Structures" (JSCE 1986) with respect to the use of FRP reinforcement for structures other than buildings. The JSCE recommendations (1997) are based on limit states design with verification of the ultimate, serviceability, and fatigue limits states. Although limits associated with durability phenomena are prescribed, these are typically given in reference to the results of a standard material property test for the FRP bar or strand considered. "Design Guidelines of FRP Reinforced Concrete Building Structures" (Sonobe et al. 1997), provides similar guidelines for building structures. A third guide, "Design Methods for Prestressed FRP-Reinforced Concrete Building Structures," is reported (Bakis et al. 2002) although it does not appear to be available in English.

The Egyptian FRP code (Egyptian Ministry of Housing, Utilities, and Urban Development 2005) represents a stand-alone formalized design code addressing FRP-reinforced concrete that includes provisions for the design and construction of concrete reinforced with FRP bars. The Egyptian document uses the limit states design approach for FRP-reinforced concrete members that is consistent with the formalized Egyptian "Code for the Design and Construction of Concrete Structures" (2001). In this approach, a member is designed based on its required strength and checked for fatigue and creep rupture endurance and serviceability criteria. The Egyptian FRP code prescribes a material-resistance factors approach related to the ductility of the failure mode considered in addition to all durability-related criteria with the exception of fire resistance.

In North America, two approaches for evaluating the resistance of members are used. In the ACI approach, the strength-reduction factor depends on the action being resisted (for example, the reduction factor is different for flexure, shear, and axial loads). The strength-reduction factor (less than 1.0) is applied to the total nominal resistance, and depends on the mode of failure. On the other hand, Canadian codes are based on a unified limit state design philosophy in which material-resistance factors, always less than 1.0, are used to reflect uncertainties in determining the nominal resistance structural components. In CSA S6, the material-resistance factor for each FRP type changes with application and was arrived at by considering both material variability and environmental factors for those specific applications.

3.3—External FRP reinforcement

Recommended design methodologies of externally bonded FRP strengthening schemes are universally consistent with concrete design methodologies. Currently, the three formalized documents addressing applications of externally bonded FRP are CSA S806, CSA S6, and the Egyptian FRP code (Egyptian Ministry of Housing, Utilities, and Urban Development 2005). All other available documents are guides. The approaches taken, in each case, are consistent with local concrete codes and related guidance for internal FRP reinforcement (Section 3.2). Similar to internal FRP reinforcement, failure modes may be associated with concrete crushing or FRP rupture or loss of bond. Additionally, the initial strain conditions of the existing internal steel reinforcement should also be considered. Load-reduction factors and material-resistance factors are consistent with those recommended for internal FRP reinforcement. Chapter 8 of this document discusses externally bonded FRP reinforcement used for structural repair or retrofit in more detail.

Beyond simply providing the required capacity, three significant design issues, associated with applications of external FRP for repair and strengthening, are:

1. Quality of bond (or anchorage, or both) between FRP and concrete substrate;
2. The durability and long-term performance of FRP materials; and
3. Strengthening limits.

The consensus of available design guides is that each of these issues requires special attention and additional research. Additionally, most guides for design of externally bonded composites address only reinforced concrete rectangular sections. ACI Committee 440 is currently preparing documents that address concrete members having pre- or post-tensioned steel reinforcement, irregular geometry, compression FRP reinforcement, near-surface-mounted (NSM) FRP applications, or upgrade of masonry structures.

ACI 440.2R provides strength-reduction factors based on ductility of the expected failure mode (as measured by reinforcing steel strain) consistent with ACI 318. ACI 440.2R specifically addresses environmental exposure with environmental reduction factors associated with various typical exposures. Fatigue and creep effects are addressed through prescribed FRP stress limits. Cover delamination and debonding of the FRP are mitigated through strain limits applied to the FRP. These limits are believed to reduce the shear transfer demand of the cover concrete and adhesive layers sufficiently to prevent bond failure. Finally, *de facto* strengthening limits are determined by stipulating that the ultimate capacity of the structure without the FRP reinforcement should exceed a minimum level that corresponds to the expected loads under such situations. Additional guidance is provided for fire event loading requirements. These strengthening limits are based on the premise that the FRP retrofit is rendered entirely ineffective under particular conditions, such as fire or vandalism, and the member (without FRP) should be able to carry service loads without collapse. A revision of ACI 440.2R to include NSM reinforcement and to update load factors to those adopted in the ACI 318-05 (ACI Committee 318 2005) is underway. Similarly, a separate document addressing the repair of masonry using externally bonded FRP reinforcement is in preparation.

According to ACI 440.2R, the shear strength can be improved by wrapping the FRP system around three sides of the member (U-wrap), bonding to the two sides of the member, or completely wrapping the member. The FRP system can be installed continuously along the span length of a member or placed as discrete strips. The nominal shear capacity of an FRP-strengthened concrete member can be determined by adding contributions from the reinforcing steel (stirrups, ties, or spirals) and concrete and the contribution of the FRP reinforcement based on a maximum FRP strain at ultimate that is no greater than 0.004. FRP contribution for shear is further reduced using a reduction factor that is less than 1.0.

For compression members, ACI 440.2R allows FRP systems to be used to increase the axial compression strength of a concrete member by providing confinement with lateral (hoop) fibers. The hoop fibers provide confining effects similar to conventional spiral or tie reinforcing steel. ACI 440.2R uses a simplified approach to determine the peak confined concrete strength as a function of the FRP confining pressure. The effectiveness of FRP confinement is determined as a function of the geometry of the section of the confined member. An efficiency factor is introduced to address the shape effects. For circular sections, the efficiency factor can be taken equal to 1.0.

ACI 440.2R gives some guidelines for detailing FRP sheets or laminates to avoid bond-related failures. The details depend on the geometry of the structure, the soundness and quality of substrate, and levels of load to be sustained by the FRP sheets or laminates.

For fatigue, ACI 440.2R suggests fatigue stress limits on FRP laminates to avoid fatigue failures of the FRP. The stress limits are lowest for glass FRP (GFRP), and highest for carbon FRP (CFRP). Existing recommendations for stress range limits for reinforced concrete beams (ACI 215R) may be applied to the internal reinforcing steel, but these typically limit the service stresses in the steel and concrete to levels that may be exceeded in cases where the structure is strengthened. Indeed, exceeding these stress limits may be the reason for requiring strengthening. Finally, it should also be taken into account that by the time strengthening takes place, the structure may have been in service for a significant number of years, exhausting some of its fatigue life.

To calculate the design strength of a strengthened member, the ACI approach uses strength-reduction factors applied to the nominal resistance, while other codes use material-resistance factors applied to the strength properties of the materials themselves (including FRPs). In addition to an overall strength-reduction factor, ACI 440.2R applies an additional partial resistance factor to the portion of the strength contributed by the FRP reinforcement. To account for durability, ACI 440.2R also recommends environmental reduction factors to reduce the strength of the FRP that can be used in design. These factors depend on the type of FRP and the exposure conditions for the element to be strengthened.

The International Code Council (ICC) Evaluation Service, Inc. has acceptance criteria AC-125 and AC-178 (ICC 2003a,b). Although these criteria are not codes or standards, they are documents intended to evaluate FRP materials as alternatives to code-approved materials. They are developed in an open forum at public meetings presided by building officials.

The Canadian CSA S806 represents the first stand-alone formalized design code (rather than guide) addressing externally bonded FRP reinforcement for concrete. For strengthening applications, CSA S806 considers all possible failure modes rather than just concrete crushing (as it does for internal reinforcement). Durability is addressed through reference to CSA S478. Although a single limiting FRP tensile strain (0.007) is provided, delamination and debonding are addressed by directing the designer to "currently available information appropriate to the combination of sheets and adhesive." Strengthening limits are based on the requirement that the structure should support its nominal service load in the event that the FRP becomes ineffectual. CSA S806 addresses the repair of masonry structures.

ISIS *Design Manual 4—Strengthening Reinforced Concrete Structures with Externally-Bonded Fibre Reinforced Polymers* (Neale 2001) provides considerable guidance and a number of design examples for the use of externally bonded FRP. This document, however, is written as a state-of-the-art report, referencing the recommendations of others rather than making its own clear design recommendations. Generally, ISIS *Design Manual 4* references the recommendations of ACI 440.2R (which was in draft form when ISIS *Design Manual 4* was released).

The European Fédération Internationale de Béton (fib) *Bulletin* 14 (fib 2001), produced by fib Task Group 9.3, also represents a combination guide and state-of-the-art report. Significantly, fib *Bulletin* 14 recognizes the difference in expected performance, not only between FRP material types, but between preformed and wet layed-up FRP systems in the form of different material safety factors. Delamination and debonding are extensively addressed in fib *Bulletin* 14 using a simplified bilinear bond model and by also addressing the effects of the loss of composite action between the FRP and concrete substrate. Durability effects are discussed, but no clear design guidance is provided. FRP-strengthened members are expected to withstand accidental loads in the case where the FRP no longer functions. In this case, reference is made to "Eurocode 1" (CEN 1994) for the loads to be considered.

In Great Britain, the Concrete Society published *Technical Report* 55 (Concrete Society 2000, 2004). This document is similar to ISIS *Design Manual 4* (Neale 2001) and fib *Bulletin* 14 (2001) in its approach and scope. Importantly, *Technical Report* 55 has been followed by *Technical Report* 57 (Concrete Society 2003). This report addresses more practical construction issues associated with externally bonded FRP materials.

The Japanese "Recommendations for the Upgrading of Concrete Structures with Use of Continuous Fiber Sheets" (JSCE 2001) adopts a performance-based approach to the design of externally bonded FRP materials. Flexure and shear capacity, column deformation capacity (jacketing applications), flexural crack width, and protection of the concrete substrate from chloride ion penetration are all considered explicitly in the JSCE recommendations. A variety of guidelines produced by various Japanese authorities are also available primarily for column and/or pier retrofit using FRP jacketing techniques (Bakis et al. 2002).

The Egyptian FRP code (Egyptian Ministry of Housing, Utilities, and Urban Development 2005) also addresses externally bonded FRP reinforcement for concrete. The approach is very similar to that of ACI 440.2R.

The Italian "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures" (CNR 2004) includes the application of externally bonded FRP materials to reinforced or prestressed concrete structures as well as to masonry structures. Delamination failure mechanisms are explained, and a fracture mechanics approach is adopted to predict such failure modes; based on that approach, an ultimate design strength for laminate/sheet is provided. This document describes a detailed design example of strengthening a two-story building with CFRPs.

CHAPTER 4—COMPOSITE MATERIALS AND PROCESSES

4.1—Introduction

FRP composites are defined as a polymer matrix, either thermoset or thermoplastic, that is reinforced with a fiber or other reinforcing material with a sufficient aspect ratio (length-to-thickness) to provide a discernible reinforcing function in one or more directions. FRP composites are different from traditional construction materials such as steel or aluminum. FRP composites are anisotropic (properties vary with the direction), whereas steel or aluminum are isotropic (uniform properties in all directions, independent of applied load). Therefore, FRP composite properties are directional, and typically the most favorable mechanical properties are in the direction of the fiber placement.

Many terms have been used to define FRP composites. Modifiers have been used to identify a specific fiber such as glass FRP (GFRP), carbon FRP (CFRP), aramid FRP (AFRP), steel FRP (SFRP), and hybrid FRP (HFRP) for composites containing different types of fibers. In addition, other acronyms were developed over the years; their use depended on geographical location or market use. For example, fiber-reinforced composites (FRCs), glass-reinforced plastics (GRPs), and polymer matrix composites (PMCs) are found in many references.

Although these composites are defined as a polymer matrix that is reinforced with fibers, this definition should be further refined when describing composites in structural applications. In the case of structural applications such as FRP composite reinforced concrete, at least one of the constituent materials should be a continuous reinforcement phase supported by a stabilizing matrix material. For the special class of matrix materials discussed herein (that is, thermosetting polymers), the continuous fibers will usually be stiffer and stronger than the matrix.

Composite materials, in the sense that they will be discussed 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 will also briefly address additives and fillers, as well as process considerations.

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).

Each of the constituent materials or ingredients plays an important role in the processing and final performance of the end product. The resin or polymer holds the fibers in place and influences the physical properties of the end product. The reinforcement provides the mechanical strength. The fillers and additives are used as process or performance aids to impart special properties to the end product.

The mechanical properties and composition of FRP composites can be engineered for their intended use. The type and quantity of materials selected, in addition to the manufacturing process to fabricate the product, will affect the mechanical properties and performance. Important considerations for the design of composite products include:

- Type and percentage of fiber or fiber volume;
- Orientation of fiber (0, 90, ±45 degrees, or a combination of these);
- Type of resin;
- Cost of product;
- Volume of production (to help determine the best manufacturing method); and
- Service conditions.

4.2—Polymer matrix: resins

Resins are divided into two major groups: thermoset and thermoplastic. Thermoplastic resins become soft when heated, and may be shaped or molded while in a heated semi-fluid state. They become rigid when cooled. Thermoset resins, on the other hand, are usually liquids or low-melting-point solids in their initial form. When used to produce finished goods, these thermosetting resins are cured with a catalyst, heat, or a combination of the two. Once cured, solid thermoset resins cannot be converted back to their original liquid form. Unlike thermoplastic resins, cured thermosets will not melt and flow but will soften when heated (and lose hardness). Once formed, they cannot be reshaped. Heat distortion temperature and the glass transition temperature T_g are used to measure the softening of a cured resin. Test methods for both heat distortion temperature and T_g measure the approximate temperature where the cured resin will soften significantly to yield (bend or sag) under load.

The most common thermosetting resins used in the composites industry are unsaturated polyesters, epoxies, vinylesters, and phenolics.

4.2.1 *Polyester*—Unsaturated thermoset polyester resins are the workhorses of the composites industry, and they represent approximately 75% of the total resins used (ACMA MDA 2006). To avoid any confusion in terms, readers should be aware that there is an unrelated family of thermoplastic polyesters that are best known for their use as fibers for textiles and clothing. Polyesters are produced by the condensation polymerization of dicarboxylic acids and difunctional alcohols (glycols). In addition, unsaturated polyesters contain an unsaturated material, such as maleic anhydride or fumaric acid, as part of the dicarboxylic acid component. The finished polymer is dissolved in a reactive monomer such as styrene to give a low-viscosity liquid. When this resin is cured, the monomer reacts with the unsaturated sites on the polymer, converting it to a solid thermoset structure.

Polyesters are versatile because of their capacity to be modified or tailored during the building of the polymer chains. They have been found to have almost unlimited usefulness in all segments of the composites industry. The principal advantage of these resins is a balance of properties (including mechanical, chemical, and electrical), dimensional stability, cost, and ease of handling or processing.

Unsaturated polyesters are divided into classes depending upon the structure of their basic building blocks. Some common examples would be orthophthalic (ortho), isophthalic (iso), dicyclopentadiene, and bisphenol fumarate resins. In addition, polyester resins are classified according to end-use application as either general purpose or specialty polyesters.

The general-purpose polyesters define products that are relatively low in cost, offer good mechanical and electrical performance, and have a well-defined set of processing/fabricating characteristics. Almost all ortho and dicyclopentadiene resins and some iso resins fall into the general-purpose category.

Because polyesters can be chemically tailored to meet the requirements of a wide range of applications, a number of specialty polyesters that address specific performance, such as flexibility, electrical insulation, corrosion resistance, heat resistance, fire retardancy, and optical translucence, are available.

4.2.2 *Epoxy*—Epoxy resins have a well-established record in a wide range of composite parts, structures, and concrete repair. The structure of the resin can be engineered to yield a number of different products with varying levels of performance. A major benefit of epoxy resin over unsaturated polyester resins is its lower shrinkage. Epoxy resin can also be formulated with different materials or blended with other epoxy resins to achieve specific performance features. Cure rates can be controlled to match process requirements through the proper selection of hardeners and/or catalyst systems. Generally, epoxies are cured by addition of an anhydride or an amine hardener as a two-part system. Different hardeners, or different quantities of hardener, produce different cure profiles and give different properties to the finished composite.

Epoxies are primarily used for fabricating high-performance composites with superior mechanical properties, resistance to corrosive liquids and environments, superior electrical properties, good performance at elevated temperatures, and excellent adhesion to a substrate. Epoxy resins do not, however, have particularly good ultraviolet (UV) resistance. Because the viscosity of epoxy is much higher than that of most polyester resins, they require a post-cure (elevated heat) to obtain ultimate mechanical properties. Epoxies have little or no odor as compared with polyesters.

Epoxy resins are used with a number of fibrous reinforcing materials, including glass, carbon, and aramid. Epoxies are compatible with most composite manufacturing processes, particularly vacuum-bag molding, autoclave molding, pressure-bag molding, compression molding, filament winding, and hand lay-up. Currently, epoxy is the predominant resin used in external repair of concrete using FRP sheet and fabric products.

4.2.3 *Vinylester*—Vinylesters were developed to combine the advantages of epoxy resins with those of unsaturated polyester resins. These resins are produced by reacting epoxy resin with acrylic or methacrylic acid. This provides an unsaturated site, much like that produced in polyester resins when maleic anhydride is used. The resulting material is dissolved in styrene to yield a liquid that is similar to polyester resin. Vinylesters are also cured with the conventional organic peroxides used with polyester resins. Vinylesters offer mechanical toughness and excellent corrosion resistance. These enhanced properties are obtained without complex processing, handling, or special shop fabricating practices that are typical with epoxy resins.

4.2.4 *Phenolic*—Phenolics are a class of resins commonly based on phenol (carbolic acid) and formaldehyde, and cure through a condensation reaction producing water that should be removed during processing. Phenolic composites have many desirable performance qualities, including high temperature resistance, creep resistance, excellent thermal insulation and sound-damping properties, corrosion resistance, and excellent fire/smoke/smoke toxicity properties. Phenolics are applied as adhesives or matrix binders in engineered woods (plywood), brake linings, clutch plates, and circuit boards. Phenolic composites are not currently widely available for the construction industry.

4.2.5 *Structural engineered polyvinyl chloride (PVC) plastisol*—Structural engineered PVC plastisol resins are, by definition, thermoplastic resins with many performance characteristics of thermosets but with many more processing-friendly attributes than traditional thermoset resins. This new resin is a single-component material with no volatiles. This resin has the potential of impacting FRP composites products; however, to date, no products have been made or tested. It features a viscosity of approximately 1.5 to 3 Pa·S (1500 to 3000 cps) with a shelf life in excess of 6 months with essentially no settling of solids. To fully fuse (or cure) this material, it is subjected to temperatures between 177 and 199 °C (350 and 390 °F) for between 10 seconds and several minutes depending on the thickness of the part and the speed of the operation. Wetout of traditional glass and carbon fibers is effective, with chemical bonding occurring between the silanes present in the sizings and the fully fused PVC plastisol.

From a processing standpoint, PVC plastisol resins are efficient in handling liquid resins and minimizing waste associated with startup and shutdown. Structural engineered PVC plastisol resins are formulated with no styrene, solvent, or volatiles. Additive packages offer additional benefits of excellent UV resistance and fire retardancy. Additionally, there is virtually no microvoiding in the finished part, which ensures much better electrical properties and significantly lower water absorption compared with traditional thermosets. The resin exhibits a T_g of approximately 63 °C (145 °F).

4.3—Reinforcing fibers

The primary function of fibers or reinforcements is to carry load along the length of the composite to provide strength and stiffness in one direction. Reinforcements can be oriented to provide tailored properties in various directions. Reinforcements can be both natural and synthetic. Most commercial reinforcements, however, are synthetic. The principal types of fibers in commercial use for civil engineering applications are glass, carbon (or graphite), and aramid.

Of these, glass fiber has by far the largest volume of reinforcement measured either in quantity consumed or in product sales. Other composite reinforcing materials include ultra-high molecular weight polyethylene, polypropylene, polyester, and nylon.

The most common form of FRPs used in structural applications is called a laminate. Laminates are made by stacking a number of thin layers (lamina) of fibers and matrixes 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 unidirectional or one-dimensional fiber arrangement is transversely isotropic. 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 (orthotropic). A three-dimensional array 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.

Most reinforcement for either thermosetting or thermoplastic resins receives some form of surface treatment, either during fiber manufacture or as a subsequent treatment. Other materials applied to fibers as they are produced include resinous binders to hold fibers together in bundles and lubricants to protect fibers from degradation caused by process abrasion.

4.3.1 *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, and other formulations for high strength (S-2® glass), improved acid resistance (ECR glass), and alkali resistance (AR glass).

Based on an alumina-lime-borosilicate composition, E-glass fibers are considered the predominant reinforcement for polymer matrix composites because of their high electrical insulating properties, low susceptibility to moisture, and high mechanical properties.

Glass fibers used for reinforcing composites generally range in diameter from 9 to 23 microns. Fibers are drawn at high speeds through small holes in electrically heated bushings. These bushings form the individual filaments. The filaments are gathered into groups or bundles called strands or tows. The filaments are water and air cooled and then coated with a proprietary chemical binder or sizing to protect the filaments and enhance the composite laminate properties. The sizing also determines the processing characteristics of the glass fiber and the conditions at the fiber-matrix interface in the composite.

Glass is generally a good impact-resistant fiber, but is denser than carbon or aramid. Composites made from this material exhibit very good electrical and thermal insulation properties. Glass fibers are also transparent to radio frequency radiation and are used in radar antenna applications.

4.3.2 *Carbon fibers*—Carbon fiber is made from polyacrylonitrile (PAN), pitch, or rayon fiber precursors. 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 1000 °C (1830 °F).

There are two basic types of carbon fiber: high modulus and high strength. The difference in properties between them is a result of the differences in fiber microstructure, derived from the precursor type and the processing temperature (1000 to 3000 °C [1830 to 5430 °F]).

PAN-based carbon fiber is the predominant form used in the civil engineering venue because of its very high strength (up to about 5000 MPa [720 ksi] in unidirectional form) and relatively high modulus (up to about 140 GPa [20 msi] in unicomposite form or 420 GPa [60 msi] in fiber form). Pitch-based carbon fibers have extremely high modulus values (up to 970 GPa [140 msi]) with lower strengths (up to about 690 MPa [100 ksi] in unicomposite form). They are used primarily in space and satellite applications. Carbon fiber is about five to 10 times more expensive than glass fiber; however, it has about twice the usable strength and four times the modulus of glass.

Carbon fibers are supplied in a number of different forms, from continuous filament tows to chopped fibers and mats. The highest strength and modulus are obtained from unidirectional continuous reinforcement. Carbon fiber composites have a lower strain capacity than do glass or aramid composites. Carbon fiber is highly resistant to alkali or acid attack. Nevertheless, carbon fibers can cause galvanic corrosion when in contact with metals. A barrier material, such as glass and resin, is used to prevent this occurrence. Also, carbon fibers can conduct electric current, and are thus vulnerable to strikes by lightening.

Rayon and isotropic pitch precursors are used to produce low-modulus carbon fibers (50 GPa [7000 ksi]). Both PAN and liquid crystalline pitch precursors are made into higher modulus carbon fibers by carbonizing above 800 °C (1400 °F). Fiber modulus increases with heat treatment from 1000 to 3000 °C (1830 to 5430 °F). The results vary with the precursor selected. Fiber strength appears to maximize at a lower temperature of 1500 °C (2730 °F) 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 to 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. 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.

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. These tows can be woven or knitted into fabrics. Carbon fiber is also available as a prepreg where the fibers are preimpregnated with resin, as well as in the form of unidirectional tow sheets.

4.3.3 *Aramid fibers*—Aramid fiber is an aromatic polyamide organic fiber for composite reinforcement. Aramid fibers offer good mechanical properties at a low density with the added advantage of toughness or impact resistance. They are characterized as having reasonably high tensile strength, a medium modulus, and a very low density as compared with glass and carbon. The tensile strength of aramid fibers is higher than that of glass fibers, and the modulus is approximately 50% higher than that of glass. These fibers increase the impact resistance of composites and provide products with higher tensile strengths. Aramid fibers are insulators of both electricity and heat. They are resistant to organic solvents, fuels, and lubricants. Aramid composites have poor compressive strength. Dry aramid fibers are tough, have been used as cables or ropes, and are frequently used in ballistic applications.

4.3.4 *Steel fibers*—A new type of FRP composite strengthening system has emerged that uses high-strength steel fibers and is commonly known as SFRP. The high-strength steel fibers demonstrate a linear elastic stress-strain relationship that is similar to carbon and glass fibers. The steel fibers (also referred to as wires) have a tensile strength in the range of 2400 to 3100 MPa (350 to 450 ksi) and an elastic modulus of 200 MPa (30,000 ksi). The high-strength steel wires are twisted together to form steel cords, each containing five to 13 wires. Typical wire diameters range from 0.5 to 1.3 mm (0.02 to 0.05 in.). Because of the unwinding effects of the cords at higher tensile loads, ultimate tensile strains in the range of 0.02 to 0.05 can be achieved. The twisted steel wires have a deformed surface that allows for adequate bond to the surrounding matrix. SFRP is currently being applied for strengthening of concrete structures in a similar manner to other externally bonded FRP materials (Casadei et al. 2005a,b; Prota et al. 2006a,b).

Typical properties of glass, carbon, and aramid fibers are reported in Table 4.1.

4.4—Types of reinforcement

Regardless of the material, reinforcements are available in forms to serve a wide range of processes and end-product requirements. Fibers supplied as reinforcement include (untwisted) roving; (twisted) yarns; milled fiber; chopped strands; and continuous, chopped, or thermoformable mats. Reinforcement materials can be designed with unique fiber architectures and be preformed or shaped depending on the product requirements and manufacturing process.

4.4.1 *Multi-end and single-end rovings*—Multi-end rovings consist of many individual strands or bundles of filaments, which are then chopped and randomly deposited into the resin matrix. They can also be woven into fabric reinforcement. Processes such as sheet molding compound (SMC) and spray-up use multi-end rovings. Multi-end rovings can also be used in some filament winding and

Table 4.1—Typical material properties of fibers

Fiber	Typical diameter, microns	Density, g/cm^3 (lb/in.^{3})	Tensile modulus, GPa (10^6 psi)	Tensile strength, MPa (ksi)	Strain to failure, %	Coefficient of thermal expansion, 10^{-6}/°C	Poisson's ratio
Commercial composite reinforcing fibers (constructed from Mallick [1988])							
Glass							
E-glass	10	2.54 (0.092)	72.4 (10.5)	3450 (500.0)	4.8	5.0	0.2
S-glass	10	2.49 (0.090)	86.9 (12.6)	4300 (625.0)	5.0	2.9	0.22
Carbon PAN-carbon							
T-300	7	1.76 (0.064)	231 (34)	3650 (530)	1.4	–0.1 to –0.5 (longitudinal), 7 to 12 (radial)	–0.2
AS	7	1.77 (0.064)	220 (32)	3100 (450)	1.2	–0.5 to –1.2 (longitudinal), 7 to 12 (radial)	—
t-40	6	1.81 (0.065)	276 (40)	5650 (820)	2.0	—	—
HSB	7	1.85 (0.067)	345 (50)	2340 (340)	0.58	—	—
Fortafil 3™	7	1.80 (0.065)	227 (33)	3800 (550)	1.7	–0.1	—
Fortafil 5™	7	1.80 (0.065)	345 (50)	2760 (400)	0.8	—	—
Toray M40J	—	1.77 (0.064)	377 (55)	4410 (640)	1.2	—	—
Zoltek (2006)	7	1.81 (0.065)	242 (35)	3800 (550)	—	—	—
Pitch-carbon							
P-555	10	2.00 (0.072)	380 (55)	1900 (275)	0.5	–0.9 (longitudinal)	—
P-100	10	2.16 (0.078)	758 (110)	2410 (350)	0.32	–1.6 (longitudinal)	—
Aramid							
Kevlar™ 49	12	1.45 (0.052)	131 (19)	3620 (525)	2.8	–2.0 (longitudinal), +59 (radial)	0.35
Twaron™ 1055*	12	1.45 (0.052)	127 (18)	3600 (533)	2.5	–2.0 (longitudinal), +59 (radial)	0.35

*Mechanical properties: single filament at 22 °C (72 °F) per ASTM D 2101.

pultrusion applications. The single-end roving consists of many individual filaments wound into a single strand. This product is generally used in processes that use a unidirectional reinforcement such as filament winding or pultrusion. These manufacturing processes are described in Section 4.8.

4.4.2 *Mats*—These are two-dimensional random arrays of chopped strands. The fiber strands are deposited onto a continuous conveyor and then passed 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 vinylester matrix, thereby allowing the mat to conform to the shape of the mold.

4.4.3 *Woven, stitched, and braided fabrics*—Many types of fabrics can be used to reinforce resins in a composite. Multidirectional reinforcements are produced by weaving, knitting, stitching, or braiding continuous fibers into a fabric from twisted and plied yarns. Fabrics refer to all flat-sheet, rolled goods. Fabrics can be manufactured with almost any reinforcing fiber. The most common fabrics are constructed with glass, carbon, or aramid fibers. Fabrics are available in several weave constructions and thicknesses from 0.025 to 10.2 mm (0.0010 to 0.40 in.). They offer oriented strengths and high reinforcement loadings often found in high-performance applications.

Fabrics are typically supplied on rolls of 23 to 274 m (25 to 300 yd) in length and 25 to 3050 mm (1 to 120 in.) in width. The fabric should be inherently stable enough to be handled, cut, and transported to the mold, but pliable enough to conform to the mold shape and contours. Properly designed, the fabric will allow for quick wet-out and wet-through of the resin, and will stay in place once the resin is applied. Fabrics, like rovings and chopped strands, come with specific sizings or binder systems that promote adhesion to the resin system.

Woven fabrics are fabricated on looms in a variety of weights, weaves, and widths. In a plain weave, each fill yarn or roving is alternately crossed over and under each fiber in the other direction, allowing the fabric to conform to curved surfaces. Woven fabrics are manufactured where half of the fiber strands are laid at right angles to the other half (0 to 90 degrees). Woven fabrics are commonly used in boat manufacturing.

Stitched fabrics, also known as nonwoven, noncrimped, stitched, or knitted fabrics, have optimized strength properties because of the fiber architecture. Stitched fabrics are produced by assembling successive layers of aligned fibers. Typically, the available fiber orientations include the 0-degree direction (warp), 90-degree direction (weft or fill), and ±45-degree direction (bias). The assembly of each layer is then sewn together. These fabrics have been traditionally used in boat hulls. Other applications include light poles, wind turbine blades, trucks, busses, and underground tanks. These fabrics are currently used in bridge decks and column repair systems. Multiple orientations provide a quasi-isotropic reinforcement.

Braided fabrics are engineered with a system of two or more yarns intertwined in such a way that all of the yarns are interlocked for optimum load distribution. Biaxial braids provide reinforcement in the bias direction only with fiber angles ranging from ±15 to ±95 degrees. Triaxial braids provide reinforcement in the bias direction with fiber angles ranging from ±10 to ±80 degrees and axial (0-degree) direction.

4.4.4 *Unidirectional*—Unidirectional reinforcements include tapes, tows, tow sheets, rovings (which are collections of fibers or strands), grids, and pultruded bars. Fibers in this form are all aligned parallel. The aligned fibers are held together with a very light crossweave or tacked in place with a thermoplastic cross yarn or other light carrier. Composites using unidirectional tapes or sheets have high strength in the direction of the fiber. Unidirectional sheets are thin, and multiple layers are required for most structural applications.

Unidirectional sheets of carbon, glass, and aramid fibers are used extensively for wrapping of concrete beams and columns, while unidirectional reinforcing bars or prestressing tendons are applied for reinforcing or prestressing concrete structures, respectively.

4.5—Additives and fillers

4.5.1 *Additives and modifiers*—A wide variety of additives are used in composites to modify material properties and tailor the performance of the laminate. Although these materials are generally used in relatively low quantity by weight compared with resins, reinforcements, and fillers, they perform critical functions. These include affecting shrinkage, fire resistance, emissions, viscosity, mold release, and coloration.

4.5.2 *Fillers*—Fillers reduce the cost of composites; they also frequently impart performance improvements that might not otherwise be achieved by the reinforcement and resin ingredients alone. Fillers can improve mechanical properties, including fire and smoke performance, by reducing organic content in composite laminates. Also, filled resins shrink less than unfilled resins, thereby improving the dimensional control of molded parts. Important properties, including water resistance, weathering, surface smoothness, stiffness, dimensional stability, and temperature resistance, can all be improved through the proper use of fillers.

Commonly used fillers include calcium carbonate, clay, alumina trihydrate, and calcium sulfate. Others include mica, feldspar, wollastonite, silica, glass microspheres, and flake glass.

When used in composite laminates, inorganic fillers can account for 40 to 65% of the composite by weight. In comparison to resins and reinforcements, fillers are the least expensive of the major ingredients.

4.6—Core materials for sandwich structures

Bonded sandwich structures have been a basic component of the composites industry for over 45 years. The concept of using relatively thin, strong face sheets bonded to thicker, lightweight core materials has allowed the industry to build strong, stiff, light, and highly durable structures that otherwise would not be practical. This technology has been demonstrated in boats, trucks, building panels, and, more recently, in bridge decks.

Face sheets can be of almost any material. In the composites industry, the most common face sheet laminates are glass and carbon. The common core materials are foam, syntactic foam, honeycomb, and balsa wood (Fig. 4.1). Some core materials can be shaped, such as a waffle pattern or corrugation, to achieve the desired mechanical properties.

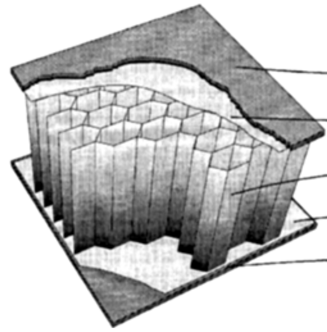


Fig. 4.1—Honeycomb sandwich construction (Hohe et al. 2001).

4.7—Adhesives

Adhesives are used to attach composites to themselves as well as to other surfaces such as concrete. Adhesive bonding is the method of choice for bonding thermoset composites. Several considerations are involved in applying adhesives effectively. The joint or interface connection should be engineered to select the proper adhesive and application method to ensure bond strength. Careful surface preparation and cure are critical to bond performance.

The most common adhesives are acrylics, epoxies, and urethanes. Epoxies provide a high-strength bond with high temperature resistance, whereas acrylics provide moderate temperature resistance with good strength and rapid curing. For applications where toughness is needed, urethane may be selected.

4.8—FRP manufacturing processes

Unique to the composites industry is the ability to create a product from many different manufacturing processes. There are a wide variety of processes available to the composites manufacturer to produce cost-efficient products. Each of the fabrication processes has characteristics that define the type of products to be produced.

4.8.1 *Pultrusion*—Pultrusion is a continuous molding process that combines fiber reinforcements and thermosetting resin. The pultrusion process is used in the fabrication of composite parts that have a constant cross-sectional profile. Typical examples include various rods and bar sections, bridge beams, and decks. Most pultruded laminates are formed using rovings aligned along the major axis of the part. Various continuous strand mats, fabrics (braided, woven, and knitted), and texturized or bulked rovings are used to obtain strength in the cross axis or transverse direction.

The process is normally continuous and highly automated (Fig. 4.2). Reinforcement materials, such as rovings, mats, or fabrics, are positioned in a specific location using preforming shapers or guides to form the profile. The reinforcements are drawn through a resin bath or wet-out where the material is thoroughly coated or impregnated with a liquid thermosetting resin. The resin-saturated reinforcements enter a heated metal pultrusion die. The dimensions and shape of the die will define the finished part being fabricated. Inside the metal die, heat is transferred under precise temperature control to the reinforcements and liquid resin. The heat activates the

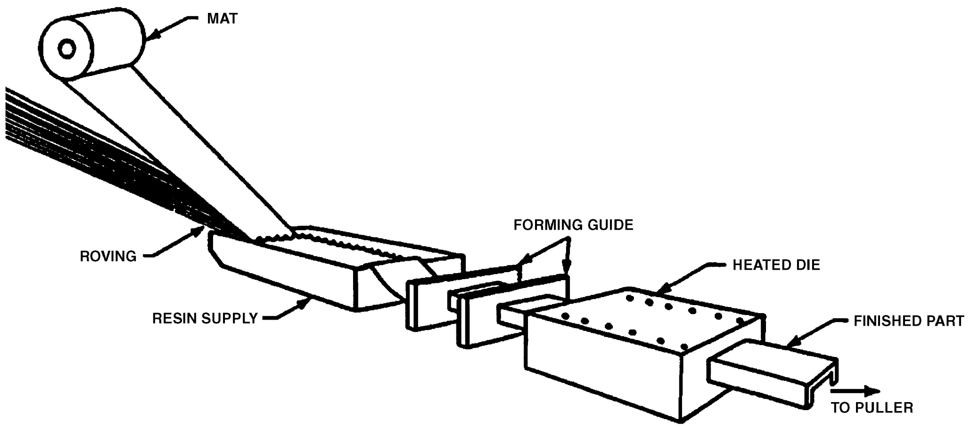


Fig. 4.2—Pultrusion process (Creative Pultrusions, Inc. 1994).

curing or polymerization of the thermoset resin, changing it from a liquid to a solid. The solid laminate emerges from the pultrusion die to the exact shape of the die cavity. The laminate solidifies when cooled, and it is continuously pulled through the pultrusion machine and cut to the desired length. The process is driven by a system of caterpillar or tandem pullers located between the die exit and the cutoff mechanism.

The process provides flexibility in the design of pultruded profiles. Currently, profiles up to 1830 mm (72 in.) wide and 530 mm (21 in.) high are possible. Pultrusion can manufacture both simple and complex profiles, eliminating the need for extensive post-production assembly of components. Because the process is continuous, length variations are limited to shipping capabilities. This process allows for optimized fiber architectures with uniform color, eliminating the need for many painting requirements.

4.8.2 *Filament winding*—This process takes continuous fibers in the form of parallel strands (rovings), impregnates them with matrix resin, and winds them on a rotating cylinder. The resin-impregnated rovings traverse back and forth along the length of the cylinder. The moving relationship between the rotating surface and the roving/matrix is usually controlled by a computer. A controlled thickness, wind angle, and fiber volume fraction laminate is thereby created. The material is cured on the cylinder and then removed (Fig. 4.3).

Pipes, torsion tubes, rocket cases, pressure bottles, storage tanks, and airplane fuselages are made by this process. For the construction industry, pipes are after-filled with concrete for piles and offer structural systems as described in Chapter 9. Filament winding is also used for concrete column repair (Chapter 8). There can be additional add-on fiber/matrix placement systems to add chopped short-length fibers or particulate materials to increase thickness at low cost. Polyester, vinylester, and epoxy matrix materials are used.

4.8.3 *Compression molding*—Compression molding is the most common method of molding thermosetting materials such as sheet molding compound (SMC) and bulk molding compound (BMC). This molding technique involves compressing materials containing a temperature-activated catalyst in a heated matched metal die using a vertical press.

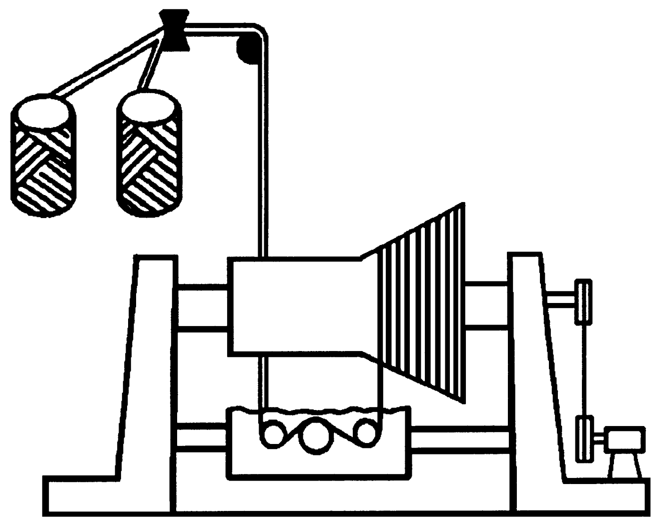


Fig. 4.3—Filament winding process (Mettes 1969).

The molding process begins with the delivery of a high-viscosity uncured composite material to the mold. Mold temperatures typically are in the range of 177 to 204 °C (350 to 400 °F). As the mold closes, composite viscosity is reduced under heat and pressure at approximately 7 MPa (1000 psi). The resin and reinforcements flow to fill the mold cavity.

While the mold remains closed, the thermoset material undergoes a chemical change or cure that permanently hardens it into the shape of the mold cavity. Mold closure times vary from 30 seconds up to several minutes depending on the design and material formulation.

When the mold opens, parts are ready for finishing operations such as deflashing, painting, bonding, and installation of inserts for fasteners. By varying the formulation of the thermoset material and the reinforcements, parts can be molded to meet applications ranging from automotive Class A exterior body panels to structural members such as automobile bumper beams.

4.8.4 *Resin transfer molding*—In resin transfer molding (RTM) (Fig. 4.4), a closed mold process, reinforcement material is placed between two matching mold surfaces that are then closed and clamped. A low-viscosity thermoset

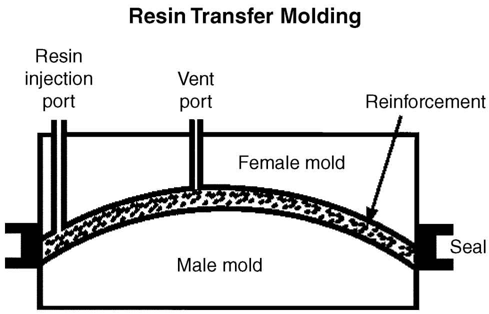


Fig. 4.4—Resin transfer molding.

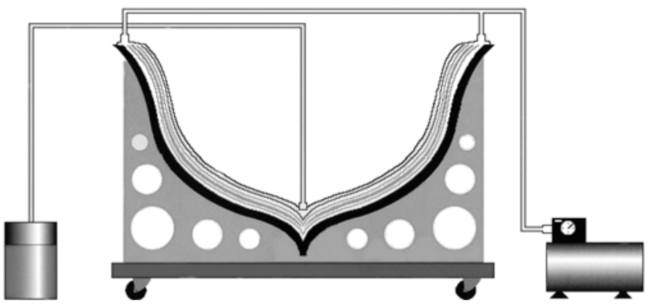


Fig. 4.5—Diagram of VARTM process.

resin is injected under moderate pressure 350 to 690 kPa (50 to 100 psi) into the mold cavity through a port or series of ports within the mold. The resin is injected to fill all voids within the mold and to penetrate all surfaces of the reinforcing materials. The reinforcements may include a variety of fiber types in various forms, such as continuous fibers or mats. The part is typically cured with heat.

RTM is compatible with a variety of resin systems including polyester, vinylester, epoxy, phenolic, modified acrylic, and hybrid resins such as polyester and urethane.

4.8.5 *Vacuum-assisted resin transfer molding*—Vacuum-assisted resin transfer molding (VARTM) parts are made by placing dry fiber reinforcing fabrics into a mold, applying a vacuum bag to the open surface, and pulling a vacuum, while at the same time infusing a resin to saturate the fibers until the part is fully cured (Fig. 4.5).

This process has been used in many new and large applications ranging from turbine blades to bridge decks. Unique to this process is the manufacturing method that allows the efficient processing of VARTM to produce large structural shapes that are generally void-free. This process has been used to make both thin and very thick laminates. In addition, complex shapes with unique fiber architectures allow the fabrication of large parts that have a high structural performance.

A ribbed distribution layer is placed under the vacuum bag, and the resin flow is initially along the surface of the part before being pulled by the vacuum through the dry fabrics.

4.8.6 *Hand lay-up*—Hand lay-up is the oldest and simplest method for producing FRP laminates. Capital investment for hand lay-up processes is relatively low. The most expensive piece of equipment is typically a spray gun for resin and gel coat application. Some fabricators pour or brush the resin into the molds so that a spray gun is not required for this step. There is virtually no limit to the size of the part that can be made. The molds can be made of wood, sheet metal, plaster, or FRP composites.

In the hand lay-up (or wet lay-up) process, highly soluble resin is sprayed, poured, or brushed into a mold. The reinforcement is then wet-out with resin. The reinforcement is placed in the mold. Depending upon the thickness or density of the reinforcement, it may receive additional resin to improve wet-out and allow better drapeability into the mold surface. The reinforcement is then rolled, brushed, or applied using a squeegee to remove entrapped air and to compact it against the mold surface. Curing may be at room temperature or elevated temperatures using roll-in ovens or lamps.

Hand lay-up is also widely applied in the field for strengthening structures with FRP sheets.

4.8.7 *Centrifugal casting*—Centrifugal casting is suited to the production of hollow parts such as pipes. Fiber reinforcement in the form of fabric is placed inside a mandrel, and the resin is sprayed onto it from inside the mandrel rotating at high speed. The centrifugal force pushes the resin outwards, impregnating the reinforcement while minimizing the void ratio.

CHAPTER 5—PROPERTIES, TEST METHODS, AND NONDESTRUCTIVE EVALUATION

5.1—Introduction

This chapter lists some of the reported test methods that have been used to evaluate various properties of FRP materials used as internal or external reinforcement for reinforced and prestressed concrete. Also included are discussions of typical FRP products and properties used in applications involving concrete, as well as a brief description of nondestructive evaluation techniques for FRP systems. ACI 440.3R should be consulted for information on generally accepted test methods for various properties of FRP reinforcement or strengthening materials. A number of relevant ASTM standards also exist for the determination of physical properties of FRP materials such as specific gravity, coefficient of thermal expansion, and fiber volume fraction. These are not discussed in detail in this document.

5.2—Typical properties of currently available products

Advances in FRP technology, increased use of FRP systems in the construction industry, and steadily decreasing costs of FRP materials have led to the development of a wide range of available FRP products for a variety of specific applications. It is beyond the scope of this document to list all currently available FRP products and systems.

The properties of currently available FRP systems vary significantly depending on their specific formulation, constituents, and manufacturing method. The reader is reminded that the material properties of FRPs are highly directionally dependent, and the properties typically quoted are in the direction of the principal reinforcement. Off-axis properties are typically significantly different than those in the fiber direction. In the fiber direction, the products typically display linear elastic behavior to failure (Fig. 5.1), but hybrid combinations, such as carbon and glass or carbon and aramid,

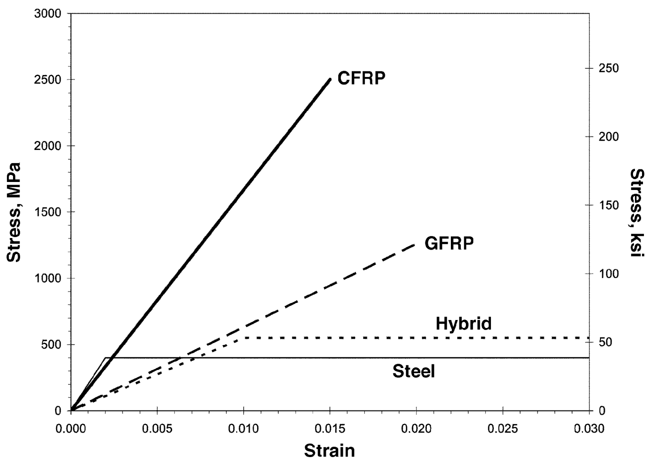


Fig. 5.1—Typical stress-strain curves for FRP products. Based on data from Teng et al. (2002), Tanužs et al. (1996), and Apinis et al. (1998).

display monotonic stress-strain behavior (Fig. 5.1) similar to that of steel (Tamužs et. al. 1996; Apinis et. al. 1998).

Tables 5.1 and 5.2 present overviews of the properties of some typical FRP products for internal reinforcement and external strengthening, respectively, of reinforced concrete structures. Included in the tables is information on FRP bars, rods, sheets, plates, and tapes. The tabulated information is that reported by the manufacturer. In Table 5.2, the material properties are reported using one of two methods specified by ACI 440.3R. Method 1 is based on composite area and uses the measured width and thickness of the specimen to determine the cross-sectional area of the specimen for calculation of the tensile strength and modulus of elasticity. Method 2 is based on equivalent fiber area and uses the equivalent thickness of a fiber layer without resin and the measured width for calculation of the tensile strength and modulus of elasticity. Specific information on the potential applications of each of these systems can be obtained by contacting the FRP manufacturers (also included in the tables). These tables should not be considered as endorsements for a particular FRP product.

TechFab LLC (2006) produces a very light carbon/epoxy grid with strengths ranging from 14 kN/m (1000 lb/ft) to over 220 kN/m (15,000 lb/ft) with grid openings from approximately 12 mm (0.5 in.) square to over 50 mm (2 in.) square. Because of the flexible nature of the textile process used to manufacture grids, grid opening sizes and strengths can be customized to suit the many various concrete product uses.

5.3—Test methods for mechanical properties

The basic mechanical properties of FRP materials can be determined by applying tensile, compressive, and shear loadings. Flexural testing is also used in some cases. Static material properties are usually of primary interest.

5.3.1 *Tension test methods*—Axial tension testing of high-strength unidirectional composites is often a challenge because load should be transmitted from the testing apparatus to the specimen via shear, and the shear strength of a unidirectional composite is typically much lower than its axial tensile strength. Further, shear gripping will load the external fibers more than the internal ones causing shear lag and progressive fiber failure. To avoid these problems, end tabs are required when testing flat laminates, as described in detail in ACI 440.3R. Special anchors are required for testing of FRP rods and bars by inserting their ends into steel cylinders that are subsequently filled with either a polymer resin or a cement-based grout (also described in ACI 440.4R). For grid-type FRP reinforcements, linear test specimens can be prepared by cutting away extraneous material in such a way as not to affect the performance of the part to be tested. ASTM D 618 (Porter and Barnes 1991; ACI 440.3R) recommends leaving a minimum 2 mm (0.08 in.) projection of crossbars.

5.3.2 *Compression test methods*—It has been said that there is no true axial compression test for composites. The mode of failure is buckling, ranging from buckling of the entire specimen cross section or local microbuckling of individual fibers. Thus, the greater resistance to buckling the test fixture provides, the higher the compressive strength values obtained. For flat laminate FRP composites, many axial compression test methods in current use are some variation of the Celanese compression test as in ASTM D 3410. This test uses a thin, straight-sided specimen that looks very much like an axial tension specimen except that the distance between tabs is much smaller. For FRP rods, tests can be carried out according to ASTM D 695. Common problems associated with compression testing of FRPs are discussed in detail by Harper et al. (1995).

5.3.3 *Shear test methods*—Several ASTM standards are available for determining the shear properties of composite

Table 5.1—Selected properties of typical currently available FRP reinforcing products as reported by the manufacturer

Reinforcement type	Designation	Diameter, mm (in.)	Area, mm^2 ($in.^2$)	Tensile strength, MPa (ksi)	Elastic modulus, GPa (ksi)
Deformed steel	No. 3	9.5 (0.375)	71 (0.11)	420* (60)	208 (30,000)
Pultrall Inc. (2005)					
V-ROD CFRP rod	3/8	9.5 (0.375)	71 (0.11)	1596 (230)	120 (17,400)
V-ROD GFRP rod	3/8	9.5 (0.375)	71 (0.11)	852 (123)	43 (6700)
Autocon Composites Inc. (2006)					
NEFMAC GFRP rod	G10	N/A	79 (0.12)	600 (87)	30 (4400)
NEFMAC CFRP rod	C16	N/A	100 (0.16)	1200 (174)	100 (14,500)
Hughes Brothers (2005)					
Aslan 100 GFRP rod	No. 3	9.5 (0.375)	84 (0.13)	760 (110)	41 (5900)
Aslan 200 CFRP rod	No. 3	9.0 (0.360)	65 (0.10)	2068 (300)	124 (18,000)
Mitsubishi (2005)					
LEADLINE™ CFRP rod	Round	12 (0.470)	113 (0.18)	2255 (327)	147 (21,300)
Sika Corp. (2007)					
CarboDur CFRP rod	3/8	9.5 (0.375)	71 (0.11)	2800 (406)	155 (22,500)

*Specified yield strength.

Table 5.2—Selected properties of typical currently available FRP strengthening systems as reported by the manufacturer

FRP system	Fiber type	Weight, g/m^2 (lb/ft^2)	Design thickness, mm (in.)	Tensile strength, MPa (ksi)	Tensile elastic modulus, GPa (ksi)	ACI 440.3R test reporting method
Fyfe Co. LLC (2005)						
Tyfo SEH51 sheet	Glass	915 (0.19)	1.3 (0.052)	575 (83.4)	26.1 (3785)	Method 1
Tyfo SCH41 sheet	Carbon	644 (0.14)	1.0 (0.040)	985 (143)	95.8 (13,900)	Method 1
Sika Corp. (2007)						
SikaWrap Hex 100G sheet	Glass	913 (0.19)	1.0 (0.040)	531 (77)	23.6 (3430)	Method 1
SikaWrap Hex 103C sheet	Carbon	618 (0.13)	1.0 (0.040)	717 (104)	65.1 (9450)	Method 1
CarboDur S plate	Carbon	1800 (0.37)	1.2 to 1.4 (0.048 to 0.055)	2800 (406)	165 (23,900)	Method 1
CarboDur M plate	Carbon	1900 (0.39)	1.2 (0.048)	2400 (348)	210 (30,500)	Method 1
CarboDur H plate	Carbon	1900 (0.39)	1.2 (0.048)	1300 (189)	300 (43,500)	Method 1
BASF (2006)						
MBrace EG 900 sheet	Glass	900 (0.19)	0.37 (0.015)	1517 (220)	72.4 (10,500)	Method 2
MBrace AK 60 sheet	Aramid	600 (0.12)	0.28 (0.011)	2000 (290)	120 (17,400)	Method 2
MBrace CF 130	Carbon	300 (0.062)	0.17 (0.007)	3800 (550)	227 (33,000)	Method 2
MBrace CF 160	Carbon	600 (0.124)	0.33 (0.013)	3800 (550)	227 (33,000)	Method 2
S&P 100/1.4	Carbon	—	1.4 (0.055)	2700 (390)	159 (23,000)	Method 1
Hughes Brothers (2005)						
Aslan 400 plate	Carbon	—	1.4 (0.055)	2400 (350)	131 (19,000)	Method 1
Aslan 500 tape	Carbon	—	2.0 (0.079)	2068 (300)	124 (18,000)	Method 1
Aslan 500 tape	Carbon	—	4.5 (0.177)	1965 (285)	124 (18,000)	Method 1

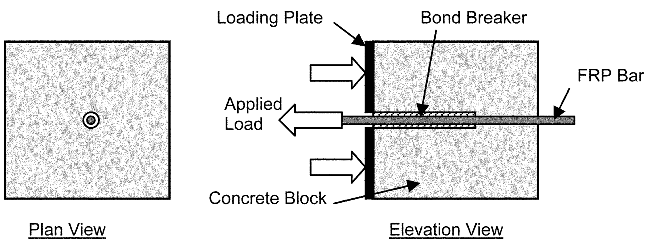


Fig. 5.2—Pullout test specimen.

materials. For flat FRP laminates, direct in-plane shear test methods include the V-notched beam test (ASTM D 5379/D 5379M), the two-rail and three-rail shear tests (ASTM D 4255/D 4255M), and the 45-degree laminate tensile shear test (ASTM D 3518/D 3518M). For FRP rods, shear test devices have been constructed so that a rod-shaped test specimen is sheared on two planes as in ACI 440.3R.

5.3.4 *Flexural test methods*—Flexural tests are relatively easy to perform; thus, they are fairly popular. The difficulty is that flexural testing does not directly provide information on basic material properties. The stress state on the loaded side of the specimen is compression, on the other side is tension, and at the neutral axis is pure shear. Usually, the shear component is minimized by making the test specimen long relative to its thickness (a ratio of approximately 32:1 is commonly used) (Adams 1992). Although flexural tests do not directly provide basic design data, their use is often justified on the basis that the material is subjected to flexural loading in service.

For testing FRP rods in flexure, ASTM D 4476 describes a standard test method for flexural properties of fiber-reinforced pultruded rods. This method has been used for FRP reinforcing bars with some modifications, as in Wang et al. (2002). FRP laminates can also be tested in flexure in three- or four-point bending as described in Adams (1992).

5.3.5 *Bond test methods for internal FRP reinforcement*—Bond characteristics influence the mechanism of load transfer between FRP reinforcement and concrete, and therefore control the concrete crack spacing, crack width, required concrete cover to the reinforcement, and the reinforcement development length. The behavior of reinforced concrete structures thus depends on the integrity of the bond. Many test methods have been developed and carried out to determine the bond strength of FRP bars and rods in concrete.

5.3.5.1 *Pullout tests*—The concentric pullout test is a popular test method adopted by researchers for comparative bond assessment for FRP reinforcement. In this method, an FRP bar specimen is embedded in a concrete block, with the embedded length of the FRP rod typically being five times the rod diameter, and the bar is pulled in tension as shown in Fig. 5.2. The average bond stress and bond stress versus slip at the loaded end can thus be obtained. The pullout test is used for comparative tests, but is not considered a valid test to determine the development that is needed for the design embedment length for flexural beam design. Additional information and specimen fabrication details are given in ACI 440.3R.

The ring test (Tepfers and Olsson 1992) is carried out as a pullout test, where the specimen is a concrete cylinder with the studied reinforcing bar embedded along the center axis of the cylinder. This test allows for determination of the splitting tendency of the bar in all stages of loading up to bond failure by determination of the angle between bond forces and the bar axis. The bond length corresponds to the height of the cylinder, which is chosen to be three diameters of the bar, so the bond stress along the bond length becomes practically evenly distributed. The cylinder is cast in a thin steel tube (the ring), which becomes a part of the test specimen.

5.3.5.2 *Flexural bond tests*—Pullout tests, while extremely useful in many circumstances, are not representative of stress field conditions in an FRP reinforced concrete beam. Flexural beam tests, such as the beam-end test, solve some of the stress field discrepancies that are present in pullout tests, and thus offer the advantages of representing the beam stress field more closely.

The beam-end test method is described in detail for steel reinforcing bars in ASTM A 944, and is specified as a test for the relative bond strength of reinforcing bars in concrete. In this method, a bar is embedded in a concrete block, and a tensile load is introduced into the bar using a special test setup as shown schematically in Fig. 5.3. This test setup has the advantages of simulating the stress field condition in a reinforced concrete member in flexure.

Various additional flexural test methods exist and have been used to study the bond behavior of FRP bars embedded in concrete. Several of these are shown schematically in Fig. 5.4, including (a) the simple beam test (Larralde et al. 1994); (b) the notched beam test (Daniali 1992); (c) the hinged beam test (Magnusson 1997); (d) the spliced beam test (Tighiouart et al. 1999); (e) the cantilever beam test (Fish 1992); and (f) the trussed beam test. If these tests are done upside down, then the effects of self-weight are practically eliminated.

5.3.5.3 *Direct axial tension test*—Two types of direct axial tension tests have been used with a concrete specimen having a continuous bar embedded in its center (Hasau et al. 1990). In the first type, load is directly applied to the bar protruding from the concrete ends. In the second type, load is applied to the concrete. A concrete specimen is cast with an FRP bar embedded at its center and a notch at midlength to allow for the formation of the first crack at a preset location (Fig. 5.5). Bond strength is studied indirectly through the observation of crack spacing and width in the concrete block.

5.3.6 *Bond test methods for externally bonded FRP reinforcement*—Flexural and shear strengthening techniques of bonding FRP plates or sheets to reinforced concrete elements is now a method of choice for structural rehabilitation. Failure of these bonded systems before achieving their FRP capacity, however, can result from a number of mechanisms collectively referred to as delamination or debonding. Teng et al. (2002) provide a thorough review of bond-related failure mechanisms and the present state of the art for modeling these. The behavior of the FRP-adhesive-concrete interface region, which should include the region of cover concrete adjacent to the interface, is collectively interpreted as bond behavior. Transfer of stresses through the cover concrete, for instance, can lead to failures through the cover concrete that are nonetheless affected by the properties of the bonding adhesive. Complicating interface region behavior is the mixed mode behavior of the debonding process and the heterogeneous material properties in this region.

Bond behavior is a critical parameter in strengthening a concrete structure with externally bonded FRP, and numerous test methods have been presented. Test methods

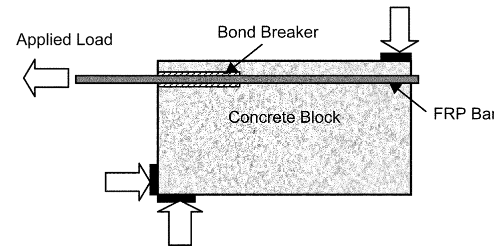


Fig. 5.3—Beam-end test method.

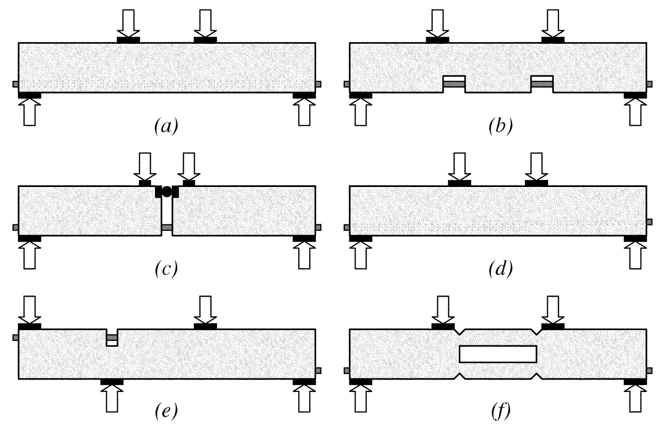


Fig. 5.4—Additional flexural bond test method configurations: (a) simple beam test; (b) notched beam test; (c) hinged beam test; (d) spliced beam test; (e) cantilever beam test; and (f) trussed beam test.

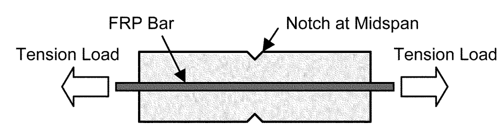


Fig. 5.5—Direct axial tension test.

can be broadly classified into three types: shear bond, tension, or mixed mode. Recent research has focused largely on mixed mode bond test methods in an effort to accurately characterize the FRP-concrete bond, which in practice experiences a combination of shear and normal stresses. A summary of bond test methods for externally-bonded FRP systems is provided by Ueda and Dai (2005).

5.3.6.1 *Shear bond type tests*—Because externally bonded FRP materials are most often used in configurations where stress transfer across the interface is dominated by shear (Mode II) stresses, various shear bond test methods are available in the literature. The various methods can be used to study the local interfacial shear bond behaviors by studying the bond strain distribution (Ueda and Dai 2005). Schematics showing the configurations of various types of shear bond tests are given in Fig. 5.6, including: (a) the single-lap shear bond test (Bizindavyi and Neale 1999; Chajes et al. 1996); (b) the push-apart; (c) the pull-apart double-lap shear bond test (Horiguchi and Saeki 1997; Brosens and Gemert 1997); (d) the bending-type shear bond test (Horiguchi and Saeki 1997); and (e) the inserted-type shear bond test (Fukuzawa et al. 1997).

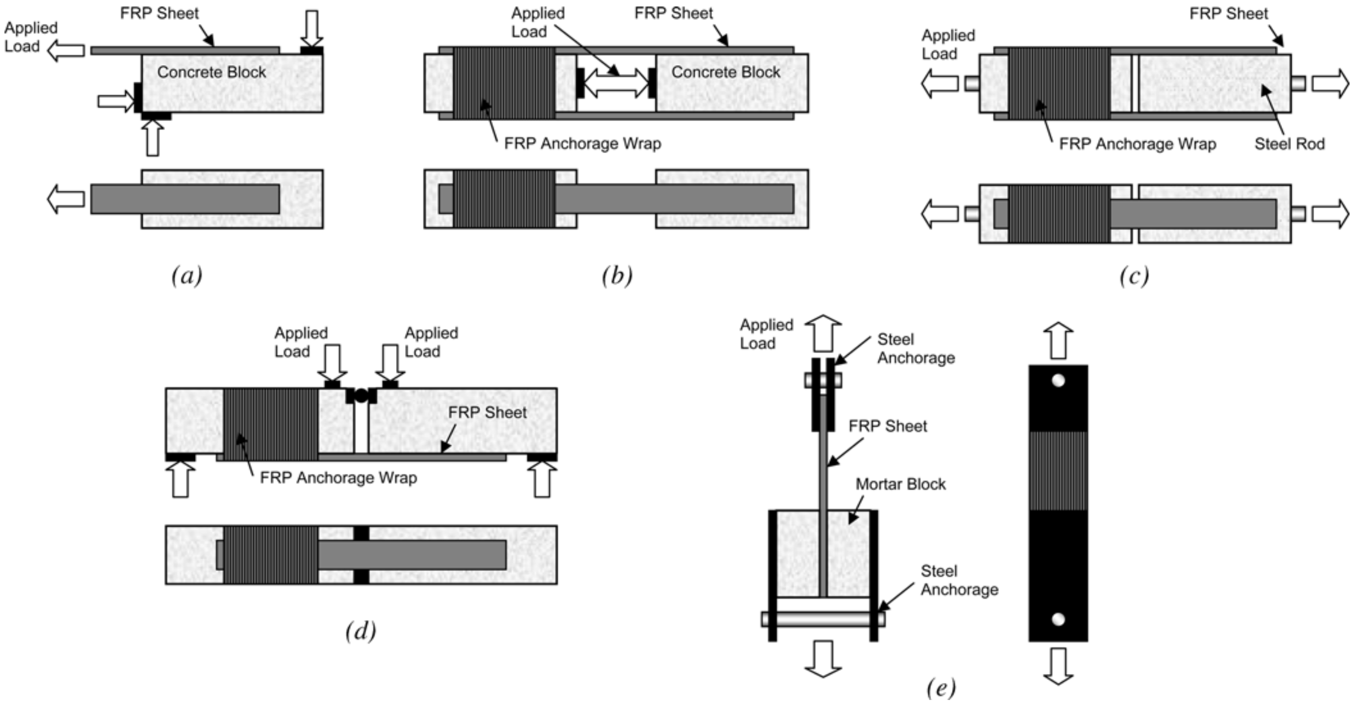


Fig. 5.6—Various shear-bond test method configurations: (a) single-lap shear bond test; (b) push-apart; (c) pull-apart double-lap shear bond test; (d) bending-type shear bond test; and (e) inserted-type shear bond test.

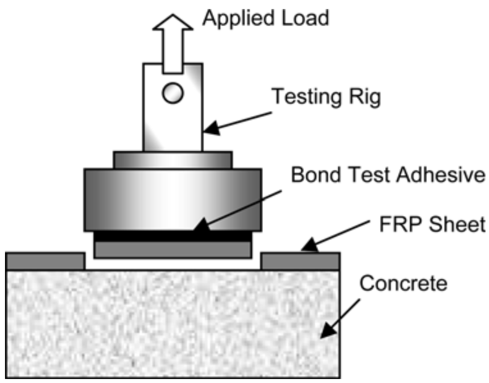


Fig. 5.7—Direct tension pulloff test method (Xu et al. 2002).

5.3.6.2 *Tension-type bond tests*—At least three test methods are available in the literature to evaluate the strength of the FRP-concrete bond subject to pure normal (Mode I) stresses. Figure 5.7 shows a schematic of the direct tension pulloff test method (Horiguchi and Saeki 1997), which is also presented in detail in ACI 440.3R. In this method, a special testing rig is used to pull a circular disc of bonded FRP away from the surface of the concrete in direct tension. The circular area is typically created using a core drill, and an adhesion disc is used, with an appropriate adhesive, to apply an increasing tensile load to the disc until bond failure occurs. For well-bonded FRP materials, the failure surface is in (and thus limited by) the substrate concrete. An alternate shear test arrangement uses a similar test setup, but twists the adhesion disc off of the substrate; this torsional pulloff test is not included in ACI 440.3R. Other tension-type bond test methods not discussed in detail herein, or in ACI 440.3R, include the three-point bending pulloff test (Xu et al. 2002) and the wedge splitting test (Dai et al. 2003).

5.3.6.3 *Mixed-mode bond tests*—Mixed mode bond test methods are more representative of the interface bond behavior for concrete structures retrofitted with FRP sheets (Ueda and Dai 2005). Mixed-mode testing methods include the variable angle peel test (Karbhari and Engineer 1996), the beam-type and slab-type dowel tests (Ueda and Dai 2005), the single contoured cantilever beam test (Boyajian et al. 2002), the double cantilever beam test (Kim et al. 2002; Wan et al. 2002), and the modified double cantilever beam test (Wan et al. 2004). These bond test methods are considerably more complex and difficult to perform than those discussed previously, and they are typically used to study the fracture behavior of FRP-concrete interfaces. The reader is encouraged to consult the listed references for more information on mixed-mode bond testing.

Test methods are also available to study tension stiffening and secondary crack formation in applications of externally bonded FRP materials (Nanni et al. 1997).

5.3.6.4 *Bond overlap tests*—Test methods are available to evaluate the strength of FRP to FRP bond overlaps or splices (ACI 440.3R), and can be conducted in single-lap or double-lap configurations.

5.3.7 *Concrete surface preparation for bond testing*—The effects of surface preparation of the concrete on the bond between concrete and FRP are very important, and have been studied by several authors. Chajes et al. (1996) found that mechanical abrasion increased the interfacial bond strength. Shen et al. (2002) also studied the effect of surface roughness on the bond performance between FRP laminates and concrete. Surface preparation was by rotary water jetting to produce different roughness grades, which were classified using a laser profilometer. Specimens were subjected to flexural bond testing and direct tension and torsional pulloff tests. They concluded that moderate surface roughness was necessary for adequate bond properties to be developed and that different CFRP systems tested had different optimal surface roughnesses.

5.4—Durability testing methods

The reduction of strength and the subsequent failure of materials subjected to cyclic or sustained loading combined with environmental effects have emerged as some of the most fundamental problems of engineering materials. Because composite materials are regarded as having good long-term performance, they are used in applications where the degradation of strength and life expectancy by fatigue, creep, and environmental effects would otherwise be a concern. Thus, testing methods to evaluate the susceptibility of FRP materials to these conditions is required.

5.4.1 *Tensile fatigue testing methods*—Fatigue tests on FRPs can be carried out using several specimen configurations, including direct tests on rods using various kinds of gripping mechanisms (ACI 440.3R), and tests on concrete beam specimens reinforced with FRP reinforcements. In the case of performing direct tests on rods, problems are often encountered in gripping the rod, because some grips perform poorly when used for cyclic tests, and failure may occur in the anchoring zone. When performing fatigue tests on concrete beam specimens, the rod is continuously in contact with concrete, which may affect the fatigue behavior of the FRP rod through fretting or friction heating (Adimi et al. 2000).

5.4.1.1 *Concrete prism tensile fatigue test method*—Because of the relatively low modulus of elasticity of FRP reinforcement combined with its high strength, it is very difficult to develop high levels of stress in FRP rods used as tensile reinforcement in concrete beams. A reinforced concrete prism specimen for tensile fatigue testing has been developed (Adimi et al. 2000). Figure 5.8 shows the details of the specimen, where top and bottom blocks of concrete serve as anchors for the FRP rod while the middle block provides the concrete environment for the FRP rod at the part where the failure is expected. The middle block is separated from the end blocks by a bond breaker during casting. This specimen is loaded in tension by collars at the shoulders of the end blocks, and can be tested under any stress amplitude and with the maximum stress almost equal to the tensile strength of the FRP rod. To avoid stress concentrations near the rod and splitting of concrete end blocks during the test, a bond breaker is included, as shown, of approximately five bar diameters, and the end blocks are precompressed near the constriction with a steel compression ring (not shown in the figure).

5.4.1.2 *Direct fatigue tests*—Fatigue tests can also be conducted directly on FRP specimens. These specimens can be rods, flat laminates, or sections of FRP grids. A suggested test method for tensile fatigue of FRP bars is discussed in detail in ACI 440.3R.

5.4.2 *Creep test methods*—FRP materials may fail by creep at strengths below the maximum static strength when subjected to a significant sustained stress for long periods. This creep-rupture strength varies according to the type of FRP, and so the creep-rupture strength should be evaluated when determining the allowable level of sustained tension in FRP used as reinforcement. A suggested test method for creep testing of FRP bars is presented in ACI 440.3R. Traditional methods of performing creep tests, including direct gravity loading, compressed spring loading, or gravity loading using a lever arm (so called creep frame) to permit greater loads, have all been adapted for testing FRP materials.

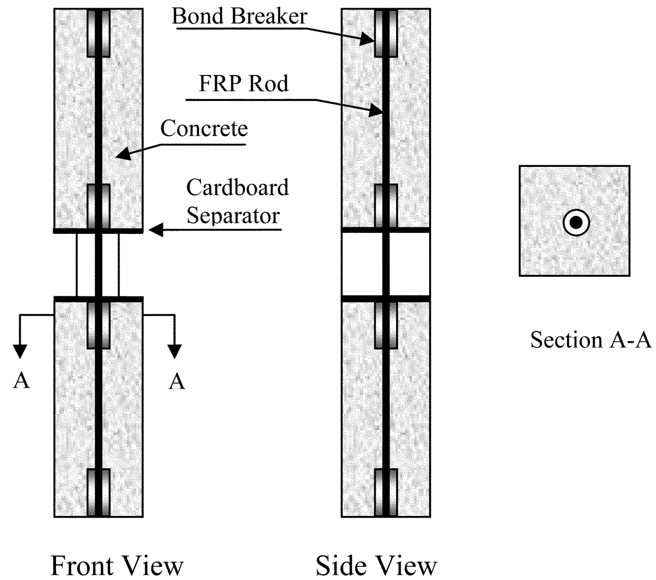


Fig. 5.8—Schematic view of test specimen (adapted from Adimi et al. 2000).

5.5—Nondestructive inspection techniques for FRP materials

Nondestructive inspection is needed to assure the integrity of FRP materials and FRP-strengthened structures over their service life (U.S. Department of Defense 2002). Structures made of traditional materials are all susceptible to damage and environmental degradation in service. As a result, they are periodically inspected. Although decades of experience have shown composites to be reliable structural materials, they are also susceptible to damage and possible environmental degradation. Consequently, structures employing composite reinforcement should also be subjected to periodic nondestructive inspection in service. Test methods should be designed to evaluate a material system economically and quickly. This section discusses quality assurance and nondestructive testing of composites.

5.5.1 *Nondestructive inspection*—Nondestructive inspection (NDI) tests are used to find flaws and damage such as resin starvation, resin richness, wrinkles, ply bridging, discoloration due to overheating or lightning strike, impact damage by any cause, foreign matter, blisters, and disbanding. Some of the discrepancies are readily discernible by a visual inspection (Crane 2000; Pethrick 2000; U.S. Department of Defense 2002).

Composite NDI techniques commonly used in production of composites include visual, ultrasonic, thermography, laser shearography, tap, and x-ray (Crane 2000; U.S. Department of Defense 2002). Advantages of thermography and shearography are that they are large area, noncontact methods that create permanent records. This is useful for monitoring possible damage growth over the life of a structure.

Despite the recognized importance of NDI to ensure reliability of composite structures, there has not been a great deal of research on NDI methods for FRP-strengthened concrete structures (Newman and Zweben 2002a). This is a significant issue considering the long service life of civil engineering structures.

5.5.1.1 *Visual inspection*—NDI by visual means is by far the oldest and most economical NDI method (U.S. Department of Defense 2002). Consequently, visual inspection is performed routinely as a means of quality control and damage assessment.

Once detected, the affected area becomes a candidate for closer inspection. Flashlights, magnifying glasses, mirrors, and borescopes are employed as aids in the visual inspection of composites. Visual inspection cannot find internal flaws in the composite, such as delaminations, disbands, or matrix crazing. In addition, impact damage is frequently not visible to the naked eye, and tight surface cracks and edge delaminations may not be detected visually. Therefore, visual inspection techniques need to be supplemented by other methods of nondestructive testing.

5.5.1.2 *Tap testing*—Sometimes referred to as audio, sonic, or coin tap, this technique makes use of frequencies in the audible range (10 to 20 Hz) to detect damage (U.S. Department of Defense 2002). Although it can be an accurate method in the hands of skilled and experienced personnel, tap testing is highly subjective, time consuming, and does not create a permanent test record. Despite these limitations, tap testing is a common technique used for the detection of delamination or disbonding. The method is performed by tapping the inspection area with a solid round disk or lightweight hammer and listening to the audible response of the structure. A clear, sharp, ringing sound is indicative of a well-bonded solid structure, while a dull sound indicates a damaged area.

5.5.1.3 *Ultrasonics*—Ultrasonic inspection has proven to be a useful tool for the detection of internal delaminations, voids, or inconsistencies in composite components not otherwise discernible using visual or tap testing (Tittmann and Crane 2000; U.S. Department of Defense 2002).

A high-frequency sound wave is introduced into the structure, and may be directed to travel normally to its surface, along the surface, or at some predefined angle to the surface. Different directions are often used because flaws may not be noticeable from only one direction.

When an ultrasonic wave strikes an interrupting object, part of the wave or energy is transmitted through the object, and the rest of the energy is reflected back to the surface. The disrupted or diminished sonic energy is then picked up by a receiving transducer and converted into a display on an oscilloscope or a chart recorder. The display allows the operator to comparatively evaluate the discrepant indications against those areas known to be good.

5.5.1.4 *Radiography*—Radiography, or x-ray as it is often referred to, allows a view into the interior of the part (Crane 2000; U.S. Department of Defense 2002) by passing x-rays through the part while recording the absorption of the rays onto an x-ray sensitive film. Because the method records changes in total density through its thickness, it is not a preferred method for detecting defects, such as delaminations, that are in a plane normal to the ray direction.

Although radiography may be useful in a factory environment, it is expensive and not practical for field application. Furthermore, most composites are nearly transparent to x-rays, so low-energy rays should be used. Opaque penetrant can be used to enhance the visibility of surface breaking defects, although this is generally not available for in-service inspections.

Operators should be protected by sufficient lead shields because the possibility of exposure exists either from the x-ray tube or from scattered radiation. Maintaining a distance from the x-ray source is essential.

5.5.1.5 *Shearography*—Shearography is an optical NDI technique that detects defects by measuring variations in reflected light (speckle pattern) from the surface of the object when stressed (Krishnaswamy 2000; U.S. Department of Defense 2002; Newman and Zweben 2002a,b). Using a laser light source, an original image of the illuminated surface is recorded via a video image. The part is subsequently stressed by heating, mechanical loading, changes in pressure, or acoustic vibrations during which a second video image is made. Changes in the surface contour caused by debonding or delaminations become visible on the video display. Shearography is being used in production environments for rapid inspection of bonded composite structures.

5.5.1.6 *Thermography*—Thermal inspection includes all methods in which heat-sensing devices are used to measure temperature variations (Thomas 2000; U.S. Department of Defense 2002; Newman and Zweben 2002a,b). The basic principle of thermal inspection consists of measuring or mapping surface temperatures when heat flows from, to, or through a test object.

All thermographic techniques rely on differentials in thermal conductivity between normal, defect-free areas and those having a defect. Normally, a heat source is used to elevate the temperature of the specimen being examined while the surface heating effects are observed. Because defect-free areas conduct heat more efficiently than areas with defects, the amount of heat that is absorbed or reflected indicates the quality of the bond. The type of defects that affect the thermal properties include debonding, cracks, impact damage, panel thinning, and water ingress. Thermal methods are most effective for thin laminates or for defects near the surface.

CHAPTER 6—PERFORMANCE OF CONCRETE MEMBERS WITH INTERNAL FRP REINFORCEMENT

This chapter summarizes research findings regarding the performance of FRP as the main structural reinforcement for non-prestressed concrete members. FRP internal reinforcement is widely used in commercial applications. Much of the development of design guidelines for internal FRP reinforcement dates back to the pioneering work of Nanni (1993). Additional work by others facilitated and contributed to the design guidelines in their current format (Jaeger et al. 1997; GangaRao and Vijay 1997; Theriault and Benmokrane 1998). The widespread implementation of FRP as 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 should 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, serviceability criteria are often governing parameters in design considerations.

6.1—Strength

Flexural failure of concrete members reinforced with currently available FRP materials is governed by either concrete crushing or FRP tensile rupture. Such failure modes are brittle and differ from the behavior of concrete beams under-reinforced with steel. In addition, the shear capacity of concrete beams is reduced as a result of increased crack width and reduced size of compressive stress blocks.

6.1.1 *Flexural strength*—Nawy and Neuwerth (1971) monotonically tested 20 simply supported rectangular beams reinforced with GFRP and steel reinforcing bars. The beams failed by compression of the concrete; hence, they did not develop the full capacity of the FRP bars. The authors suggested that because the modulus of FRP bars was only slightly higher than that of concrete, limited tensile stresses could be transmitted from the concrete to the FRP reinforcement. In a second study, Nawy and Neuwerth (1977) found that the behavior 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. Satoh et al. (1991) reached similar conclusions.

Faza and GangaRao (1992) investigated the flexural performance of simply supported rectangular concrete beams reinforced with GFRP reinforcing bars. They concluded that, to take advantage of the high FRP reinforcing bar ultimate strength, use of high-strength concrete instead of normal-strength concrete (69 versus 27.6 MPa [10 versus 4 ksi]) is essential. The ultimate moment capacity of high-strength concrete beams increased by 90% when an equal area of FRP reinforcing bars of ultimate tensile strength of 896 MPa (130 ksi) was used instead of mild steel reinforcing bars. Sand-coated FRP reinforcing bars, in addition to high-strength concrete, were 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.

Bank and Xi (1992) tested nine simply supported slabs having shear-span-to-effective-depth ratios of approximately three and reinforced with a variety of molded and pultruded GFRP gratings. Following initial cracking, flexural cracks developed in the constant moment region at regular spacing of approximately 76 mm (3 in.). 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 as predicted by ACI 318.

Nanni et al. (1992) tested five concrete beams reinforced with hybrid reinforcing bars, steel deformed bars, and FRP reinforcing bars. Load-deflection behavior for the different reinforcing bar types was 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 consisting of uncracked section behavior followed by cracked-section linear-elastic behavior to failure; and 3) for hybrid reinforcing bars, a three-stage behavior typical of under-reinforced steel beam characterized by an uncracked section and linear-elastic response followed by steel core yielding before ultimate failure. Test results showed that, relative to ultimate flexural capacity, coating the FRP and hybrid reinforcing bars with sand increased flexural capacity by approximately 25%. Smaller crack widths and higher postcrack flexural rigidity were also reported for the sand-coated reinforcing bars as compared with the corresponding uncoated reinforcing bars.

Matthys and Taerwe (2000) carried out four-point bending tests on eight reinforced concrete slabs. The materials used to reinforce the slabs were steel, carbon, and hybrid carbon-glass NEFMAC grids and CFRP bars. The authors found that the design of concrete structures with FRP reinforcement was mainly governed by serviceability criteria. In general, crack control was less restrictive than deflection control. A deformability index was defined as the ratio of the deflection at ultimate load to deflection at service load. For the FRP-reinforced slabs of this study, the deformability index was between 10 and 110% higher than the deformability index obtained for steel-reinforced slabs. The margin between the ultimate load and the service load of the FRP-reinforced slabs was on the order of 2.5 to 4.5 times higher than for steel-reinforced structures.

Nanni and Faza (2002) outlined design and construction using FRP bars with an objective to provide a meaningful overview of the design protocol for FRP-reinforced concrete members. Thiagarajan (2003) presented an analytical and experimental study on the flexural behavior of concrete beams reinforced with sand-coated CFRP rods and found that the analytical capacities, predicted using equilibrium equations; strain compatibility condition; and finding the concrete compressive forces by integrating Hognestad's equations, were within 8% of the experimental capacities.

6.1.2 *Beam shear*—Larralde (1992) tested a series of eight FRP-grating/concrete composite slabs in which the shear span-depth ratio and concrete deck thickness was varied in an attempt to force different types of failures. Test results showed that, for samples with a shear span-depth ratio 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 shear span-depth ratios of 5 or less, failure occurred as a result of diagonal tension cracking.

Shehata (1999) investigated the effect of bend radius, the crack angle, the stirrup anchorage, the stirrup spacing, and the material type of flexural reinforcement in tests on 52 panel specimens. The following limitations were proposed for detailing FRP stirrups to achieve a capacity of at least 50% of the guaranteed strength parallel to the fibers: 1) the bend radius should not be less than 50 mm (2 in.) or four times the effective bar diameter, whichever is greater; and 2) the tail length should not be less than 70 mm (2.75 in.), or six times the effective bar diameter, whichever is greater. Beams reinforced with CFRP strands for flexure showed less concrete contribution to shear strength than did beams reinforced with steel strands. This observation was attributed to the wide cracks, the small depth of the compression zone, and poor dowel action associated with the use of FRP as longitudinal reinforcement. Beams with GFRP stirrups showed better performance than those with CFRP stirrups.

Razaqpur and Mostofinejad (1999) tested four continuous beams that were reinforced with FRP grids as shear reinforcement. The beams did not collapse, and retained nearly 80% of their strength when they reached their load capacity in the negative moment regions. CFRP stirrups experienced strains of 4000 $\mu\varepsilon$ without rupture, which was well above the limit of 2000 $\mu\varepsilon$ set by the CSA S6. The authors observed the same tendency in the behavior of beams reinforced with CFRP stirrups and beams reinforced with steel stirrups. The use of CFRP for longitudinal reinforcement reduced the shear strength of the beams due to a lower contribution of aggregate interlock to shear resistance. The bars in dowel action carried low shear force due to the relatively poor strength and stiffness of FRP bars in the transverse direction. After failure, the concrete was removed from some of the beams to inspect the CFRP grid stirrups. Researchers noticed that, even in the most distressed parts of the beam, no CFRP stirrup had ruptured.

Deitz et al. (1999) tested 12 concrete deck panels reinforced with three reinforcing schemes. One scheme used GFRP reinforcement at the top and bottom of the deck. Another scheme used all epoxy-coated steel reinforcement, and the third scheme used GFRP bars at the top and steel at the bottom. Two modes of failure were observed: the specimens reinforced with steel failed in flexure, while the specimens with the other two reinforcement schemes failed in diagonal tension. The slabs with GFRP failed in diagonal tension because the lower modulus of the GFRP reinforcement compared with steel allowed wider diagonal cracks leading to shear failure before flexural failure. The diagonal tension modes provided adequate warning of failure because large crack widths and displacements developed.

In 2006, ACI 440.1R adopted the beam shear model of Tureyen and Frosch (2003) in which the concrete shear capacity V_c of flexural members using FRP bars as main reinforcement is based on the depth of the elastic cracked section neutral axis. The formulation is independent of reinforcing bar material properties but accounts for the axial stiffness of the FRP reinforcement indirectly through the elastic neutral axis depth. This method has shown good agreement with test data from a variety of bar stiffnesses and concrete strengths (Tureyen and Frosch 2003). Additional testing by El-Sayed et al. (2005, 2006) found that the method proposed by Tureyen and Frosch (2003) gave reasonably conservative results.

6.1.3 *Punching shear*—El-Ghandour et al. (1999) carried out tests on flat slabs reinforced with FRP bars. The authors found problems of bond slip and crack localization over the main flexural bars. As a result, the slabs failed at loads lower than their expected flexural and punching shear capacities. According to the authors' observations, the ACI 318-95 (ACI Committee 318 1995) equation for punching shear capacity ignored the influence of tension flexural reinforcement when calculating the concrete shear resistance, which was heavily dependent on the concrete strength. The authors estimated that the ACI equation for punching shear was not conservative, and proposed an alternative equation.

Matthys and Taerwe (2000) conducted tests on 17 FRP circular reinforced concrete simply supported slabs reinforced with FRP grids. The authors found that most slabs had a punching shear failure mode. The slabs with FRP reinforcement with a similar strength as the reference steel reinforced slabs required a lower load to produce cracking and had lower postcracking stiffness than the reference slabs. Evaluation of their test results showed that the punching shear design provisions in the European, Japanese, and ACI 318 codes need to be modified to accurately predict the punching capacity of slabs with FRP reinforcement. The computed punching failure load was lower than the measured failure load for slabs reinforced with FRP bars with a low modulus. The authors proposed to modify the value in the punching shear capacity code equations that account for the reinforcement ratio effect with the factor E_{frp}/E_s.

Ospina et al. (2003) tested four interior column two-way slab specimens subjected to uniform gravity loading, two of which were reinforced with GFRP bars and one with a GFRP grid. Results show that the axial stiffness of the FRP reinforcement, as well as the concrete strength f_c', significantly affect the concentric punching shear response of FRP-reinforced slabs. Punching shear failure in slabs reinforced with FRP bars was sudden and brittle; however, two-way slabs reinforced with FRP grids exhibited a much more ductile failure (Bank and Xi 1992; Jacobson et al. 2006; Ospina et al. 2003). El-Gamal et al. (2005) tested six full-scale bridge deck slabs reinforced with GFRP, CFRP, or steel bars. They found little to no correlation between reinforcement ratio and punching shear capacity for the GFRP and CFRP reinforced slabs.

Ospina (2005) performed a statistical evaluation of available punching shear test results from slabs with either FRP reinforcement or steel and concluded that the one-way shear design model of Tureyen and Frosch (2003) can be modified to predict the punching capacity of two-way slabs with FRP reinforcement. This modified equation forms the basis of the punching shear equation adopted by ACI 440.1R.

6.2—Serviceability

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

6.2.1 *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 as used for steel-reinforced concrete beams underestimated test results for loads above 50% of ultimate; deflection values were fairly well predicted at load levels up to approximately 30% 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.

Faza and GangaRao (1992) found predicted deflections of FRP-reinforced beams to be underestimated using the effective moment of inertia I_e as prescribed by ACI 318-89 (ACI Committee 318 1989). The authors introduced a new method of calculating the effective moment of inertia of concrete beams reinforced with FRP reinforcement. The new expression was based on the assumption that the concrete section between the point loads was fully cracked, while the end sections were assumed to be partially cracked.

Benmokrane et al. (1994) compared the flexural behavior of concrete beams reinforced with GFRP bars to conventionally reinforced ones. At 25% of M_u, the crack pattern and spacing in FRP-reinforced beams were similar to those in conventional steel-reinforced beams. At service (50% of M_u) and ultimate (90% of M_u) loads, there were more and wider cracks in the FRP-reinforced beams than in the steel-reinforced beams. At service and ultimate loads, FRP-reinforced beams experienced maximum deflections three times higher than steel-reinforced beams. Thus, predicted deflections using the Branson expression for effective moment of inertia I_e, as prescribed in ACI 318-89 (ACI Committee 318 1989), resulted in underestimated deflections. This was attributed to the width, depth, and spacing of the cracks.

A study by Engel et al. (1999) proposed modified ACI 318 equations to compute the deflection of concrete beams reinforced with FRP grids. The researchers tested beams with different grid configurations. Grids with fibers that crossed the grid intersections and grids with fibers turned at a right angle to the grid intersections were tested. For all grid joint designs, the modified ACI code design predictions were in good agreement with the measured deflection response for the midpoint deflection of FRP grid-reinforced concrete beams. The stiffness of the grids can be calculated from a grid section in a stand-alone tension test.

Joh et al. (1999) tested 17 beams reinforced with nine types of FRP bars. The beams were reinforced with GFRP, AFRP, CFRP, and steel. The average ratio of long-term deflection, measured at 9.5 months, to short-term deflection was 0.34 for beams with GFRP bars, 0.65 for beams with AFRP bars, and 0.56 for beams with CFRP bars. The ratio for the steel-reinforced control beam was 0.60. Although the AFRP and CFRP reinforced beams had ratios similar to the control beam, the GFRP-reinforced beams' ratio averaged only 56% of the control beam ratio. This finding is in agreement with Brown (1997), who performed a parametric study of GFRP-reinforced rectangular beams and concluded that the ratio of long-term deflection to short-term deflection was, on average, 55% of the same ratio for steel-reinforced beams.

Toutanji and Saafi (1999) tested concrete beams reinforced with GFRP bars. The authors found that the deflections predicted using the ACI 318 equation were lower than the measured deflections.

A method for determining deflections of FRP-reinforced concrete beams was proposed by Razaqpur et al. (2000), assuming that the moment-curvature relation of FRP sections was linear in the precracked and postcracked stages, and that the tension-stiffening effect could be ignored. The predicted deflections agreed within 5%, with deflections obtained in a number of flexure tests on reinforced concrete beams and slabs measured at service load (50 to 60% of ultimate). The method presented did not account for shear deformations.

Rasheed et al. (2004) developed closed-form expressions for deflection calculation by exactly integrating the bilinear moment-curvature response along the beam span. The results were in excellent agreement with experiments. A modified version of ACI 318 was proposed.

Kassem et al. (2002) tested 12 full-scale concrete beams reinforced with carbon FRP bars. The test parameters included the type of the CFRP bars (mechanical properties and surface finishing) and the reinforcement ratio. They concluded that the deflection predictions based on beams reinforced with GFRP bars may underestimate the experimental deflection values for CFRP-reinforced beams, especially at load levels less than twice the cracking moment.

Because of the variable stiffness, brittle-elastic nature, and particular bond features of FRP reinforcement, deflections of FRP-reinforced concrete members are more sensitive to the variables affecting deflection than steel-reinforced members of identical size and reinforcement layout (Ospina et al. 2001). Deflections in members with FRP reinforcement tend to be greater in magnitude because of the lower stiffness of most commercially available FRP reinforcements. An indirect deflection-control procedure in which deflections in beams and one-way slabs with FRP reinforcement are controlled by the selection of a minimum member thickness for preliminary design (Ospina and Gross 2006) is presented in ACI 440.1R.

6.2.2 *Crack width and patterns*—As with steel reinforcement, the maximum crack width at the tension face of a reinforced concrete member is heavily influenced by the strain level in the reinforcement, the concrete cover, the bar spacing, and the mechanical properties of the reinforcement (Frosch 1999). ACI 440.1R includes a variation of Frosch's (1999) crack control equation found in ACI 318, with a recommendation to limit the predicted crack widths to 0.5 and 0.7 mm (0.020 to 0.028 in.) for exterior and interior exposure conditions, respectively. These values, adopted from CSA S806, are larger than the crack width limits traditionally accepted in steel-reinforced concrete design due to the superior corrosion resistance of FRP.

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

Faza and GangaRao (1992) determined that concrete beams reinforced with spirally deformed FRP reinforcing bars using normal-strength concrete (27.6 MPa [4000 psi]) exhibited crack formation that 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. This sudden propagation of cracks and wider crack widths decreased when higher-strength concrete 51.7 to 69 MPa (7.5 to 10 ksi) and sand-coated FRP reinforcing bars were employed. Based on the assumption that maximum crack width can be approximated by an average strain in FRP reinforcing bar multiplied by expected crack spacing, an expression for maximum crack spacing governed by the following parameters was developed: 1) bond strength of the FRP reinforcing bar; 2) splitting tensile strength of the concrete; 3) area of the concrete cross section in tension; 4) number of reinforcing bars in tension; 5) size of reinforcing bar; and 6) working stress of the FRP reinforcing bar.

In a study by Toutanji and Saafi (2000), the authors suggest that crack width depends primarily on average crack spacing and on average reinforcement stress, which are related to the mechanical properties of the concrete, the reinforcement, and the bond between them. They proposed an equation for crack width prediction.

6.3—Bond and development of reinforcement

The evaluation of bond characteristics of FRP reinforcement is of prime importance in the design of FRP-reinforced concrete members. Bond characteristics are influenced by factors such as: 1) size and type of reinforcement (wires or strands); 2) surface conditions (smooth, deformed, and sand-coated); 3) Poisson's ratio; 4) concrete strength; 5) concrete confinement (helix or stirrups); 6) type of loading (static, cyclic, and impact); 7) time-dependent effects; 8) amount of concrete cover; and 9) type and volume of fiber and matrix.

A study on bond of GFRP reinforcing bars was conducted by Tao (1994) on 102 straight and 90-degree hook specimens. New limits for allowable slip were suggested as 0.064 mm (0.0025 in.) at the free end, or 0.38 mm (0.015 in.) at the loaded end. According to this study, the basic development length of straight and hooked GFRP reinforcing bars could be computed knowing the bar size, ultimate strength of the reinforcement, and concrete strength.

Shield et al. (1999) tested 72 inverted half-beam specimens. The tests did not use stirrups in the beams to simulate the lack of confinement present in bars of bridge decks. GFRP rods of 19.1 mm (0.75 in.) diameter and No. 5 bars were tested, and the development length was determined to be between $42d_b$ and $63d_b$. Nonuniformly spaced cracks perpendicular to the bar developed.

Katz et al. (1999) studied the bond behavior of five reinforcing bar types at room and high temperatures. One bar had lugs on the surface to provide the desired bond to concrete. Three other bar types had an extra layer of resin on the surface and a coat of sand. Also, a steel bar was included in the study for comparison purposes. The bars were tested at temperatures of 20, 130, and at 250 °C (68, 266, and 482 °F). Two main bond failure mechanisms were observed. At room temperature, portions of the concrete surface in contact with the FRP bars pulled out in some specimens. At high temperatures, however, all the slip took place at the surface of the FRP bar because the shear properties of the resin on the surface of the FRP bar deteriorated. At 250 °C (482 °F), all FRP bars lost 80% or more of their room temperature bond strength, while the steel bars lost only 40% of their room temperature bond strength. The bond strength of all FRP bars decreased with increases in temperature, and it appeared to level off at a temperature of approximately 200 °C (392 °F). The authors agreed with a model proposed by Greszczuk (1969) that assumed the controlling parameters for bond strength were the shear modulus and the thickness of the outer layer of resin on the FRP reinforcing bar. The bond stiffness (initial slope of the bond-slip curve at the loaded end) decreased with increasing temperature. FRP bars with helical fiber wrapping and sand coating tended to exhibit better bond behavior than was seen in bars that relied on lugs of polymer for bond strength. The FRP bars with lugs of resin showed abrupt drops in the bond-slip curves after the peak load was reached.

Several weaknesses in the use of pullout tests (cylinders or pullout specimens) have been identified in discussions in ACI Committees 408 and 440 because such tests do not sufficiently account for all types of mechanical behavior, such as the flexural curvature and the combinations of shear and flexure. Many attempts have been made to find a better standard test method.

Malvar (1995) conducted tension and pullout bond tests on four types of reinforcing bars and studied the effects of confining pressure on bond. All bars had a diameter of 19 mm (3/4 in.) and were composed of E-glass fibers with a fiber volume fraction of 45%, embedded in a vinylester or polyester resin.

Five different confining pressure levels were studied: 3.45, 10.3, 17.2, 24.1, and 31 MPa (500, 1500, 2500, 3500, and 4500 psi). The measured adhesion (bond stress at zero average slip) was between 0.69 and 2.1 MPa (100 and 300 psi). Beyond this adhesion, the slope of the bond-slip curve appeared to increase with higher confinement. For an identical amount of confinement, the bond strength for a steel bar was, on average, 1.2 to 1.5 times higher than that for an FRP bar. Large variations in indentation depths of the bars resulted in a large variation of bond strength. Bond strength can be increased threefold by increasing confining pressure by a factor of 7.

Cosenza et al. (1997) reviewed the current knowledge of bond performance of FRP bars embedded in concrete. They found that the longitudinal shear modulus of the FRP bar was an important parameter. Coating the surface of bars with sand enhanced the bond strength and bond stiffness, but led to more brittle bond behavior. Bars with a surface coated with sand had excellent bond strength ($\tau_m > 10$ MPa [1.45 ksi]), even higher than steel bars. The bond strength of smooth FRP bars was poor ($\tau_m < 2.37$ MPa [0.34 ksi]). The bond strength of spirally glued bars had values slightly higher than those of smooth bars (τ_m = 4.50 MPa [0.65 ksi]).

Wambeke and Shield (2004) consolidated a database of 269 beam bond tests. The database was limited to beam-end tests, notch-beam tests, and splice tests. Two equations for development length were developed; one based on a splitting mode of failure, and the other on a pullout mode of failure. These equations form the basis for the design recommendations in ACI 440.1R.

Research on spliced FRP bars conducted by Aly (2005), Aly and Benmokrane (2005), and Aly et al. (2005, 2006) found that the development length of FRP bars was dependent on the square root of the longitudinal modulus of the bars.

6.4—Fatigue performance

Benmokrane et al. (1999) tested concrete slabs 3200 mm (126 in.) long, 1000 mm (39.3 in.) wide, and 260 mm (10.2 in.) thick, reinforced with CFRP gratings. The beams were subjected to 4 million cycles of loading, with load amplitude of 10 to 100 kN (2.25 to 22.5 kips). The first 2 million cycles were applied at a frequency of 2 Hz, and the remaining cycles at 3 Hz. After 4 million cycles, the maximum deflection was 12 mm (0.47 in.) (span/250). The same slab had a deflection of 8 mm (0.34 in.) under a 100 kN (22.5 kips) load applied statically before the fatigue test.

Rahman et al. (2000) conducted a 4 million cycle fatigue test on a full-scale model of a bridge deck slab. The slab was a composite structure supported by steel I-beams. The deck was reinforced using CFRP grating size C10 bars with 46 mm^2 (0.071 in.^{2}) cross-sectional area and 100 mm (4 in.) grid spacing. The FRP reinforcement ratio was 0.3%. The researchers found that the deflection after 4 million cycles increased by 26% when compared with the deflection under static 100 kN (22.5 kip) loading. A deflection increase of 32% was observed for the 125 kN (28.1 kip) loading. Crack widths increased 42% after 4 million cycles under the 100 kN (22.5 kip) load. Stresses in the reinforcement were also increased by the fatigue loading by approximately 10%. The failure modes of the slab were punching shear in five loading cases and flexure in one loading case. The minimum capacity of the slab was over five times the design service load.

Kumar and GangaRao (1998) found that the rate of fatigue degradation in bridge decks reinforced with GFRP bars was similar to that of bridge decks reinforced with steel for a range of stringer stiffnesses and amount of composite action.

El-Ragaby et al. (2006) tested four full-size concrete deck slabs reinforced with GFRP bars to failure under a concentric fatigue loading. They concluded that the GFRP reinforced concrete bridge decks had better fatigue performance and longer fatigue life than equivalent steel reinforced decks.

6.5—Members reinforced with FRP grating systems

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 slabs were designed for a live load moment designated by AASHTO (1989) using a nominal HS-25 loading with a live load impact factor of 30%. 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. The service load midspan deflections for all FRP-reinforced slabs were close to the allowable limit of 4.9 mm (0.192 in.). Deflection was found to stabilize after a limited number of cycles. All slabs failed at loads in excess of three times the service load.

Bank and Xi (1992) tested the performance of four full-scale two-span continuous concrete slabs, doubly reinforced (top and bottom) with 51 mm (2 in.) deep, commercially produced, pultruded FRP gratings having longitudinal bar spacings of 76 and 51 mm (3 and 2 in.) on center, respectively. Slabs were loaded as follows: first, under a monotonically increasing load to 115 kN (26 kips), then subjected to 10 loading/unloading cycles of 0 to 115 kN (0 to 26 kips), and finally loaded monotonically to failure. 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 ratios 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 before failure. The local radius of curvature in the positive moment region generally satisfied AASHTO serviceability specifications. In the negative moment region, however, this criterion was violated. Service load deflections were well below the L/500 limit, where L is the length of the beam.

6.6—Members reinforced with FRP grids

Since 2004, carbon fiber/epoxy resin grids have been used to reinforce a wide variety of precast concrete structures including insulated wall panels, architectural cladding panels, double tees, and residential members. In addition to being used a secondary reinforcement for crack control, the grids have also been used as primary reinforcement.

Most grids being used as internal reinforcement for concrete parts are constructed using continuous carbon fibers impregnated with thermoset epoxy resins. Glass and basalt fibers may also be used. Grids can be produced by a number of manufacturing processes. Grids can be woven and then impregnated with a thermoset or thermoplastic resin.

Grids further enhance the potential for more effective reinforcement of concrete structures using composite materials because they can be used near either surface of the concrete structure (but embedded within the concrete) to control cracking, reduce the thickness of the concrete structure, and to provide tension reinforcement. Grids may be used as the only reinforcement material or in combination with traditional reinforcement such as reinforcing bars and welded wire fabric. Because of differences in the physical and mechanical behavior of FRP materials versus steel, combined with the fact that grids offer the unique opportunity to position a noncorrosive structural reinforcement very close to the surface, unique guidance on the engineering and construction of concrete structures reinforced with FRP grids is needed and is being developed as an activity of ACI Committee 440.

6.7—Pavement applications

FRP dowel bars have been extensively investigated by Davis and Porter (1999) and others. These dowel bars have been used primarily in highway concrete pavement construction. The results of this research have shown that GFRP dowels are able to carry the shear transfer across the pavement joint. For example, 44 mm (1.75 in.) diameter GRFP dowels spaced at 200 and 300 mm (8 and 12 in.) were shown to give favorable results compared with the typical 38 mm (1.5 in.) diameter steel dowel for up to 10 million cycles of repeated loading simulating 40 kN (9 kip) wheel loads on each side of the joint for full-scale pavement slabs.

Choi and Chen (2003, 2005) studied the concrete stresses induced in a GFRP-reinforced concrete slab due to concrete shrinkage and temperature variations. The results showed that the stress level in concrete was reduced with GFRP reinforcing bars because of the low Young's modulus of GFRP. The stress distribution and crack width in the GFRP-reinforced continuously reinforced concrete pavement section were predicted under various design considerations, such as the coefficient of thermal expansion (CTE) of concrete, the friction from the subbase of the pavement, and the bond-slip between concrete and reinforcement. The stress levels in the GFRP reinforcement, the crack widths, and the crack spacing of the proposed pavements were shown to be within the allowable AASHTO pavement design requirements. Other work on FRP-reinforced concrete pavements has been conducted by Walton and Bradberry (2005).

CHAPTER 7—PRESTRESSED CONCRETE MEMBERS

Much of the available research has concentrated on the short-term static behavior of bonded FRP prestressed beams. Both pretensioned and post-tensioned systems have been considered. ACI 440.4R provides details of prestressing systems for FRP and guidelines for the design of FRP prestressed concrete structures.

7.1—FRP tendons

This section examines commercially available tendons and anchorage systems. For more detailed information, the reader is referred to ACI 440.4R. Properties and characteristics of selected available tendon/anchorage systems are summarized in Table 7.1 and are based on the manufacturer's published data. The trade names of the products are used for clarity. The properties of the tendons may change as the manufacturers modify their products. More details about the manufacturers are provided in ACI 440.4R. Variations in the properties of the tendons (for example, modulus and ultimate stress) are generally due to differences in the fiber volume fraction or to the orientation of the fibers in the tendon. For example, Arapree tendons have a volume fraction of 0.45, whereas Parafil rope has no polymer at all. Thus, the strength and modulus values for Parafil are higher. Also, the CFCC tendons are twisted seven-wire strands; thus, not all the fibers are aligned along the longitudinal axis. Therefore, the modulus and strength properties are lower than the Leadline tendons that have all the fibers oriented longitudinally.

7.2—Anchorages

Conventional anchorage systems for steel prestressing reinforcement have hardened teeth that bite into the reinforcement to enhance transfer of stress. With FRP reinforcement, these teeth cause premature failure of the reinforcement at the grip location. Different anchorage systems have been used with FRP tendons, such as clamp, plug and cone, resin sleeve, resin potted, metal overlay, and split wedge anchorages, as shown in Fig. 7.1. These anchorages and their gripping mechanisms are briefly described herein and in more detail in ACI 440.4R.

7.2.1 *Clamp anchorage*—A clamp anchorage (Fig. 7.1(a)) consists of two or four grooved steel plates sandwiching the FRP tendon and held together by pretensioned bolts and springs. The force is transferred by a shear-friction mechanism (Malvar and Bish 1995).

7.2.2 *Plug and cone anchorage*—The plug and cone (or barrel and spike) anchorage (Fig. 7.1(b)) is made of a socket housing and a conical spike (Burgoyne 1992). Such a system is suited to anchor Parafil ropes where the fibers are not encased in resin media but are held only by an outer protection sheath.

7.2.3 *Resin sleeve anchorage*—In this anchorage system, the FRP tendon is embedded in resin filling a metallic housing such as a steel or copper tube (Fig. 7.1(c)). The resin used ranges from nonshrink cement with or without sand to epoxy-based material. The load transfer mechanism depends completely on the bonding and interlocking between the anchorage components. This type of anchorage has been used for both round and flat tendons.

7.2.4 *Resin potted anchorage*—The potted anchorage (Fig. 7.1(d)) varies depending on the internal configuration of the socket: straight, linearly tapered, or parabolically tapered. The load transfer mechanism from the tendon to the sleeve is by interfacial shear stress that is a function of bonding and normal stress produced by the variation of the resin profile (Kim and Meier 1991). The practical drawbacks include precutting the tendons to length and the curing time for the resin or grout.

7.2.5 *Metal overlaying*—The die-cast wedge system (Fig. 7.1(e)) for the carbon fiber composite cable (CFCC) requires that the tendon length be predefined so that metal can be cast onto the tendon during fabrication with the result that adjustment on site is not possible. The tendon and the metal material are compressed by a typical wedge anchorage similar to those used on steel tendons. The load transfer of this anchorage is achieved by shear friction, which is a function of compressive stress and friction at the contact surfaces.

7.2.6 *Split wedge anchorage*—Split wedge anchorages (Fig. 7.1(f)) are generally preferred because of their compactness, ease of assembly, reusability, and reliability. The wedge system is widely used in anchoring steel tendons. The anchorages are modified for use with FRP tendons by

Table 7.1—Manufacturer's material properties (Nanni et al. 1996)

Tendon	Fiber type	Type	Fiber volume ratio	Nominal diameter, mm (in.)	Cross section, mm^2 (in.^{2})	Young's modulus, GPa (ksi)	Ultimate stress, MPa (ksi)	Ultimate strain, %	Density, g/cm^3 (lb/ft^3)	Poisson's ratio	Anchorage type	Reference
Arapree	Aramid	—	0.45	7.5 (0.30)	44 (0.071)	60 (8700)	1400 (200)	2.4	1.25 (78)	0.38	Wedge	Gerritse and Werner 1988; Sireg 2007
Parafil	Aramid	Type G	1.00	13.5 (0.53)	87 (0.135)	120 (17,400)	1200 to 1900 (170 to 270)	1.5	1.44 (90)	N/A	Plug and cone	Burgoyne 1988; Linear Composites 2007
Leadline	Carbon	Indented	0.65	7.9 (0.31)	46 (0.071)	150 (21,800)	2250 (325)	1.3	1.53 (95)	0.27	Wedge	Mitsubishi 2005
CFCC	Carbon	Twisted seven wire	0.65	12.5 (0.49)	76 (0.118)	137 (19,900)	1900 (270)	1.6	1.50 (94)	N/A	Resin-filled die-cast/ wedge	Santoh 1993

increasing their length and placing a soft metal sleeve around the tendon to prevent notching. The mechanism of gripping relies on friction and clamping force between the wedges, barrel, and tendon.

A metallic anchorage was developed (Sayed-Ahmed and Shrive 1998; Campbell et al. 2000) for 8 mm (0.31 in.) CFRP. This anchorage has also been studied experimentally and analytically by Al-Mayah et al. (2001). The researchers also developed a new anchor system that allowed the maximum load capacity of the CFRP rod to be achieved and required no presetting of the wedges (Al-Mayah 2004; Al-Mayah et al. 2003). Nonmetallic versions of this anchorage have also been developed consisting of either ultra-high-performance concrete wrapped with CFRP sheets (Reda Taha and Shrive 2003a,b) or carbon fiber-reinforced reactive powder concrete (Shaheen 2004).

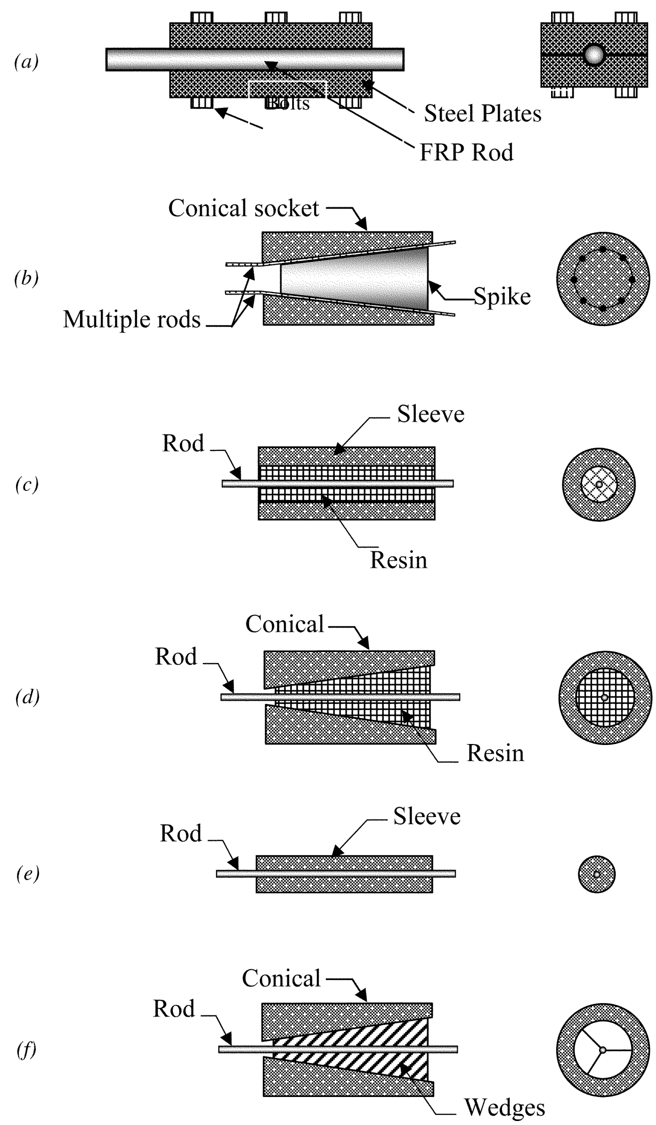


Fig. 7.1—(a) Clamp; (b) plug and cone; (c) resin sleeve; (d) resin potted; (e) metal overlay; and (f) split wedge anchorage systems.

7.3—Flexural behavior

The load-deflection response of FRP prestressed beams is generally bilinear due to the linear elastic nature of the FRP tendons. The beams exhibit a linear load-deflection response up to cracking and with a reduced stiffness after cracking up to failure (Abdelrahman et al. 1995; Abdelrahman and Rizkalla 1997; Arockiasamy et al. 1995).

Two modes of flexural failure have been reported for FRP prestressed beams: 1) rupture of prestressing reinforcement; and 2) crushing of concrete in compression. These failure modes are dependent on whether or not the cross section has sufficient rotation capacity to develop the ultimate tensile stress of FRP reinforcement. When the concrete section has enough rotation capacity (enough concrete area in compression), failure by rupture of the tendon occurs. Rupture of the tendon is normally accompanied by horizontal cracks at the level of the prestressing reinforcement, causing spalling of the concrete cover. According to a number of researchers (Abdelrahman et al. 1995; Naaman and Jeong 1995; Currier 1995), rupture of FRP prestressed reinforcement is accompanied by a release of elastic strain energy that is partly absorbed by the concrete, thus causing extensive cracking and spalling. When failure of a prestressed beam is caused by crushing of concrete (when the cross section has insufficient rotation capacity), cracking is less extensive compared with those beams that fail by rupture of tendons. Some researchers have reported that failure of FRP prestressed beams by crushing of concrete results in increased ductility. This behavior is attributed to the progressive failure of concrete in compression (Grace and Abdel-Sayed 1998a).

Comparison of flexural behavior of similar concrete beams prestressed with FRP and steel reinforcement shows that the beams possess very similar precracking stiffnesses to each other, but the postcracking stiffnesses are markedly different. Abdelrahman and Rizkalla (1997) found that beams pretensioned with FRP reinforcement had a much lower postcracking stiffness resulting in larger postcracking deflections. The ultimate deflection in the steel prestressed beam was, however, much greater due to yielding of steel reinforcement. The ultimate load for the CFRP beams, however, was higher compared with steel prestressed beams.

For conventional steel prestressing, bonded tendons are often preferred due to concerns over corrosion of unbonded tendons. Because FRP tendons are resistant to corrosion, bonding may not be required. This would save on the extra expenditure required for grouting. Unbonded tendons also facilitate replacement of prestressing tendons in case of strength inadequacy or increase in the design loads. Unbonded FRP prestressed members are also more deformable than similar bonded prestressed members. Problems that arise with unbonded FRP tendons in beams include anchorage of FRP reinforcement and the fatigue strength of these beams, especially at the tendon anchorage junction.

Post-tensioned beams with unbonded FRP tendons exhibit less cracking under short-term loading conditions compared with beams with bonded prestressed reinforcement (Maissen and De Smet 1995), but the cracking is normally concentrated in a few wide cracks. The load-deflection response of unbonded post-tensioned beams is bilinear, but compared with pretensioned and bonded post-tensioned beams, the postcracking stiffness is much less.

Research on crack widths of flexural members prestressed with bonded FRP was performed by Dolan et al. (2000). Both monotonic and cyclic loadings were studied. Crack widths were larger than for comparable steel prestressed beams because of the lower modulus of the FRP. Fatigue loading increased the crack widths slightly.

Bryan and Green (1996) studied the low temperature behavior of beams prestressed with bonded CFRP. No significant differences in behavior were observed for short-term exposure to low temperature.

Braimah et al. (1998) and Marshe and Green (1999) investigated the feasibility of transversely prestressing concrete bridge deck slabs with bonded CFRP tendons. They found that this approach was effective in improving the serviceability and strength of the deck slabs. CFRP prestressed slabs performed as well, if not better than, comparable steel prestressed deck slabs in terms of serviceability and strength requirements.

Parallel-lay aramid ropes have been developed for use as unbonded prestressing cables at the University of Cambridge (Burgoyne 1992, 1993, 2001). Parafil is not strictly an FRP because no resin (polymer) binds the fibers together. The ropes have an external polyethylene sheath that serves to retain the shape of the rope, and also to shield the aramid fibers from UV light, to which they are susceptible. The intended uses for the ropes are as external prestressing tendons, or as unbonded tendons inside ducts in the section. This arrangement avoids any problems associated with the fibers coming into contact with the alkaline concrete. Stresses are limited by stress-rupture considerations, typically approximately 1000 MPa (145 ksi) at initial stressing. Several prototype structures have been built with these tendons in Japan (as external strengthening tendons), in England (for prestressed brickwork), in Canada (prestressed timber), and in Scotland (as stay cables to the all-composite bridge at Aberfeldy).

7.4—Fatigue behavior

Fatigue resistance of beams is typically investigated by calculating the stress range Δf_p produced in the prestressing tendon during cyclic loading, and comparing this stress range with that obtained from the S-N curves for the prestressing tendon.

If the level of precompression in a prestressed concrete member is sufficient to ensure an uncracked section throughout the service life of the member, the fatigue characteristics of the prestressing reinforcement and anchorages are not likely to be critical design factors. In cracked prestressed concrete members, however, localized stress increases in prestressing reinforcement across cracks in pretensioned beams, fretting fatigue of prestressing reinforcement in post-tensioned beams, and the fatigue of the tendon-anchorage assembly become very significant. ACI Committee 215 and the Post-Tensioning Institute have recommended that tendon-anchorage assemblies consisting of prestressing steel and anchorage systems be able to withstand, without failure, 500,000 cycles of stress varying from 60 to 66% of the specified ultimate strength of the assembly (ACI 215R; Post-Tensioning Institute 1990).

Iwamoto et al. (1993) conducted tests on concrete beams reinforced with two types of bonded prestressing tendons: AFRP or steel wire. Three levels of initial tension of the tendon were selected at 40, 60, and 70% of the tensile strength of the FRP bar. The failure modes of the beams with aramid tendons included tendon rupture, development of bond cracks, and shear compression failure. The fatigue strength at 2 million cycles was not less than 65% of the static ultimate strength of the beams. The bond between aramid tendons and concrete deteriorated more than that of the specimens containing prestressing wire because of rubbing between aramid fibers and concrete. Therefore, the fatigue strength of beams with aramid fiber tendons cannot be predicted by fatigue tests on isolated aramid fiber bars. The rigidity of the beam did not decrease much under cyclic loading.

Abdelrahman et al. (1995) tested four prestressed beams using two bonded CFRP reinforcement types (CFCC and Leadline™). The fatigue loading was performed with a maximum load equal to the cracking load and the minimum equal to 70% of the cracking load. The authors concluded that the beams survived 2 million cycles of fatigue loading without measurable effects on the beam stiffness.

Dolan et al. (2000) reported that cracked CFRP pretensioned beams showed no fatigue failure after 3 million flexural cycles with nominal tensile stresses of $0.5\sqrt{f_c'}$ MPa ($6\sqrt{f_c'}$ ksi) at the extreme fiber of the beams. Gradual softening of the girders was observed, but the beams indicated no loss of strength due to the fatigue loading.

Braimah (2000) completed tests on unbonded CFRP prestressed concrete beams. Only one CFRP tendon survived the full 2 million cycles. Most of the tendons ruptured at the anchorage/tendon interface when the fatigue stress range in the tendon was greater than 100 MPa (15 ksi). Braimah concluded that totally unbonded tendons may not be appropriate for fatigue loading due to stress concentrations at the tendon/anchorage junction.

7.5—Time-dependent behavior

Material research has shown that FRP reinforcement has different relaxation properties than steel reinforcement (Erki and Rizkalla 1993) leading to the need to evaluate the long-term behavior of FRP prestressed members. Long-term prestress losses are computed based on creep and shrinkage of the concrete and relaxation of the tendons. Losses for FRP tendons due to creep and shrinkage are typically less than the corresponding losses for steel tendons due to the lower modulus of elasticity of the FRP. Relaxation losses are more problematic because there is little data that describes relaxation loss profiles. Relaxation characteristics vary with the fiber type, and are generally less than 12% over the life of the structure (JSCE 1993).

Sen et al. (1999) examined AFRP pretensioned beams under simulated tidal and temperature changes and found that the combined effects of temperature change and tidal simulation were detrimental to the performance of AFRP pretensioned beams. The loss of moment capacity of the beams increased with length of exposure, varying between 43 and 55% within the first 21 months.

Early research results from long-term tests showed that FRP prestressed beams exhibited similar behavior to those prestressed with steel. The long-term deformations depended on the fiber type, and were often higher than those of steel prestressed beams. Currier (1995) found that long-term deformations of FRP prestressed beams could be predicted by using conventional methods developed for steel prestressed beams, albeit with minor modifications.

Matthys and Taerwe (1998) tested three series of pretensioned concrete slabs using steel wires or AFRP tendons as prestressing reinforcement. They found that the long-term deformations of the AFRP pretensioned slabs were higher than those of the steel pretensioned slabs due to the lower elastic modulus and higher relaxation characteristics of the AFRP tendons.

Braimah et al. (1999) and Braimah (2000) tested three pretensioned CFRP beams and one steel prestressed beam under sustained loading for up to 2 years. The long-term behavior of the CFRP beams was very similar to that of the steel prestressed beam. A model for predicting the long-term deflections of the beams was also developed and was shown to predict the experimental measurements with reasonable accuracy.

7.6—Ductility and deformability

Ductility describes the ability of a structural member to sustain large inelastic deformations in comparison with elastic ones before collapse without significant loss in resistance. FRPs have no ductility as is commonly associated with yielding of steel because they exhibit linear elastic behavior until brittle failure. The lower strain at failure is of significant concern to design engineers. A portion of the strain is used for prestressing and flexural strain, leaving very little strain reserve for safety against brittle failure.

The most common ductility ratios or indexes are given in terms of curvature, rotation, or deflection as follows

$$\mu_\phi = \frac{\phi_u}{\phi_y},\ \mu_\theta = \frac{\theta_u}{\theta_y},\text{ or } \mu_\Delta = \frac{\Delta_u}{\Delta_y} \tag{7-1}$$

where μ = ductility index; ϕ = curvature, θ = rotation, and Δ = deflection. The subscript y denotes yielding of the reinforcement, while u denotes ultimate limit state. Because FRP reinforcements do not yield, the conventional definition of ductility is not applicable. A number of alternative definitions have been suggested in the literature.

Abdelrahman et al. (1995) proposed deformability ratio criteria that relied on the equivalent deformations of the uncracked section and deformations at ultimate. Curvature and displacement-based deformation indexes were defined as the ratio of the curvature of deflection of the beam at failure to the equivalent elastic curvature or displacement of the uncracked beam at a load equal to the ultimate load.

Naaman and Jeong (1995) proposed an energy-based definition of the ductility index for perfectly elasto-plastic behavior. According to the proposed definition, the ductility ratio was expressed as the ratio of the total energy to the elastic energy at the failure state of a beam.

The energy-based definition of the ductility index of concrete structures appears to be the most elegant, and a number of researchers including Grace and Abdel-Sayed (1998a,b) have tried to improve flexural ductility by using different techniques to achieve an increase in the inelastic component of the total energy in FRP prestressed or reinforced structures. Most of these efforts have, however, achieved increased deflections at ultimate without a significant increase in the inelastic energy.

According to Lees and Burgoyne (1999), the high rotation capacity in FRP prestressed beams is essential to warn of incipient failure. These large rotations are most often achieved in unbonded or partially bonded beams where the beam rotations are concentrated in a few cracks. If the tendons are partially bonded (for example, where the bond is chosen to be of a known low shear strength or the tendon is bonded intermittently along the length), then a system can be designed where a controlled amount of debonding takes place at the crack locations. In this manner, both a high ultimate moment capacity and a large rotation capacity concentrated at the few cracks can be achieved.

Another approach is to use a ratio of the curvatures under ultimate and service loads (Dolan and Burke 1996).

Deformability indexes based on this rationale were calculated for harped prestressed beams from published data (Table 7.2). The service loads were considered as the load to produce a tensile stress in the concrete of $0.25\sqrt{f'_c}$ MPa ($3\sqrt{f'_c}$ in psi). The index is effectively a function of the ratio of ultimate strain to the prestressing strain, with slight modification due to the differences in the neutral axis of

Table 7.2—Indexes and ratios to evaluate deformability

Reference	Condition	Deformability index	Ultimate/ initial prestress	Maximum deflection/ span	*c/d* ratio
Abdelrahman (1997)	Under-reinforced	3.6	3.6	1/32	0.115
Abdelrahman (1997)	Over-reinforced	4.4	3.6	1/32	0.345
Currier (1995)	Over-reinforced	2.4	2.0	1/32	0.307

Table 7.3—Typical transfer length and development length based on tests of various FRP tendons

Material	Type	Diameter, mm (in.)	f_{pi}/f_{fu}	l_t/d_b	l_d/d_b
Aramid	Arapree	10.0 (0.39)	0.5 to 0.7	16 to 50	100
Carbon	Leadline	7.9 (0.31)	0.5 to 0.7	50 to 80	175
	CFCC	8.3 (0.33)	0.5 to 0.7	50	N/A
Steel	Seven-wire	12.7 (0.50)	0.75	50 to 60	106

elastic and inelastic behavior. Abdelrahman's indexes are higher because his prestress level was lower than the other tests.

The data indicate that the most efficient method to obtain high deformability is to reduce the initial prestress. Reducing the initial prestress strain provides more tendon strain reserve, greater curvature or deflection capacity, and a higher index.

One other method for improving the deformability, ductility, or both of beams prestressed with FRP is combining FRP prestressed reinforcement with non-prestressed stainless steel reinforcement (Tung and Campbell 2002).

7.7—Transfer and development length

The transfer and development length of an FRP tendon is a function of the perimeter configuration area and surface condition of the FRP, the stress in the FRP, and the method used to transfer the FRP force to the concrete. The mechanism of bond differs between FRP and steel strands due to the large variation of FRP bars in terms of shapes, surface treatments, and elastic moduli.

7.7.1 *Transfer length*—The transfer length in pretensioned concrete is the length required to transfer the full prestressing force to the concrete. In general, most FRP tendons exhibit transfer lengths shorter than that of steel.

Nanni et al. (1992) examined the transfer length of braided epoxy-impregnated AFRPs and found that friction was the predominant bonding mechanism in aramid fibers and that these fibers showed little tendon slippage compared with steel. Taerwe and Pallemans (1995) also studied AFRP tendons and suggested a transfer length of 16 times the nominal diameter of the tendons. Ehsani et al. (1997b) found the transfer length to be between 33 and 50 bar diameters, depending on the type of tendon.

Mahmoud and Rizkalla (1996) studied the bond characteristics of CFRP tendons in 24 pretensioned concrete beams and found that the measured transfer length varied from 450 to 650 mm (17.7 to 25.6 in., or 56 to 81 bar diameters) for Leadline™ tendons, and from 300 to 425 mm (11.8 to 16.7 in.) for CFCC strands. After further study and comparison with an analytical model, they recommended a formula for transfer length dependent on the tendon prestress, diameter and type, and the concrete strength (Mahmoud et al. 1997).

Ehsani et al. (1997b) also conducted tests on Leadline™ (8 mm [0.31 in.]) and CFCC (8.3 mm [0.33 in.]) carbon tendons and found the transfer length to be 54 bar diameters for Leadline™ and 50 bar diameters for CFCC.

Soudki et. al. (1997) reported the transfer length for Leadline™ CFRP tendons as 80 times the bar diameter, and found that existing models for steel may give unconservative transfer lengths for the CFRP tendon.

7.7.2 *Flexural bond length*—Nanni et al. (1992) examined the development and flexural bond lengths in 25 AFRP pretensioned concrete beams. The development length was determined to range from 80 to 120 times the nominal diameter of the tendon.

Ehsani et al. (1997b) examined the transfer and flexural bond lengths of CFRP and AFRP prestressing tendons and compared the results with those of steel strands. Two types of CFRP strands were tested: Leadline™ and CFCC. Three AFRP types were tested and they concluded that the ACI development length requirements for steel strands were conservative for AFRP tendons, but were not adequate for Leadline™ CFRP tendons.

7.7.3 *Summary*—The bond and development for prestressing FRP tendons made of carbon, aramid, or glass fibers are intended to provide bond integrity for the strength of the member. Typical values for transfer length and development lengths of various FRP tendons are given in Table 7.3.

7.8—Shear behavior

One of the earliest applications of FRP as shear reinforcement was demonstrated in the one-third scale model beams for the AASHTO girders of the Taylor Bridge in Canada, which were tested by Fam et al. (1997). In this study, five I-girders, each 9.3 m (30.5 ft) long, were exclusively reinforced for shear and prestressing using CFRP, including both CFCC and Leadline™ reinforcement.

To simulate the composite action of the bridge deck, the CFRP stirrups were projected from the girder into the slab, which was cast after fabrication of the girders. Various web reinforcement ratios were used for each type of CFRP reinforcement. This was controlled by the stirrup size and number of legs. The web reinforcement ratio was shown to affect the induced stress level in stirrups and the diagonal crack width; however, the effect was not linearly proportional to the web reinforcement ratio. Also, no slip was measured between the deck slab and girder, which suggested that the stirrups had adequate dowel strength. Because these beams were controlled by flexural capacity, the web reinforcement ratio had virtually no effect on the overall performance.

The study demonstrated the feasibility of using CFRP harped prestressed reinforcement and shear reinforcement in bridge girders, and led to successful detailing of stirrups at the junction between the web and bottom flange to avoid premature shear failure. The study also resulted in a successful design and construction of the Taylor Bridge in Canada, which was the first to use FRP shear reinforcement of this nature.

A comprehensive study was later performed on shear strength of beams reinforced by GFRP and CFRP stirrups (Shehata 1999) and showed that the effective capacity of FRP stirrups might be as low as 50% of the guaranteed strength as described in detail in Section 6.1.2.

7.9—External tendons

External unbonded tendons can be an effective method to exploit the properties of FRP. The unbonded FRP tendon is strained much less than a bonded FRP tendon, and its brittle behavior may be less critical than for a bonded FRP tendon. Nevertheless, the overall behavior is dependent on the performance of the anchorage, and the tendons are fully exposed to potential damage. Furthermore, the contribution of the unbonded tendon to the flexural strength of the member is significantly less than that of a bonded tendon.

CSA S6 permits FRP tendons as the main components of an external post-tensioned strengthening system. The stress limits for external tendons are slightly lower than those for bonded tendons, but GFRP tendons are allowed in nonalkaline grout.

In external prestressing, special attention should be paid to the anchorage that grips the tendon and the interaction of the tendon with the deviation saddles. A slight reduction in the cable strength of CFCC is observed when the angle of curvature at the saddles is 10 degrees or less (Santoh 1993). To reduce the friction between the tendons and the saddle, a cushion material or Teflon sheaths may be used at the saddle point.

In a comparative study between the behavior of concrete members prestressed with internal unbonded and external prestressed FRP tendons, Burgoyne (1992) tested two beams prestressed with parallel-lay aramid (Parafil) ropes. The beam with the internal unbonded tendon did not collapse completely, but the externally prestressed beam failed suddenly and completely, with a total loss of prestressing. Based on this test, Burgoyne proposed that if the tendons were external to the concrete, they should pass through loose rings so that, in the event of failure, the tendons would be forced to deflect with the beam.

Other studies on beams with external tendons were conducted by Mutsuyoshi and Machida (1993), Tsuchida et al. (1993), Miyamoto et al. (1994a,b), El-Hacha (1997), and Jerrett et al. (1996). Saeki et al. (1993, 1995) found that beams prestressed with external tendons could withstand 2 million cycles of fatigue loading.

Horiguchi et al. (1995) studied the effects of temperature on the fatigue properties of externally prestressed concrete beams. They found that the ultimate flexural strength of the beams improved, while their fracture toughness decreased at low temperature. In the fatigue tests, only tension failure of steel reinforcement was observed, but the fatigue strength of external prestressing cables decreased at low temperature.

Grace and Abdel-Sayed (1996a,b) suggested a new system consisting of precast prestressed concrete double-tee girders covered with a deck slab. The double-tee girders were reinforced with GFRP bars and prestressed internally and externally with harped CFRP tendons. The test results indicated that the system was a promising approach to produce a ductile, crack-free and long-lasting bridge system. Also, the tested double-tee bridge had confirmed its ability to sustain a repeated load of 60% of its ultimate load-carrying capacity for 7 million cycles without any significant changes in its dynamic and static characteristics.

El-Hacha et al. (1996) conducted an analytical study on externally prestressed beams and compared the results with available experimental results (that is, strength and stress in prestress at ultimate). They concluded that the equation proposed by Harajli and Kanj (1991) provided the closest results to the experimental values on the conservative side, while the equation proposed by Naaman and Alkhairy (1991) overestimated the strength of beams prestressed with FRP.

Miyamoto et al. (1998) carried out an analytical and experimental study. They applied external prestressing to strengthen the Misaka Bridge in Japan to evaluate the performance of the bridge such as load-carrying capacity and deflection control. The bridge was 35.3 m (116 ft) in length, 6.0 m (19.7 ft) in width, and composed of three simply supported composite steel girders. The external tendons were placed in a queen post arrangement through a rigid concrete diaphragm. Static load tests and moving load tests were carried out before and after completion of the strengthening for comparison purposes. They compared the calculated results of finite element analysis with the measured values, and good agreement was obtained. Their static load test results showed that the strengthening reduced deformation to approximately 50% of the prestrengthening level. The moving vehicle load test results showed that the dynamic deformation was not significant.

7.10—Prestressed poles

Road environments are known to be very aggressive, causing corrosion at the fixture of plain steel, steel-reinforced, or steel-prestressed concrete lighting posts. When steel tendons are used, in many applications, a significant concrete cover (40 to 50 mm [1.5 to 2.0 in.]) is required to protect the prestressing steel from aggressive internal or external environments, or both. In contrast, when durable CFRP tendons are used, only a relatively small concrete cover (15 to 20 mm [0.6 to 0.8 in.]) is required. Therefore, CFRP-prestressed poles can be significantly lighter than equivalent steel-prestressed structures. Recent work has considered the design and installation of 8 m (26 ft) nominal height CFRP prestressed concrete lighting columns fulfilling the requirements of the relevant European Standards (Terrasi and Lees 2003).

CHAPTER 8—REPAIR, STRENGTHENING, AND RETROFITTING

Applications of FRP materials for retrofitting and strengthening existing concrete structures have been rapidly growing in North America (Labossière et al. 1997; Hazen et al. 1998; Grace and Abdel-Sayed 2003), Europe (Meier et al. 1992; Steiner 1996; Nanni 1997; Matthys et al. 2004; Blasi et al. 2004; Rostasy et al. 2004), and Japan (Ichimasu et al. 1993; Katsumata et al. 2001). This strengthening technique provides an efficient, noncorroding alternative to externally bonded steel plates. The purpose of retrofitting by FRP composite strengthening systems is to strengthen or improve the flexural capacity, shear capacity, axial capacity, and ductility, or any combination of them. The high durability of FRP is valuable in environments that may cause steel corrosion. FRP is gaining popularity in view of its many advantages such as low unit weight, ease of handling and application, and low installation and maintenance costs. Some of the disadvantages of FRP include their linear stress-strain behavior to failure, low strain at failure compared with steel, negligible dowel action at flexural and shear cracks, and relatively high material cost. In certain situations, however, FRP provides the most economical solution to strengthening problems because labor costs can be dramatically reduced (Meier and Erki 1997). ACI Committee 440 has developed guidelines for external strengthening applications such as ACI 440.2R. This chapter is introduction to the subject of repair, and the guide should always be followed for design and application purposes.

FRP can be used to strengthen concrete beams in flexure and shear; slabs in flexure; and columns in compression, shear, and flexure. FRP sheets, plates, or NSM reinforcement can be attached to the tensile face of beams or slabs for flexural strengthening. For shear strengthening of beams, FRP sheets can be bonded to the sides of the beams. For columns, FRP sheets or precured shells can be placed on the exterior of columns to increase confinement, ductility, and strength.

The process for the field installation of externally bonded FRP reinforcement consists of the following basic steps: concrete surface preparation (that is, cleaning, sealing cracks, rustproofing of existing steel reinforcement, and smoothing), application of a primer coat, application of a resin undercoat, adhesion of the sheets, curing, and application of the finish coat.

8.1—Flexural strengthening with non-prestressed FRP

The initial development of the FRP-strengthening technique with prefabricated laminates took place in Switzerland (Meier and Kaiser 1991) and Germany. The initial work on strengthened full-scale reinforced concrete beams (Meier and Kaiser 1991) validated the strain compatibility method in the analysis of cross sections and suggested that inclined cracking (shear cracks) may lead to premature failure by peeling-off of the strengthening laminate at the crack. An analytical model for the composite plate anchoring, which agreed with the test results, was developed (Meier and Kaiser 1991). A comprehensive analytical and experimental study of the short-term flexural behavior of strengthened FRP reinforced concrete beams was carried out by Triantafillou and Plevris (1990 and 1992). They concluded that the flexural behavior of reinforced concrete beams strengthened with FRP laminates can be adequately described by the classical theory of plane-sectional analysis to predict the moment-curvature of the load-deflection response at a specific section when premature peeling or debonding failure of the FRP is avoided. Other researchers have subsequently validated these results (Assih et al. 1997).

8.1.1 *Static behavior in flexure*—Meier and Kaiser (1991) conducted tests on 26 flexural reinforced concrete beams with a 2.4 m (8 ft) span strengthened with CFRP laminates. A typical load-deflection curve is shown in Fig. 8.1, which shows a doubling of strength for the strengthened beam, but with reduced deflections, and hence ductility, at failure. The laminate also caused a more distributed cracking pattern in the strengthened beam with reduced crack widths. Other researchers have subsequently found similar results (Hefferman and Erki 1996; Swamy et al. 1996a,b; Beber et al. 1999; Naaman 1999; Kachlakev and Barnes 1999; Jonaitis et al. 1999).

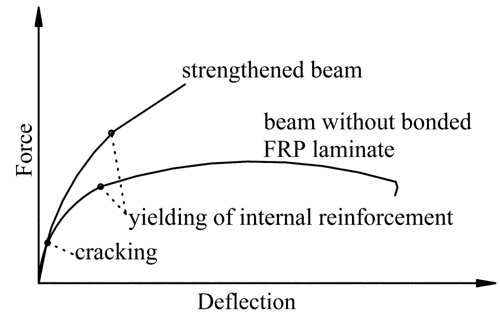


Fig. 8.1—Typical load-deflection curve of strengthened and unstrengthened concrete beams (adapted from Meier and Kaiser [1991]).

Several failure modes controlling the ultimate strength in concrete beams strengthened with FRP laminates have been observed and reported in the literature by many researchers (Meier and Kaiser 1991; Triantafillou and Plevris 1992; Triantafillou 1998a). These studies show that the main failure modes are:

- Crushing of the concrete in compression before yielding of the reinforcing steel;
- Yielding of the steel in tension followed by rupture of the FRP laminate;
- Yielding of the steel in tension followed by concrete crushing;
- Shear/tension delamination of the concrete cover (cover delamination); and
- Debonding of the FRP from the concrete substrate (FRP debonding).

An et al. (1991) performed a parametric study of reinforced concrete beams strengthened with FRP plates. They found that for beams with high internal reinforcement ratios, a stiffer plate in combination with higher-strength concrete is more effective than a plate with a lower stiffness in combination with lower-strength concrete. For beams with the same internal reinforcement ratios and concrete compressive strength, they found that, as the plate strength and stiffness increased, the ultimate moment capacity increased until the failure mode changed to crushing of the concrete in compression. Cha et al. (1999) found similar results.

Triantafillou and Plevris (1992) developed charts to relate the failure mode of strengthened beams to the level of external FRP and internal steel reinforcement (Fig. 8.2). For a given amount of internal steel reinforcement, the failure mode changed from steel yielding to concrete crushing as the amount of FRP was increased. FRP fracture only occurred for beams with low amounts of internal steel reinforcement.

They validated these charts with tests on small-scale beams (1.2 m [4 ft] span) and found that peeling failures placed an upper limit on the amount of FRP that could be effective for strengthening.

Triantafillou and Plevris (1995) developed a reliability-based design procedure for strengthening with CFRP sheets or plates. Two strength-reduction factors were derived to achieve a reliability index of approximately 3 over a broad spectrum of design conditions. From the analysis, they proposed a general strength-reduction factor, $\phi = 0.85$, and a partial reduction factor, $\phi_{fc} = 0.95$, for the fiber composite strength.

White et al. (1998) investigated the effects of loading rates on the behavior of 3.0 m (9.8 ft) span reinforced concrete beams strengthened with CFRP laminates. Their test results showed that service and ultimate flexural capacity increased as the rate of loading increased. Cracking and failure modes were not affected by the rate of loading.

Maalej and Bonacci (1998) tested two 3.7 m (12.1 ft) span reinforced concrete beams with external CFRP sheets to study the effect of strengthening under a loaded condition. The beam strengthened under load was approximately 6% weaker than a similar beam strengthened without an applied load. The difference occurred because the force in the sheet was lower in the beam strengthened under load. This result demonstrated the importance of considering the initial strain condition in the beam when strengthening occurs and was confirmed by the research of Tan and Mathivoli (1999). On the other hand, David et al. (1999) studied the influence of precracking and found that it did not affect the structural capacity but did reduce the stiffness of the strengthened member.

Aboutaha (1999) experimentally investigated the application of CFRP sheets for strengthening a damaged AASHTO Type II prestressed concrete girder (9.4 m [31 ft] span). To simulate the damage, the undamaged girder was loaded to its ultimate flexural capacity, resulting in severe flexural cracks under the loading points and bond deterioration between the steel strands and concrete. The response of the girder in its damaged condition showed a serious deterioration of stiffness, approximately 50% lower than that of the original girder. The stiffness of the repaired girder was approximately 25% lower than that of the original girder. The ultimate strength of the repaired girder was much higher than that of the original girder.

Rasheed and Pervaiz (2003) developed a bond-slip analytical solution for externally strengthened beams with FRP plates using beam theory with a deformable adhesive layer. The solution provided closed-form deflection expressions for beams with long and short FRP plates subjected to three-point bending, four-point bending, or uniform load. The results showed that the FRP tension force cannot be fully developed when the adhesive shear modulus is below 65 MPa/mm (239 ksi/in.) of the adhesive layer thickness regardless of the length of the plate.

Charkas et al. (2003) developed an analytical solution for the deflection calculation of simply supported reinforced concrete beams strengthened with FRP at any load stage. The solution assumes a trilinear moment-curvature response. It incorporates some tension-stiffening effects and assumes the section to be fully cracked only upon or near steel yielding, depending on the concrete nonlinearity in compression. Comparisons with experiments indicate the effectiveness of the procedure for properly anchored plates. An extensive parametric study was conducted to reveal a single linear relationship between the cracked moment of inertia of the section I_{cr} and the overall effective beam moment of inertia at yielding I_{ey} for a wide range of geometric and material parameters.

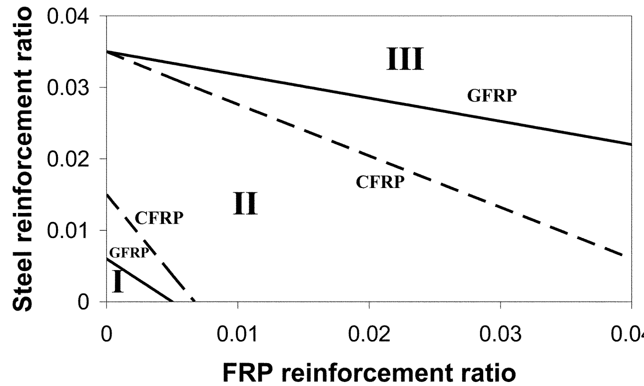


Fig. 8.2—Failure diagram for different composite sheets: CFRP and GFRP. (I = steel yield-FRP fracture [below lower lines]; II = steel yield-concrete crushing [between bounding lines]; and III = compression failure [above upper lines].) (Adapted from Triantafillou and Plevris [1992]).

Other work has been conducted by Kotynia (1999, 2005a) and Kaminska and Kotynia (2000).

8.1.2 *Debonding failures*—Most methods of externally bonding FRP materials to a concrete substrate have exhibited some degree of failure due to debonding. Flexure and shear retrofit measures are typically bond critical, and a number of debonding modes have been observed. Debonding has been characterized in a wide variety of ways in attempts to formulate rational approaches to debonding mitigation. Oehlers (2005) provides a summary of the five generic debonding modes observed in bond critical FRP applications:

8.1.2.1 Axial intermediate crack debonding describes the debonding observed in direct pull tests where the FRP is in tension and there is no curvature present in the concrete substrate. Axial intermediate crack debonding is a fundamental bond property occurring only in laboratory tests.

8.1.2.2 Plate end debonding results from large normal stresses that occur at the curtailment of an FRP plate or sheet. Plate end debonding typically propagates through the cover concrete, often at the level of the internal reinforcement. Plate end debonding may be effectively mitigated through the use of transverse clamping reinforcement (U-stirrups of FRP) (Garden and Hollaway 1998; Garden et al. 1997; Spadea et al. 2001), mechanical anchorage (Shahrooz et al. 2002), or by ensuring that the FRP curtailment extends into the uncracked region of a beam (Roberts 1989). Additionally, using a more flexible adhesive or tapering the FRP plate (width, thickness, or both) can reduce the stress concentration at the end of the plate (Roberts 1989).

8.1.2.3 Intermediate crack-induced debonding initiates at flexure or flexure-shear cracks, or both, located in the shear span of a member, and propagates in the direction of decreasing moment. Such cracks result from the small degree of prying action resulting from the relative distortion present at the toe of a crack in the shear span of a member. Marginal improvement in intermediate crack-induced debonding behavior has been observed when continuous transverse clamping reinforcement is present (Kotynia 2005b). Intermediate crack-induced debonding has been observed to initiate at multiple cracks over the same shear span (Harries et al. 2006).

8.1.2.4 Critical diagonal crack-induced debonding is similar to intermediate crack-induced debonding but is associated with a single critical shear crack in the member. Mitigation of critical diagonal crack-induced debonding is affected by controlling the formation of the shear crack itself.

8.1.2.5 Interfacial shear-induced debonding is associated with the elementary interface shear between stress concentrations (for example, between cracks in a constant moment region). Such debonding is rarely observed in conventional bonded applications but may control the behavior of FRP bonded to prestressed members where cracking is suppressed or in cases where large FRP thicknesses are used to control deflections at service load (and thus, largely uncracked) conditions (Oehlers 2005).

8.1.3 *Plate and sheet anchors*—Examination of the various premature failure modes in concrete leads to the conclusion that premature failure usually initiates at the end of the plate or sheet. This is due to the geometric singularity created by termination of the plate, and it implies that proper detailing of the plate end and additional plate anchorage techniques should be used.

Several approaches have been investigated to avoid peeling failures. Some of these include mechanical anchorages at the ends of the sheet, wrapped sheets around the web of the beam over the longitudinal FRP sheet, or changes in the geometry of the sheets in the anchorage zones as suggested by Karam (1992).

Swamy et al. (1987) showed that premature debonding of steel plates can effectively be avoided by ensuring that the width-thickness ratio of the plate is not less than 50. Swamy and Mukopadhyaya (1995) have shown that this recommendation holds true for FRP plates when glass, glass-carbon, and aramid fibers are used. In the case of CFRP, the plates are generally so thin that the criterion is automatically satisfied. Another technique can be used if multiple layers are being prestressed. The layers can then be terminated (and the prestress transferred) at different locations along the beam. This technique is effective at reducing the magnitude of shear and normal stress concentrations that occur at the ends of the plate (Wight 1998).

By using bonded angle plates or transverse FRP wraps, longitudinal FRP strips can be effectively anchored to the tensile face of the beam. The effectiveness of this technique has been commented on by Arduini et al. (1995) who found that a great deal of ductility could be observed until rupture of the plate or shear failure of the wrap or angle plates.

Deblois et al. (1992) compared bonded unidirectional and bidirectional GFRP sheets with bolted unidirectional GFRP sheets using 1.0 and 4.1 m (3.3 and 13.5 ft) specimens. They found that bolted sheets or bidirectional sheets were more effective than unbolted unidirectional sheets.

Sharif et al. (1994) studied three different anchorage schemes for GFRP plates using small-scale beams: 1) bolting the plates to the tension face; 2) bolting combined with FRP plates bonded to the sides of the beams; and 3) a special one-piece I-jacket plate that was glued to the bottom along the whole span and to the sides of the beam in the shear span. The I-jacket was the most effective anchorage scheme because it prevented all types of peeling failures. Bolts were not very effective because the beams failed by shear peeling.

Lamothe et al. (1998), Spadea et al. (1998), Grace et al. (1999), and Van Gemert (1999) also found that wrapping the sides of the beams with vertical FRP sheets provided effective anchorage for the flexural sheets. Quantrill et al. (1996) found that GFRP angles attached to the sides of the beams were effective anchors. On the other hand, Naaman (1999) tested 3.0 m (9.8 ft) span T-beams and did not find any improvement in the strength of the beams when U-shaped anchors were used.

Garden et al. (1997) tested concrete beams strengthened with CFRP plates to study the effects of three parameters: plate aspect ratio (b/t plate width divided by plate thickness) at constant cross-sectional area, shear span-depth ratio of beams (a/h), and the form of plate end anchorage. The plates were anchored by extending them under the supports or attaching them to GFRP angles bonded to the sides of the beams. The ultimate capacities of the strengthened beams decreased with reducing plate width-thickness ratios. Failure was always accompanied by concrete cover separation from internal reinforcement. Increasing the shear span-depth ratios resulted in improved ultimate capacities. Anchoring the sheets increased the strength of the beams.

Teng et al. (1999) conducted an experimental study into strengthening deficient cantilever concrete slabs by bonding GFRP strips on the top surface. Different anchorage systems were used, and the most effective method was to anchor the GFRP strips into the walls through horizontal slots and into the slab with fiber anchors. This method allowed the full strength of the strips to be developed, and the strength was almost four times that of the unstrengthened beam.

8.1.4 *Flexural strengthening using inorganic matrix*—An inorganic matrix is a low-viscosity resin prepared by blending an aluminosilicate powder with a water-based activator. It is suitable for penetrating carbon or glass fiber sheets and fabrics (Kurtz and Balaguru 2001; Foden et al. 1996; Toutanji et al. 2001). One disadvantage of organic matrix composites is their lack of fire resistance (Chapter 12). Inorganic matrixes have a number of advantages such as high resistance to fire and high temperature, resistance to UV radiation, and easy handling because the inorganic resin is water-based and emits no odor and toxins during construction or curing. Temperature exposure tests show that, by using an inorganic matrix, only 37% of the composite's initial flexural strength is lost after 1 hour of exposure at 800 °C (1472 °F) (Foden et al. 1996).

Kurtz and Balaguru (2001) compared the performance of an inorganic matrix with that of an organic one when used to externally strengthen reinforced concrete beams with carbon fiber sheets. Strength, stiffness, ductility, failure pattern, and cracking of beams strengthened with the two systems (organic and inorganic) were compared. The results showed that the inorganic matrix was as effective in increasing the strength and stiffness of the reinforced concrete beams as the organic matrix, with a minor reduction in ductility. The failure mechanism changed from sheet delamination for the organic system to sheet rupture for the inorganic system.

Toutanji et al. (2001) confirmed these results with further tests and studied the fatigue behavior (Toutanji et al. 2006). Large-scale reinforced concrete beams were strengthened with three layers of carbon fiber sheets and tested under fatigue. The relationships between fatigue strength, crack width, and the number of cycles were studied and analyzed. Results showed that both load capacity and the number of fatigue cycles of reinforced concrete beams were significantly increased with carbon fiber sheets.

8.1.5 *Slabs*—Arockiasamy et al. (1995) conducted a test on the flexural capacity of a structurally damaged solid slab bridge model strengthened with bonded thin CFRP laminates. The unstrengthened slab was tested to failure and then retrofitted with CFRP. The retrofitted slab showed a 90% recovery in the ultimate moment, but with larger deflections.

Malvar et al. (1996) tested six 1/5-scale under-reinforced concrete slabs. Both one-way and two-way strengthening schemes were evaluated. Their test results showed that slabs with one-way CFRP sheets exhibited an increase in the ultimate load of up to 32%, and a decrease in the ultimate deflection of up to 42%. On the other hand, the slabs with two-way CFRP sheets exhibited an increase in ultimate load of 47% and a decrease in ultimate deflection of 27% when compared with the control specimen. Half-scale slabs strengthened with three longitudinal and one transverse CFRP layer exhibited a 20% increase in the ultimate load. The typical failure mode for all the specimens was punching shear.

Ospina et al. (2001) examined the punching shear behavior of damaged two-way slabs with FRP sheets. They tested two full-scale 4.2 m (13.8 ft) square slabs under concentric uniform gravity loading. The slabs were strengthened with 250 mm (10 in.) wide CFRP strips bonded to the top surface in a cruciform pattern. Results showed that the bonded FRP strips tended to stiffen the slab, but no punching shear strength increase was obtained compared with the results of a control virgin slab specimen.

8.2—Flexural strengthening with prestressed FRP

Another method for FRP flexural strengthening is achieved using prestressed FRP laminates. This specialized application combines the benefits of passive bonded FRP laminate systems with the advantages associated with external prestressing. By applying a prestress to the laminate, the material may be used more efficiently because a greater portion of its tensile capacity is engaged. Prestressed FRP

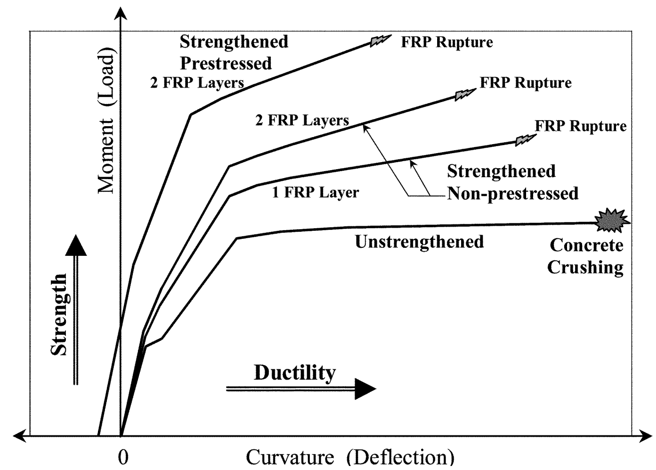


Fig. 8.3—Comparison of prestressed and non-prestressed laminates (El-Hacha 2000).

laminates, when applied to the tension face of a concrete beam, can improve the serviceability of a beam and provide control for cracks by closing existing cracks completely and delaying the formation of new ones. The benefit of prestressed FRP laminates in delaying the cracking and closing of existing cracks may help to restore durability of the structure because the cracks provide additional access for moisture into the concrete and the adhesive layer.

Anchoring the ends of prestressed FRP laminates may further increase the ultimate strength by preventing premature failure modes (Meier 1995; Wight et al. 2001). Prestressed CFRP laminates are likely to show fewer long-term losses than external unbonded tendons (Hollaway and Leeming 1999). Prestress levels of at least 25% of the laminate strength may be necessary to achieve a significant improvement in terms of structural stiffness and load-carrying capacity of the concrete (Garden et al. 1998). Meier et al. (1992) suggested that a prestress level as high as 50% of the plate strength might be necessary to increase ultimate strength by delaying the peeling failure of the FRP plate.

Highly tensioned prestressed laminates will tend to have high shear stresses where the laminates terminate, and this may cause premature shear peeling failure (Triantafillou and Deskovic 1991). To prevent this failure, the shear strength of the concrete may be increased by the application of transverse reinforcement or anchors (Wight et al. 2001; Lees et al. 1999).

Typical load-deflection curves for a reinforced concrete beam, and similar concrete beams strengthened with non-prestressed and prestressed FRP laminates, are shown in Fig. 8.3 (El-Hacha 2000). As shown in the figure, the prestressed laminates are able to increase both the strength and stiffness of the beam. Several systems have been developed to induce a prestress in the laminate for flexural strengthening, and these generally fall into three categories: cambered beam systems, systems that tension the FRP laminate against an independent external reaction frame, and systems that tension the FRP laminate against the strengthened beam itself. The literature concerning this technology is very limited when compared with that for conventional non-prestressed applications. El-Hacha et al. (2001) recently summarized much of the pertinent literature.

A new concept for prestressing the CFRP laminates was outlined recently by Stoecklin and Meier (2003). Once the FRP is prestressed, the central portion is cured and bonded to the beam using heating elements. The prestress level is then reduced progressively as the curing moves towards the ends of the laminates. Using this method, the prestress profile can be designed for the specific beam with the maximum prestress in the middle and the lower prestress towards the ends of the laminate. The shear stresses in the concrete cover at the end of the laminates are also reduced, and anchors are not required to transfer the prestress at release of jacking.

The fatigue of beams strengthened by prestressed FRP laminates has been investigated by Wight and Erki (2001) and Ford (2004).

8.3—Shear strengthening

Javor et al. (1999) conducted tests on 1.6 m (5.2 ft) span reinforced concrete beams that were deficient in shear. The beams were strengthened with CFRP plates bonded on the sides of the beams. Two arrangements were considered: vertical plates and plates at a 45-degree angle. Both arrangements increased the shear capacity of the beams by approximately 40%.

Naaman (1999) considered shear strengthening with either plates or sheets bonded vertically to the sides of rectangular beams (1.3 m [4.3 ft] span) and T-beams (1.9 m [6.2 ft] span). Both the plates and the sheets were found to increase the shear strength by at least 30%. The sheets were found to be more effective than the plates because they could be anchored around the bottom of the beams.

Variations in the number of layers, as well as the fiber orientation and layout, were studied (Lamothe et al. 1998; Naaman 1999; Triantafillou 1998a; Javor et al. 1999; Kachlakev and Barnes 1999; Hutchinson and Rizkalla 1999; Fanning and Kelly 1999; Khalifa et al. 2000; Triantafillou and Antonopoulos 2000; Deniaud and Cheng 2000; Lombard et al. 2000). It was observed that the shear strength increased with increasing number of laminate layers. The maximum shear strength was obtained for beams that were wrapped over their full depth by sheets having fibers perpendicular to the beam's longitudinal axis.

8.4—Axial strengthening of columns

Wrapping concrete columns with FRP provides confinement to the specimen. Confinement is highly dependent upon the cross-sectional shape and aspect ratio of the column (Prota et al. 2006b; Mukherjee et al. 2004), and is most effective for circular sections (Demers and Neale 1994). Transverse strain in the column section caused by longitudinal cracking and section dilation results in hoop strains in the FRP that produce confining pressure. This increase in confining pressure can cause an increase in axial strength and ductility. Concrete confined with fiber composite materials behaves differently than concrete confined with steel. Harmon et al. (1995) found that FRP-wrapped composite concrete failed at higher axial and radial strains than steel-wrapped concrete. Another difference in behavior between steel- and fiber-wrapped specimens is the ductility of the failure. Steel, in general, results in a ductile failure with adequate warning, whereas fibers, such as carbon, have violent explosive failures with little to no warning.

Demers and Neale (1994) studied small-scale (circular and square) unreinforced specimens, wrapped with carbon or glass fiber unidirectional sheets. They concluded that FRPs produced an increase in strength of up to 70% over unwrapped specimens and improved ductility. Demers and Neale (1994) also examined a few confinement models and concluded that they tend to overestimate the specimen capacity.

Howie and Karbhari (1995) and Karbhari and Howie (1997) investigated the effects of various composite wrap architectures on the confining effects and overall performance enhancement of concrete cylinders. They altered the wrap architecture by changing the orientation of the entire composite layer. The wraps varied in configuration from along the longitudinal axis, 45 degrees off the axis, and in the hoop direction. They found that the failure strength increased with the percentage of composite reinforcement in the hoop direction.

Toutanji (1999) investigated the performance of concrete columns confined with FRP composite sheets. Concrete columns were wrapped with three different types of FRP composites. Axial load and axial lateral strains were obtained to evaluate stress-strain behavior, ultimate strength, stiffness, and ductility. The results showed that both the strength and ductility of tested specimens were significantly enhanced over unwrapped specimens. In addition, an analytical model was developed to predict the entire stress-strain relationship of the wrapped specimens. Results from a series of experimental tests on concrete confined with FRP sheets compared favorably with the results obtained by the proposed model.

Confinement effectiveness of FRP jackets in concrete columns was studied by a number of researchers (Mirmiran and Shahawy 1996; Saafi et al. 1999). The improvement in mechanical properties depends on several parameters, including concrete strength, types of fibers and resin, fiber volume fraction and fiber orientation in the jacket, jacket thickness, shape of cross section, length-diameter ratio of the column, and the interface bond between the core and the jacket.

Spoelstra and Monti (1999) developed a uniaxial model for concrete confined with FRP. The model explicitly accounted for the continuous interaction with the FRP wrap due to the lateral strain of concrete through an incremental-iterative approach. The relation between the axial and lateral strains was implicitly derived through equilibrium between the dilating confined concrete and the wrap. The model was compared with a set of experimental tests, and showed very good agreement in both the axial stress-strain and the stress-lateral strain response.

Seible et al. (1999a) conducted a large-scale test on one as-built and four composite-wrapped rectangular flexural bridge spandrel columns to assess the effectiveness of different retrofit schemes using FRP composite jackets. The tests showed that FRP composite jacketing systems clearly can be installed without affecting the overall geometry or appearance of the structure. The authors emphasized the importance of designing retrofit strategies to control the mode of failure. Retrofitting one weakness without considering other potential modes of failure could lead to ineffective and poor designs (Seible et al. 1999a).

Monti et al. (2001) proposed a design equation to determine the optimal thickness of FRP jackets and to enhance the ductility of existing reinforced concrete bridge piers with circular cross sections. The design procedure stems from the definition of an upgrading index given as the ratio of the target to the available ductility at the pier base section attained through FRP jacketing; it results in a simple expression in terms of easily computable quantities, such as the ultimate strain and the peak strength of concrete before and after upgrading. The index, despite its simplicity, yields excellent predictions of the ductility increase obtained through FRP wrapping, and is used to develop a design equation. The equation allows the design of the optimal thickness of FRP jackets in terms of the desired upgrading index, mechanical characteristics of the selected composite material, and quantities defining the initial state of the pier section.

The experimental and analytical behavior of axially loaded large-scale columns confined with FRP wrapping reinforcement was investigated by Matthys et al. (2005, 2006). Different wrapping orientations and different types of FRP reinforcement, consisting of CFRP sheets, GFRP fabrics, and hybrid FRP (HFRP) fabric were used to confine the concrete columns. The effective circumferential FRP failure strain and the effect of an increasing confining action on the stress-strain behavior of reinforced concrete columns confined with FRP were examined. Different models for the prediction of the stress-strain behavior of FRP-confined concrete were reviewed. The results showed that few of the available models that were derived based on small cylinders could accurately predict the ultimate strength and the stress-strain response of large-scale columns.

8.5—Seismic strengthening and retrofitting

The various applications of external FRP composite retrofit in buildings as well as bridges have shown that, in most cases, the implementation of composite materials provides adequate seismic detailing to the structure, and improves ductility and seismic performance. This provides an economic alternative to rebuilding. Seismic rehabilitation is not covered in 440.2R, but is planned to be for future revisions.

The flexural behavior of earthquake-damaged reinforced concrete columns repaired with FRP wraps has been studied by Saadatmanesh et al. (1997a). The columns were repaired with prefabricated FRP wraps and retested under simulated earthquake loading. Seismic performance of repaired columns in terms of their hysteretic response was evaluated and compared with those of the original and unretrofitted columns. The results indicate that the proposed repair technique was highly effective. Both flexural strength and displacement ductility of the repaired columns were higher than those of the original columns.

The behavior of rectangular bridge columns with substandard design details for seismic forces was investigated by Saadatmanesh et al. (1997b). High-strength FRP straps were wrapped around the column in the potential plastic hinge region to increase confinement and to improve the behavior under seismic forces. Results indicated that significant improvement in ductility and energy-absorption capacity could be achieved due to this retrofitting technique.

8.5.1 *Seismic retrofit design*—In the aftermath of the 1995 Hyogo-Ken Nanbu earthquake in which many failures of bridges and piers occurred, numerous studies were conducted on ways to retrofit existing reinforced concrete columns and piers. Mutsuyoshi et al. (1999) found that continuous fiber sheets offered a feasible means of retrofitting. Consequently, several design guidelines on the use of FRP sheets for retrofitting highway, railway, and subway structures have been proposed in recent years. The JSCE Concrete Committee on FRP Sheets has been commissioned to establish a new design method for seismic retrofitting of bridge columns and piers. It seeks to unify all the existing guidelines on a performance-based design concept. The new design method for shear strengthening and ductility enhancement of retrofitted reinforced concrete structures using FRP sheets is described in the study.

Seible et al. (1997a,b) and Seible (2001) described jacket design criteria for various seismic column failure modes and provided detailed examples of their application to retrofit columns with circular and rectangular geometry, different reinforcement ratios, and detailing. The carbon jacket designs were validated through large-scale bridge column model tests and were found to be just as effective as steel shell jacketing in providing desired inelastic design deformation capacity levels.

Sheikh (2001) tested columns retrofitted with CFRP and GFRP composites to improve the seismic resistance of concrete columns. The results showed that retrofitting with FRP for both circular and square columns could improve their brittle behavior, thus significantly improving their seismic resistance.

Pantelides and Gergely (2002) presented analysis and design procedures for the CFRP composite seismic retrofit of a reinforced concrete three-column bridge bent. The seismic retrofit was successful, and the bridge bent retrofitted with CFRP composites reached a displacement ductility level in excess of the target ductility and double the hysteretic energy dissipation of the as-built bent. Descriptions of the CFRP composite layout and validation of the design assumptions from the experimental results were presented. Recommendations for improvement of the original CFRP composite seismic retrofit design were offered based on the lessons learned from the in-place tests.

Saatcioglu and Grira (2001) carried out an experimental investigation to verify the use of FRP grids as transverse reinforcement in concrete structures, placing emphasis on concrete confinement and seismic performance. Large-scale column specimens were tested under simulated seismic loading. Test parameters included grid spacing and pattern, the volumetric ratio of grid reinforcement, and the level of

Fig. 8.4—Attaching FRP strips with power-actuated fasteners (Bank et al. 2003).

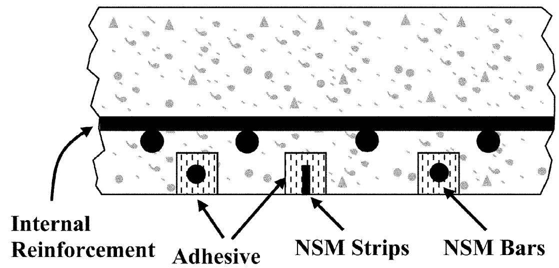


Fig. 8.5—Cross section of a slab strengthened with NSM FRP reinforcement.

axial load. Results showed improved deformability of columns when confined by properly designed grids.

Sause et al. (2004) proposed a design procedure for seismic retrofit of columns using FRP jackets that was based on curvature capacity and concrete strain demand. The design recommendations were verified with experimental data of four full-scale square columns tested under reverse cyclic loading. One column was tested without retrofitting, while the remaining three were retrofitted with various plies of carbon jackets. The authors concluded that a simple fiber model approach, accounting for regions of confined and unconfined concrete, appropriately estimated a conservative extreme concrete fiber compression strain required to achieve a desired deformation capacity. Further, adequate structural response of building columns may be achieved using lower levels of confinement than those previously presented for highway bridge pier columns with larger cross sections.

8.6—Mechanically fastened fiber-reinforced polymer (MF-FRP) laminates

A method for strengthening reinforced concrete members using pultruded FRP laminates and mechanical fasteners has been developed (Lamanna et al. 2001, 2004; Bank et al. 2003; Rizzo et al. 2004). The strengthening is obtained by attaching pultruded FRP laminates with high bearing and longitudinal strengths to the concrete using several, closely spaced steel power-actuated fastening pins or steel expansion anchors (Fig. 8.4). The MF-FRP method is rapid and uses conventional, typically available hand-tools and lightweight materials. In addition, unlike the bonded method, the MF-FRP method requires minimal surface preparation and permits immediate use of the strengthened structure. Previous research on this method on a variety of beam sizes has shown promising results in terms of installation efficiency, the level of strengthening achieved, and the prevention of laminate delamination before concrete crushing.

8.7—Strengthening using near-surface-mounted FRP reinforcement

Near-surface-mounted (NSM) FRP bars and laminates are being used as an alternative technology to externally bonded FRP laminates. Figure 8.5 shows a cross section of a slab strengthened with NSM. The NSM reinforcement technique involves cutting a groove in the surface of the member, roughening and cleaning the groove, filling the groove halfway with a structural adhesive, installing the FRP reinforcing bar or laminate, filling the groove completely with structural adhesive, and leveling the surface. The NSM FRP technique may be more suitable for cases in which the concrete surface is very rough, weak, or requires significant surface preparation. NSM systems can also be used for shear and flexural strengthening of unreinforced masonry walls (Tumialan et al. 2003a,b,c).

8.7.1 *Flexural strengthening*—Alkhrdaji (2001) strengthened a reinforced concrete bridge deck span with NSM CFRP bars that was tested in place to failure. Compared with an unstrengthened span, they achieved approximately a 30% increase in flexural capacity. Failure was due to the rupture of CFRP bars.

Nordin et al. (2001) conducted a pilot study on concrete beams strengthened in flexure with prestressed NSM CFRP strips. Test results showed a substantial increase in cracking and failure loads for the strengthened specimens. Prestressing the strips did not influence the mode of failure. Compared with unstrengthened specimens, the prestressed beams had considerably smaller deflections at failure.

De Lorenzis et al. (2000) presented the results of tests on large T-beams (with steel reinforcement ratio equal to approximately 25% of the balanced amount) and rectangular beams (with two steel reinforcement ratios, equal to approximately 35 and 59% of the balanced amount). Debonding of the FRP bars was the dominant failure mode for the T-beams and the rectangular beams with the lower reinforcement ratio. The rectangular beams with the higher reinforcement ratio failed by concrete crushing. The authors proposed a system for post-tensioning of the NSM system in cases where access to the ends of the beam was not feasible.

Hassan and Rizkalla (2002) investigated the feasibility of using different techniques for strengthening prestressed concrete bridge decks in flexure. Large-scale models of a prestressed concrete bridge deck were strengthened with

FRP systems and tested to failure. Test results showed that the efficiency of NSM CFRP strips was three times greater than that of the externally bonded strips.

Rosenboom et al. (2004) strengthened 12 prestressed concrete girders with various CFRP systems and tested them under static and fatigue loading. The NSM CFRP bars and strips strengthened girders and achieved a 20% increase in ultimate flexural capacity compared with the control girder when monotonically loaded to failure. The NSM-strengthened girders also performed well under fatigue loading conditions, surviving over 2 million cycles of increased service loading with little degradation and reduced crack widths.

8.7.2 *Shear strengthening*—De Lorenzis and Nanni (2001a,b) investigated the structural performance of simply supported reinforced concrete beams strengthened in shear with NSM CFRP rods. Test results showed NSM FRP rods were effective at enhancing the shear capacity of reinforced concrete beams. The beams strengthened in shear showed an increase in capacity from 28 to 41% over the control specimen.

Nanni et al. (2004) applied NSM CFRP shear strengthening as well as externally bonded precured CFRP laminate flexural strengthening to a prestressed concrete girder taken off an overloaded bridge. The girder was tested in four-point bending, and the use of the rectangular NSM CFRP bars increased the shear capacity of the prestressed girder by at least 53%.

8.7.3 *Development length and bond*—Hassan and Rizkalla (2003, 2004) studied the bond characteristics of NSM CFRP bars and strips. The influence of the groove dimensions, groove spacing, and the limited adhesive cover was investigated. They concluded that the tensile stresses at the concrete-adhesive interface as well as at the FRP-adhesive interface were highly dependent on the groove dimensions and controlled the mode of failure of NSM FRP bars and strips. They recommended widening of the groove to minimize the induced tensile stresses at the concrete-epoxy interface and increase the debonding loads of NSM bars. A mathematical model to find the development length of NSM FRP bars was presented along with design charts correlated to experimental results.

De Lorenzis and Nanni (2002) and De Lorenzis et al. (2004) investigated the bond characteristics between NSM FRP rods and concrete. The influence of several parameters, such as bond length, diameter and surface configurations of the rods, surface condition of the groove, groove size, and type of groove-filling material, was examined. These tests showed that bars with some surface deformation should be used to avoid interfacial failure between the bar and the epoxy, and that the degree of groove roughness as normally obtained by saw-cutting was able to prevent pure interfacial failure at the epoxy-to-concrete interface. Debonding was then controlled by failure within the epoxy or the concrete, or a combination of the two. The effects of bar surface deformation and a groove size-to-bar diameter ratio on failure mode and bond strength were analyzed. A groove size-to-bar diameter ratio equal to 2.0 was recommended. A less satisfactory performance of cement paste as a groove-filling material was also reported. De Lorenzis (2004) proposed equations to describe the local bond-slip curves of different NSM systems and analytical curves giving the bond failure load as a function of the bond length, outlining a design methodology for anchorage. A simplified analytical model of splitting bond failure and a three-dimensional finite element model of the NSM joint were also reported.

Sena Cruz and Barros (2004) analyzed the influence of bond length and concrete strength on NSM CFRP reinforcement in reinforced concrete by conducting pullout tests. A local bond stress-slip relationship was calibrated with the experimental results. They concluded that the concrete strength had marginal influence on the pullout behavior and that various parameters defining the local bond stress-slip relationship were dependent on the slip at peak bond stress.

Other work has been conducted by Alkhrdaji et al. (1999), De Lorenzis et al. (2000), Arduini et al. (2001), Carolin et al. (2001), Täljsten and Carolin (2001), El-Hacha and Rizkalla (2004), and Kotynia (2005b).

8.8—Design procedures

Interest in developing flexural design procedures for FRP-strengthened sections was evident in the mid-1990s. Picard et al. (1995) developed analysis equations to compute the ultimate flexural capacity of FRP-strengthened beam sections using ACI or CSA-type formulations. Chaallal et al. (1998) and Saadatmanesh and Malek (1998) independently presented comprehensive design approaches for reinforced concrete flexural members strengthened with externally bonded FRP. Iterations were required to determine the FRP area corresponding to a certain strengthening level. Rasheed and Pervaiz (2003) formulated closed form design equations allowing for a direct procedure without iterations.

Efforts to develop shear strengthening design models appeared in early 1990s. Triantafillou (1998a) reviewed the earlier models, concluding that they yield contradictory results in most cases. He extended the design model of the Eurocode 2 to account for the contribution of FRP shear reinforcement. Khalifa et al. (1998) modified Triantafillou's equation to describe combined shear failure and FRP rupture. Triantafillou and Antonopoulos (2000) further refined these models by considering three limit states of effective FRP strains.

CHAPTER 9—STRUCTURALLY INTEGRATED STAY-IN-PLACE FRP FORMS

9.1—Introduction

The construction industry is expressing great demand for innovative and durable structural members such as bridge decks and piers, piling, utility poles, and highway overhead sign structures. FRPs have been widely used for structural retrofit in the form of bars and tendons for concrete structures as replacements for steel reinforcement. FRP for structurally integrated stay-in-place (SIP) formwork maximizes the advantages of both FRP and concrete, while simplifying the construction procedure and reducing construction time. In the last several years, a number of such systems have been developed. One of the first published studies on SIP forms proposed a rectangular FRP box section with one open side to be filled with concrete for beam applications as shown in Fig. 9.1(a) (Fardis and Khalili 1981). Deskovic et al. (1995)

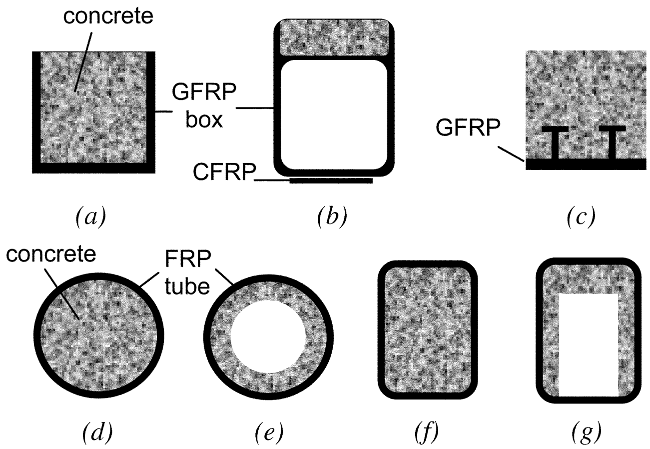


Fig. 9.1—Examples of different configurations of SIP FRP form systems for structural concrete members (Fam 2000).

proposed a novel beam system that consisted of a GFRP box beam with a CFRP laminate bonded to its tension side to take the tension forces and a concrete layer cast on its compression side as shown in Fig. 9.1(b). Hall and Mottram (1998) proposed a novel slab system that consisted of a concrete slab cast onto a pultruded FRP panel with a flat continuous base and two T-up stands as shear studs. The FRP panels serve as permanent forms and reinforcement for the slab as shown in Fig. 9.1(c). Concrete-filled FRP tubes (CFFT) are another promising system, shown in Fig. 9.1(d), that could be used as piles in corrosive marine environments and as bridge girders, piers, and columns (Mirmiran and Shahawy 1996; Fam and Rizkalla 2001a; Seible 1996). The tubes could also be rectangular, as shown in Fig. 9.1(f) and (g) for beam applications (Fam et al. 2003a; Mirmiran et al. 1999). Furthermore, in flexural applications, providing a central hole inside the tube as shown in Fig. 9.1(e) and (g) could substantially reduce the self-weight of the member (Fam and Rizkalla 2002).

9.2—Advantages and limitations of system

The most common characteristics of the (SIP) FRP form systems are as follows:

1. The FRP shape acts as a permanent form for the concrete; hence, it will save the cost of formwork involved in conventional cast-in-place or precast industries;

2. Main reinforcement for concrete is provided externally by the FRP shape, even though additional reinforcement of other materials, such as steel or FRP or both, may be provided internally. In CFFT, the tube provides both shear and flexural reinforcement using the multidirectional fiber orientation. This saves the time and cost of assembling longitudinal bars together with stirrups in conventional construction. The closed composite tubes provide passive confinement to the concrete, which significantly improves the strength and ductility;

3. Depending on the nature of loading, the capacity and performance of the system may depend on the composite action between the concrete and FRP shape. For example, under axial loads, it is desired to unbond the FRP tube from the concrete core, while under bending, it is essential to provide full composite action;

4. The SIP FRP form system lends itself to optimization based on material properties of each component. A hybrid system provides the designers with several flexible parameters that can be controlled to achieve optimum design in individual applications including type of fibers, orientation of the fibers, number of layers in the composite shell, and the composite shell wall thickness; and

5. The confined concrete, in the case of CFFT, is protected from intrusion of moisture with corrosive agents that could otherwise deteriorate the concrete core.

It is also important to note the limitations of the system. Fire is a concern in some structural applications, and measures should be taken to provide fire protection. Chapter 12 discusses this issue in more detail. Vandalism is also a concern; thus, measures should be taken either to limit access to the FRP composite or to provide a paint or texture similar to concrete. Finally, when the FRP form is in direct contact with water, it should be protected against moisture intrusion through selection of an appropriate resin. Such durability issues are discussed in Chapter 11.

9.3—Structural composition of FRP forms

The FRP component of the SIP system accommodates the concrete, acts as reinforcement in one or more directions, and, in some cases, provides confinement for the concrete at the same time. The FRP form is typically a thin shell (laminate) that could be a closed form, such as circular and rectangular tubes, or open forms. The FRP laminate typically includes a number of layers, each consisting of unidirectional fibers or woven fabric in a polymeric matrix. Glass fibers are the most economical and commonly used fibers in such applications. The direction of the fibers in each layer θ with respect to a global longitudinal axis (usually defined along the member length) could vary to provide the required strength and stiffness in more than one direction. The in-plane laminate behavior can be treated as an orthotropic membrane, mainly subjected to in-plane normal and shear forces. The configuration of the laminate, indicating its layer ply composition and sequence of various plies, is referred to as stacking sequence. A laminate is considered symmetric when, for each layer on one side of the middle surface, there is a corresponding layer at an equal distance from that reference plane on the other side with identical thickness, orientation, and properties. A laminate is considered balanced when it consists of pairs of layers of identical thickness and elastic properties, but with similar $+\theta$ and $-\theta$ orientations. Given the structure of the laminate in terms of number of layers, fiber orientation in each layer, thickness of individual layers, fiber and resin types, fiber volume fraction, and mechanical properties of a single unidirectional layer, classical lamination theory is typically used to analyze FRP laminates (Daniel and Ishai 1994). The theory provides equivalent mechanical properties of the laminate in the major directions and allows the prediction of strength under different loading conditions. Mechanical properties of a single layer can be obtained experimentally using coupon tests following ASTM specifications (ASTM 1990).

9.4—Fabrication processes of FRP structural forms

The most commonly used fabrication processes of FRP forms are filament winding, pultrusion, centrifugal casting, hand lay-up, and vacuum infusion processing (resin infusion). Filament winding and centrifugal casting are generally used to produce hollow closed FRP forms, such as tubes. Details on these processes are provided in Section 4.8 of this report.

9.5—Concrete component

Concrete may be plain, reinforced, or prestressed. The internal reinforcement could be steel or FRP and either material could be prestressed. Hybrid combinations are also possible. While any conventional concrete mixture may be used, it is possible to use a shrinkage-compensating admixture to improve bond with FRP in confined sections. On the other hand, recent studies have found shrinkage of concrete cores in CFFT to be quite negligible because the core is essentially sealed from the drying environment (Naguib and Mirmiran 2001). Additional measures, such as shear connectors or sand-coating the FRP surface, may be taken to improve bond between FRP and concrete.

9.6—Construction considerations

9.6.1 *Concreting*—FRP tubes may be filled with concrete in a vertical, inclined, or horizontal position. Care should be taken to support the weight of fresh concrete until it hardens to avoid sagging. Concrete may be pumped or poured in place. Some fabricators prefer to pressure-pump the concrete fill uphill against gravity as a means of eliminating air voids. Concrete may be consolidated using electric vibrators that may be hand-held or mounted on the FRP form. Plugs or caps may be necessary to contain the wet concrete at the ends. Figure 9.2 shows a CFFT cast in an inclined position (Fam et al. 2003c).

9.6.2 *Bond between concrete and FRP*—Composite action and load transfer between the FRP composite and concrete depend on their interface bond. Push-off tests have been used to obtain the average interfacial bond strength in concrete-filled FRP tubes (Helmi et al. 2005). The following methods have been proposed to achieve the required bond strength:

1. Mechanical interlock can be provided by forming circumferential ridges or a grid of pockets on the inside of the FRP form. Alternatively, extensions of the FRP shell into the concrete could be provided. Mechanical interlock can also be achieved by coating the FRP surface with adhesive, followed by applying a thin layer of coarse silica sand;

2. Chemical bond between concrete and FRP may be achieved using special concrete adhesives. The drawbacks of this method include the cost and difficulty of application; and

3. Shrinkage-reducing admixtures can also be used in concrete mixtures to keep the concrete engaged with the FRP form in tubular sections.

9.6.3 *Protective coating of FRP*—A durable finish may be required for FRP forms exposed to UV rays or to maintain color and gloss (Lancaster Composite, Inc. 2006). The success and longevity of the coating depend on the coating material itself and on the quality of the surface preparation. Gentle sanding or blasting with sand or metal may be sufficient

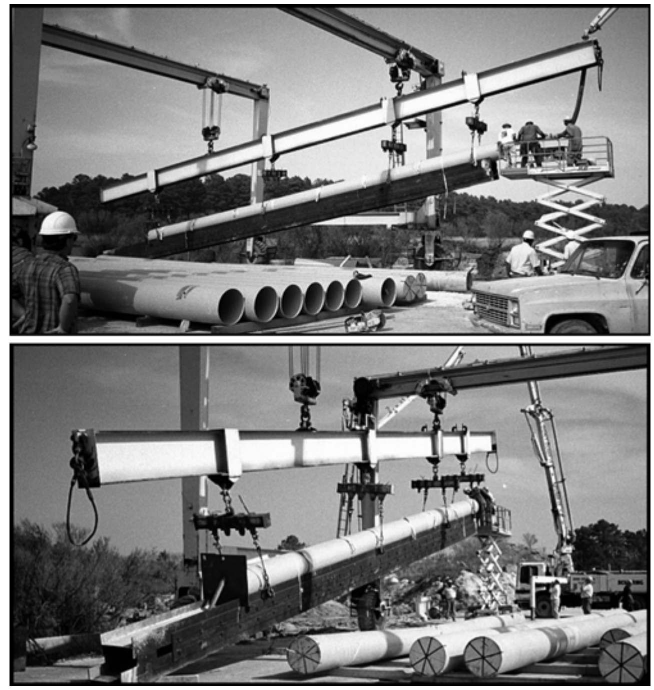


Fig. 9.2—Filling FRP tubes with concrete at an inclined position (Fam et al. 2003c).

to roughen the surface and promote mechanical lock with the coating material. Care should be taken, however, not to penetrate the resin-rich surface and expose the fibers. The service environment often determines the most appropriate coating material for use. Sufficient curing time should be allowed before shipping and handling. Special care should be taken not to scratch the finished surface during handling and transportation.

9.7—Behavior of axial members

The most common forms of axial members are circular and rectangular CFFT (Fig. 9.1(d), (e), and (f)). The FRP tube should be able to withstand the lateral pressure of fresh concrete. The tube should also have adequate stiffness and strength in the circumferential direction to provide confinement for the concrete core under axial loads. The following sections are focused primarily on axial members with circular FRP tubes.

9.7.1 *Background*—Attempts to confine concrete with nonmetallic shells started in the late 1970s, using PVC pipes (Kurt 1978) with limited success. Fardis and Khalili (1981) used GFRP jackets to confine concrete cylinders. They pointed out that fibers in the hoop direction confine the concrete by controlling its lateral expansion and microcracking, whereas fibers in the axial direction resist tension caused by bending and improve buckling resistance. They also established that failure of the system was governed by failure of the GFRP shell. Mirmiran and Shahawy (1996) reported the results of CFFTs of standard cylinder size with three different shell thicknesses controlled by the number of GFRP layers. The shells were made from filament-wound E-glass/polyester, with all the fibers oriented at winding angles of ±85 degrees. In loading the specimens axially, care

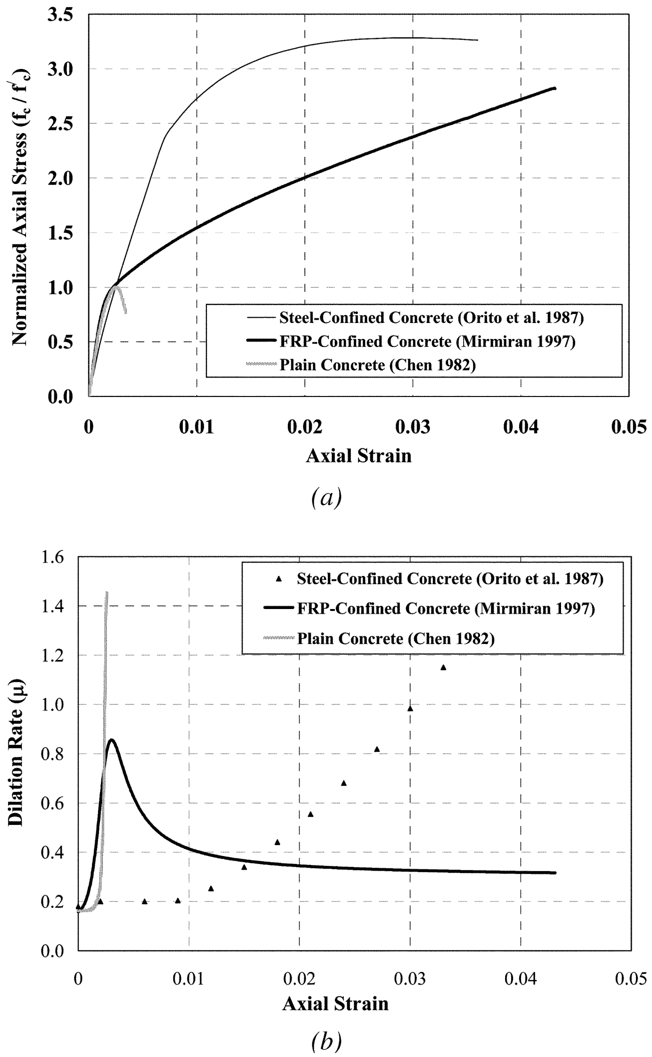


Fig. 9.3—FRP-confined concrete versus steel-confined concrete: (a) stress-strain curves; and (b) dilation behavior (Samaan et al. 1998).

was taken to only load the concrete and to not load the tube axially. Significant improvement in strength and ductility, compared with unconfined concrete, was observed. The stress-strain curve was almost bilinear, with a curved transition near the point of unconfined concrete strength, as shown in Fig. 9.3(a). Fam and Rizkalla (2001a) tested concrete-filled filament-wound GFRP tubes with 2/3 of the fibers oriented in the hoop direction and 1/3 in the axial direction. Axial load was applied to both the tube and concrete. Specimens with and without central holes of different sizes and a tube-in-tube system with concrete filling in between were tested. The study showed that the larger the size of the inner hole, the lower the confinement effect. Also, loading the tube axially reduced the confinement effect. Kanathrana and Lu (1998) and Fam and Rizkalla (2001a) showed that FRP pultruded tubes, with fibers in the axial direction, would split before adequate confinement occurred. This effect was also recognized by Davol (1998), who tested concrete-filled filament-wound CFRP tubes with 80% of the fibers oriented at ±10 degrees and 20% oriented mainly in the hoop direction.

9.7.2 *FRP confinement versus steel confinement*—The stress-strain behavior of concrete-filled GFRP tubes is compared with that of concrete-filled steel tubes in Fig. 9.3(a) (Samaan et al. 1998). The steel jacket confines concrete at low load levels if not loaded axially, due to its high Young's modulus. FRP-confined concrete is insensitive to small lateral expansion due to its lower Young's modulus, particularly for GFRP. As the unconfined strength is approached, high lateral expansion occurs due to major microcracking, which activates the FRP jacket. Passive confining pressure, continuously increasing due to the linear characteristics of the FRP, is induced. Once the jacket reaches its hoop strength, it ruptures, and the concrete fails. On the other hand, steel jackets induce constant confining pressure once the steel yields. Samaan et al. (1998) attributed the difference in behavior to the distinctly different dilation behavior of concrete as shown in Fig. 9.3(b). The dilation response of FRP-confined concrete is initially similar to unconfined concrete. It reaches a peak value, after which it decreases and finally stabilizes. Steel jackets effectively confine the concrete and control dilation before yielding. After yielding of the confining steel, the lateral confining pressure from steel confinement becomes essentially constant, thus allowing for increasing lateral dilation of confined concrete with increasing axial strains.

9.7.3 *Critical factors affecting confinement*—The following equation summarizes the critical parameters affecting the confinement pressure σ_R in circular CFFT, which directly contributes to the enhanced axial strength of the concrete (Fam and Rizkalla 2001b)

$$\sigma_R = \frac{\nu_c}{\frac{R}{E_f t} + \frac{1 - \nu_c}{E_c}} \varepsilon_{cc} \tag{9-1}$$

At any axial strain level ε_{cc}, σ_R is increased as the stiffness of the tube in the hoop direction ($E_f t/R$) is increased, where E_f is the equivalent orthotropic elastic modulus of the tube in the hoop direction, t is the thickness of the tube, and R is the radius. The confinement pressure σ_R also increases as the dilation of concrete increases as reflected by its Poisson's ratio ν_c. E_c is the modulus of the concrete core.

Equation (9-1) represents an FRP tube fully utilized in the circumferential direction (debonded from concrete and not loaded axially), as shown in Fig. 9.4(a). Figure 9.5 shows the effect of the stiffness of the tube ($E_f t/R$) on the confinement effectiveness (f'_{cc}/f'_c), where f'_{cc} and f'_c are the strengths of the confined and unconfined concrete, respectively (Fam and Rizkalla 2003).

9.7.4 *Effect of loading tube axially*—Axial loading of the tubes is often unavoidable due to friction, adhesion, and surface irregularities, which provide some axial load transfer to the shell, as shown in Fig. 9.4(b). In this case, the confinement pressure σ_R can be given by an expression similar to Eq. (9-1), except that the numerator is ($\nu_c - \nu_f$) instead of ν_c, where ν_f is the longitudinal Poisson's ratio of the tube defined as the expansion in the hoop direction due to loading in the axial direction. This indicates that, as the tube is loaded axially, it expands radially and tends to separate from the concrete, which reduces the confinement effect (Davol et al. 2001; Fam and Rizkalla 2001b). Therefore, the FRP laminate should be designed to provide a small longitudinal Poisson's ratio ν_f. Unlike steel, FRP laminates can be engineered to control such variables. Loading the FRP tube also results in development of a biaxial state of stress as shown in Fig. 9.4(b), which reduces the strength of the FRP shell in the hoop direction. In this case, the biaxial failure envelope of the laminate needs to be established and the combined state of stress (axial compression and hoop tension) compared with the failure envelope to detect failure, as shown in Fig. 9.4(c) (Fam and Rizkalla 2001b).

9.7.5 *Effect of central holes*—Providing a central hole inside the concrete core as shown in Fig. 9.1(e) reduces the self-weight of the member, but it also reduces the confinement effect by reducing the confining pressure as shown in Fig. 9.5, which shows the effect of inner-outer diameter ratio (D_i/D_o) on the confinement ratio (Fam and Rizkalla 2003). It was also shown that providing an additional FRP tube inside the hole would improve the confinement efficiency of these members.

9.7.6 *Slenderness effect*—Mirmiran et al. (1998) reported a 20% reduction in axial strength of CFFTs with length-diameter ratios (L/D) of 5:1 as compared with specimens with (L/D) of 2:1. More recent studies (Mirmiran et al. 2001; Yuan and Mirmiran 2001) concluded that for CFFT columns not braced against sidesway with slenderness ratio (as defined in ACI 318) of 11 or less, the effect of slenderness may be neglected. For CFFT columns with slenderness ratio higher than 11, however, the maginified sway moment should be taken into account. It should be noted that this limit is lower than that specified in ACI 318 for conventional reinforced concrete columns, which is typically 22. To calculate the maginified sway moment, the study further recommended that the stiffness reduction factor in the moment magnification method of ACI 318 for CFFTs be reduced according to the equivalent orthotropic axial modulus of the FRP tube.

9.7.7 *Effect of geometry of cross section*—Square CFFTs with a corner radius equal to 4.2% of the width of the columns were tested by Mirmiran et al. (1998). Filament-wound GFRP tubes with different wall thicknesses were used. The ultimate axial strength was lower than the peak strength and stabilized at 70% of the unconfined peak strength, irrespective of the tube thickness. The specimens failed due to stress concentration at the corners (Mirmiran 2003). Picher et al. (1996) tested specimens with corner radius varying from 3.3 to 50% of the width of the specimen. Although the specimens were wrapped with CFRP sheets instead of CFRP tube, the effect of corner radius was clearly shown as in Fig. 9.6. The figure shows that increasing the radius causes the behavior of the square columns to become gradually similar to that of circular specimens. This concept was implemented in the proposal of confined-rectangular sections that feature a corner radius of 1/3 of the section width (Burgueño et al. 1996). The behavior of concrete-

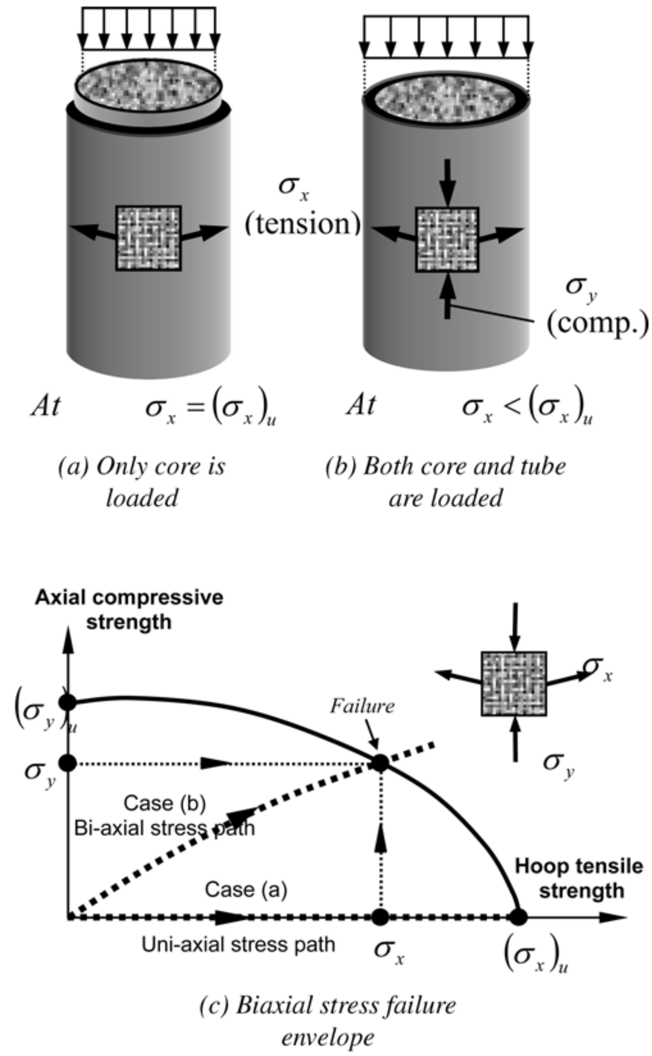


Fig. 9.4—Effect of loading FRP tube axially (Fam 2000).

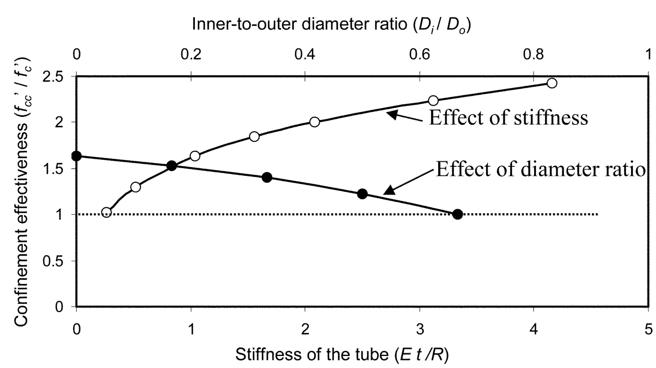


Fig. 9.5—Effect of stiffness of tube and inner hole size on confinement (Fam and Rizkalla 2003).

filled confined-rectangular CFRP tubes was investigated by Davol (1998).

9.7.8 *Effect of sustained loading*—Creep of concrete in CFFT is much lower than that estimated by ACI 209R models due to sealing of concrete by the tube, lateral confinement, and stress redistribution that takes place between concrete and the FRP tube in the axial direction (Naguib and

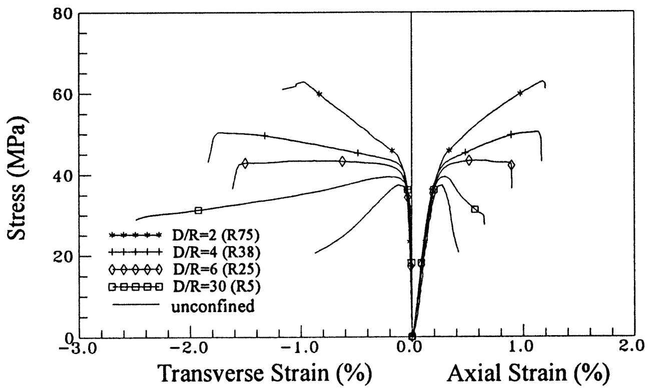


Fig. 9.6—Effect of corner radius on confinement of rectangular members (Picher et al. 1996).

Mirmiran 2001). As the stiffness of the tube increases relative to that of concrete core, larger stress redistribution occurs over time, yielding a lower creep coefficient for the concrete core.

9.7.9 *Bond effects*—Bond between the FRP jacket and the concrete core is more important for flexural members than it is for axial members, and can be achieved by using adhesive or mechanical shear connectors. Fam and Rizkalla (2001b) have shown that, for circular CFFTs under axial compression, the confinement effect is higher if the tube is not loaded axially, which can be achieved by debonding and loading the core only. Mirmiran et al. (1998) tested square CFFTs, with shear connector ribs in the axial and transverse directions, under axial compression. Although the specimens did not achieve the same effectiveness of circular tubes, it was reported that the ribs helped improve the load-carrying capacity by distributing the confining pressure more effectively around the circumference of the tube and minimizing the stress concentrations at the corners. Longitudinal ribs increased the buckling and compressive resistance. The horizontal ribs helped to maintain the cross-sectional shape of the skin and aided the shell in resisting hoop stresses.

9.7.10 *Confinement models*—Different analytical models have been developed to predict the stress-strain response of confined concrete using FRP round shells. The models recognize that confining pressure is variable and is a function of the interaction between the dilating concrete core and the FRP tube with linear characteristics (that is, passive confinement), which is different from steel-confined concrete, where the confining pressure is stable once the steel yields. Some of the available models can predict the entire stress-strain relationship for confined concrete, whereas others may only predict the peak strength (Harries et al. 1998). Some models are based on dilation relationship for confined concrete (Davol 1998; Samaan et al. 1998), modified Ahmad and Shah's model (Saafi et al. 1999; Toutanji 1999), or modified Mander's model (Mander et al. 1988) using different relationships between lateral and axial strains (Mirmiran and Shahawy 1995; Spoelstra and Monti 1999; Burgueño 1999; Fam and Rizkalla 2001b), the energy-balanced technique (Parent and Labossière 1997), or octahedral stress theory for triaxial stress state (Becque 2000). Some of the models account for the biaxial state of stress in the FRP shell under axial loading as well as the effect of a central hole in the concrete core (Fam and Rizkalla 2001b; Davol et al. 2001). A recent study (Naguib and Mirmiran 2001) showed the significant effect of strain rate on the level of confinement, which could potentially affect the comparison of various confinement tests. Several researchers (Picher et al. 1996; Mirmiran et al. 2000a) used plasticity models, such as Drucker-Prager, in finite element analysis to predict the behavior of FRP-confined concrete. A comparative study and survey of some of the confinement models that were published before June 2001 can be found in De Lorenzis and Tepfers (2003).

9.8—Behavior of flexural and axial/flexural members

Hybrid structural members, such as those shown in Fig. 9.1, have great potential for flexural and combined loading applications. Although the confinement of concrete is significantly reduced in bending, even in CFFTs, the advantages of using FRP as permanent formwork and ease of fabrication still make this system attractive. The role of concrete is to resist the compressive stresses, while that of the FRP component is to resist the tensile stresses. In CFFTs, however, concrete has another role to support the FRP shell in the compression side to avoid local buckling failure. Davol (1998), Burgueño (1999), and Fam and Rizkalla (2002) have reported increases in flexural strength of CFFTs of over two times that of the hollow tube and have also shown that, the thicker (or stiffer) the tube, the lower the contribution of concrete to strength.

9.8.1 *Background of closed form systems*—Seible et al. (1995) introduced a CFFT using carbon shells for bridge piers and girders. Two different connection designs were introduced for bridge piers in seismic zones: a ductile design concept with steel bars connecting the CFFT to the footing, and a strength design concept with the CFFT embedded inside the footing. A carbon/epoxy CFFT system was also developed and proposed as the main element for entire bridges such as a dual tied arch, a space truss with post-tensioned members, and cable-stayed bridges (Seible et al. 1998). Conventional reinforced concrete and FRP decks were also proposed to be connected to the CFFT girders using steel or FRP dowels (Burgueño 1999). A special mandrel was used to produce helical ribs on the inside of the shell to provide mechanical bond to the concrete (Burgueño 1999). Studies by Seible et al. (1997a), Burgueño et al. (1998), and Davol et al. (2001) have shown that the design of carbon CFFT structures is stiffness driven. Application of carbon CFFT to bridge systems for the California DOT was investigated through a building-block research program, including full-scale testing of CFFT components (columns and girders), connections, and beam-slab assemblies (Karbhari et al. 1998; Davol 1998; Burgueño 1999; Wernli 1999; Zhao 1999). Tests on 40% scale test units were used to develop a design philosophy for carbon CFFT bridge columns under seismic loads. Flexural behavior was investigated on full-scale carbon CFFT girders for beam-slab bridges through four-point bending tests (Davol 1998; Burgueño 1999). The application of carbon CFFT to beam-slab bridges was validated through the full-scale testing of single and multiple beam/slab assemblies with conventional reinforced concrete decks and modular E-glass/vinylester FRP decks as shown in Fig. 9.7 (Burgueño 1999; Zhao 1999). These later experimental studies addressed not only short-term behavior, but also long-term damage tolerance through simulated cyclic and fatigue loading.

Fam (2000) conducted a large experimental program to study the flexural behavior of hollow and concrete-filled tubes ranging from 90 to 940 mm (3.5 to 37 in.) in diameter with different laminate structure, central holes, and with different diameter-to-wall thickness ratios to model fender piles. The spans of the beams ranged from 1.07 to 10.4 m (3.5 to 34 ft) as shown in Fig. 9.8. Fam et al. (2003b) also studied the behavior of CFFTs under combined bending and axial loads by applying eccentric axial loads. The full interaction curves were developed including tension, compression, and balanced failure regions. Behavior of rectangular CFFTs was studied by Davol (1998), Mirmiran et al. (1999), and by Fam et al. (2003a) using configurations similar to those in Fig. 9.1(f) and (g), respectively.

9.8.2 *Background of open form systems*—Fardis and Khalili (1981) proposed casting concrete into FRP boxes as shown in Fig. 9.1(a). They pointed out that FRP resisted the tensile forces, provided partial confinement of concrete, and carried part of the shear force. They also showed that adhesion between the concrete and FRP was not necessary provided that the FRP box was closed at the two ends. The strength of all the beams tested was greater than that of typical conventional reinforced concrete beams of the same dimensions. Triantafillou and Meier (1992) and Deskovic et al. (1995) presented a hybrid box section, shown in Fig. 9.1(b), consisting of a GFRP box section to resist shear, with an upper layer of concrete in the compression side and a thin layer of CFRP in the tension side. They observed gradual failure, starting by fracture of CFRP before the GFRP, accompanied by the sudden drop of load and reduced stiffness. This provided adequate warning before complete failure (pseudoductility). Seible et al. (1998) proposed a similar hybrid system of prefabricated hybrid carbon-E-glass/vinylester box sections compositely connected to a fiber-reinforced concrete deck. Hall and Mottram (1998) presented a hybrid section combining GFRP off-the-shelf pultruded sections, commercially available as floor panels, with concrete as shown in Fig. 9.1(c). The GFRP system had two T-up stands and a continuous base. Beam tests concluded that adhesive bond was essential and that failure was often by concrete in shear. The beams were over-reinforced in flexure.

Dieter et al. (2002) reported results of laboratory tests conducted on a concrete bridge deck constructed using FRP stay-in-place forms. The pultruded FRP form spanned between, but was not continuous over the girders. Each 457 mm (18 in.) wide FRP form was stiffened by two 76 mm (3 in.) square, hollow corrugations. To ensure composite action through horizontal shear transfer, coarse aggregates were bonded to the FRP surface using epoxy. Additional FRP grid and reinforcing bar reinforcement was placed in the top of the slab and in the longitudinal directions. The deck failed by punching shear. The tested system was used to construct a 66.5 x 13.7 m (218 x 45 ft) bridge deck on prestressed concrete girders on State Highway 151 over State Route 26 in Waupun, WI, in 2003. The bridge is currently in service, carrying an average daily traffic of 18,000 vehicles. The construction and cost analysis of the bridge were described in Berg et al. (2004).

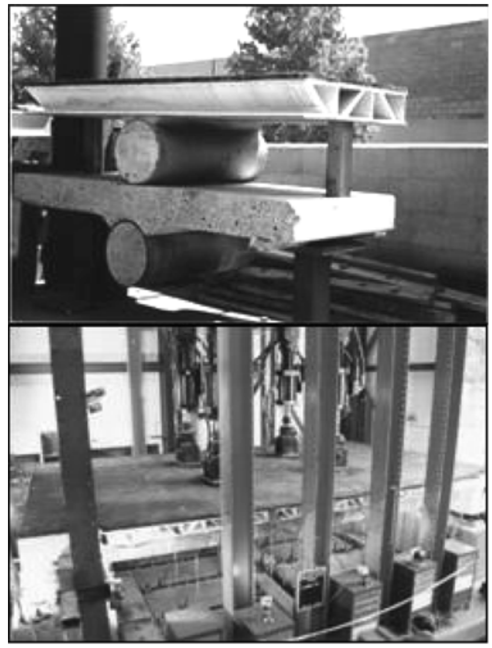


Fig. 9.7—Testing of CFFT with concrete and GFRP deck for bridge applications (Burgueño 1999).

Fig. 9.8—Beam test of large CFFT for marine pile applications (Fam 2000).

Ulloa et al. (2004) reported the use of a U-shape GFRP beam section in composite action with a concrete deck for the San Patricio County bridge in Texas. The connection consisted of horizontal steel bars passing through holes in the GFRP webs and embedded in the concrete deck. The research demonstrated that composite action from a reinforced concrete deck could overcome deflection limitations inherent in FRP beams.

9.8.3 *Effect of reinforcement ratio and laminate structure in CFFT flexural members*—Fam and Rizkalla (2002) showed that flexural strength and stiffness of CFFTs were

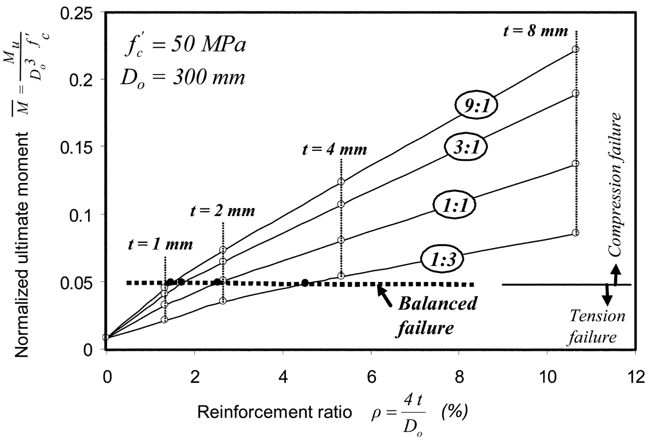


Fig. 9.9—Variation of flexural strength with reinforcement ratio for different laminate structures (Fam and Rizkalla 2002.)

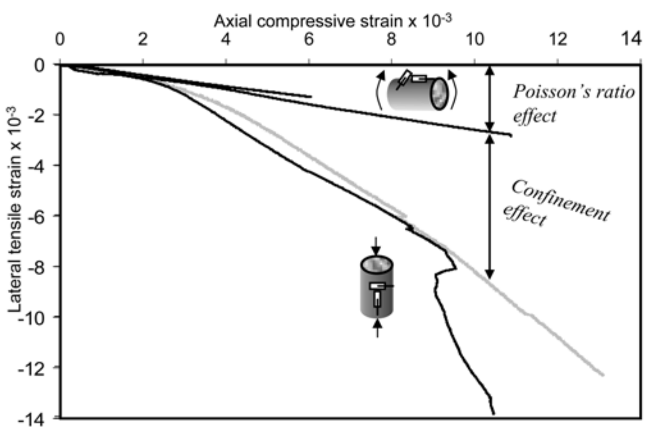


Fig. 9.10—Comparison between axial-lateral strain behavior in beams and columns (Fam and Rizkalla 2002).

governed by both the reinforcement ratio ρ and the laminate structure of the tube, where $\rho = 4t/D$, D is the diameter, and t is the thickness. Figure 9.9 shows the variation of the flexural strength with ρ for different laminate structures of the FRP tube, which had fibers oriented in the axial and hoop directions with various proportions designated as 1:3, 1:1, 3:1, and 9:1. A 1:3 laminate indicates that only 25% of the fibers are oriented in the axial direction. For a given laminate structure, increasing the wall thickness could change the failure mode from tension to compression. Similarly, for a given ρ, changing the laminate structure by increasing the stiffness in the axial direction could change the failure mode from tension to compression. Figure 9.9 also shows that the balanced ρ is reduced as the tube becomes stiffer (or thicker) in the axial direction.

9.8.4 *Confinement effect in CFFTs in bending*—When CFFT round members are subjected to bending, experimental studies (Burgueño 1999; Davol et al. 2001; Fam and Rizkalla 2002) have shown that the effect of confinement of concrete is insignificant. Figure 9.10 shows the axial strain versus the lateral strain behavior of the FRP tube in the compression zone of a beam, tested by Fam and Rizkalla (2002), versus that of a column of the same type. The figure shows that the behavior is bilinear for columns, with significant increase in lateral strains due to confinement. For beams, however, the behavior is linear, with a slope proportional to the longitudinal Poisson's ratio of the tube, which indicates lack of confinement. This is attributed to the strain gradient, where most of the cross section of the beam is in tension.

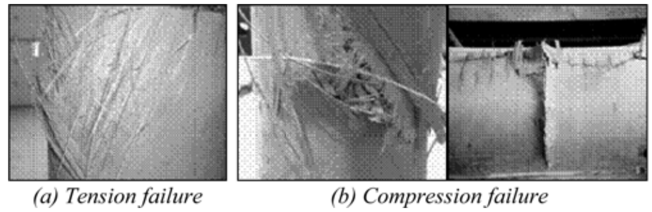


Fig. 9.11—Failure modes of CFFT: (a) tension failure; and (b) compression failure (Fam et al. 2003b).

9.8.5 *CFFTs subjected to combined bending and axial loads*—Round CFFTs have been tested under constant axial loads and increasing bending (Seible et al. 1995; Mirmiran et al. 2000b) and under increasing eccentric axial loads (Fam et al. 2003b). Compression and tension failures were achieved as shown in Fig. 9.11. Fam et al. (2002) studied the effect of both the wall thickness and laminate structure (different proportions of fibres in the axial and hoop directions) on the interaction curves. The study showed that, for thin tubes, increasing the ratio of fibers in the hoop direction would increase the axial strength and reduce the flexural strength as evident from the curves in Fig. 9.12(a), which intersect at the optimal points for laminate design for each eccentricity. For thick tubes, increasing the amount of fibers in the axial direction increased both the axial and flexural strength, as shown in Fig. 9.12(b). Additionally, for thick tubes, the entire interaction curve could be governed by compression failure.

9.8.6 *Splices and joints in CFFTs*—Because of the limited lengths of CFFTs, splices could be needed. Parvathaneni et al. (1996) produced a 13.7 m (45 ft) long CFFT pile using three 4.57 m (15 ft) long units, spliced using short steel tubing 0.6 m (2 ft) long, which matched with the inside of the GFRP tubes. Ductile joints have been proposed between CFFT bridge columns and footings using short steel dowels (Seible et al. 1998). Pseudoductile plastic hinges have also been proposed for girders (Wernli and Seible 1998; Wernli 1999) using CFRP dowels that provide ductility through gradual slip between the concrete and the bars. The achieved ductility, however, is only in one direction, and the deformation and damage cannot be reversed. The load-slip characteristics for CFRP dowels with varying anchorage details were determined through numerous pullout tests. The behavior of the connection concepts was validated through full-scale flexural testing of longitudinally spliced girders (Wernli 1999). Seible et al. (1998) have also introduced connections between CFFT beams and deck slabs using steel dowels. The connections were studied through pushout tests and full-scale testing of CFFT beam/slab assemblies, which led to the development of design and analysis recommendations (Zhao 1999). Steel dowels for CFFT columns in seismic zones were introduced by Seible et al. (1995) and Burgueño (1999).

9.8.7 *Prestressed members*—Parvathaneni et al. (1996) proposed using filament-wound CFFTs prestressed in the

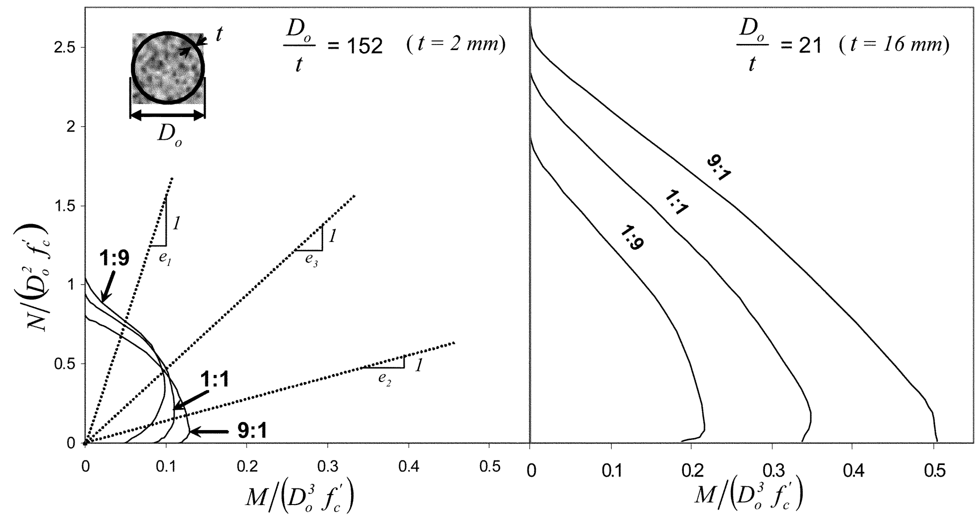
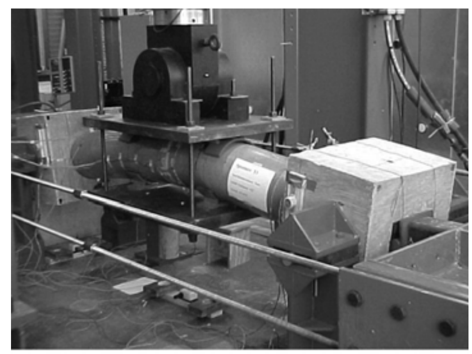


Fig. 9.12—Effect of thickness and laminate structure of tubes on interaction curves of CFFT (Fam et al. 2003b).

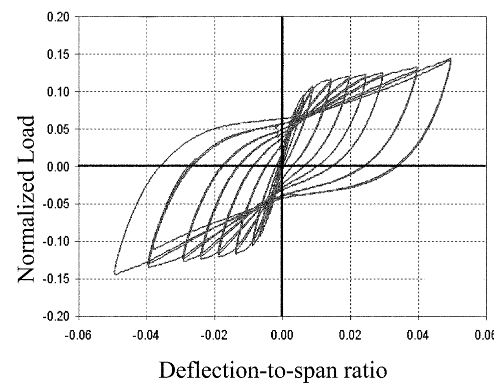
axial direction to produce alternative mooring piles. It was decided to take advantage of the high confined strength by prestressing the concrete to a high compressive stress, which was described as super-prestressing. Three 35 mm (1.4 in.) diameter steel Dywidag bars were pretensioned inside the tubes; 35 MPa (5 ksi) concrete was cast and cured; and finally, the bars were destressed, producing a 31 MPa (4.5 ksi) compressive stress in concrete. The conventional method of driving the pile was used. The maximum recorded dynamic strain in concrete was 1360 microstrains in compression, and no tension stresses were induced.

9.8.8 *Hysteretic behavior of CFFTs*—Seible et al. (1995) have tested carbon shell CFFTs with and without mild steel reinforcement anchorage bars as the CFFT-to-footing connection to study the response of CFFT bridge columns under simulated seismic loads. Shao (2003) has modeled and tested CFFTs under low-cycle fatigue, including the effect of loading and unloading on FRP-confined concrete and the seismic behavior of CFFTs with and without internal reinforcement, as shown in Fig. 9.13. Fan et al. (2000) have reported a ductility factor of 10 for CFFTs with internal mild steel reinforcement.

9.8.9 *Sustained loading*—Recent experimental and analytical investigations by Naguib and Mirmiran (2001) have shown that creep effects reduce the flexural stiffness of CFFT. Ultimate strength, however, is not significantly altered. A slow rate of loading and short-term creep at 70% of static capacity may cause premature rupture of the tube. Fiber analysis of CFFT beam-columns by discretizing the section into filled and hollow FRP tubes can adequately simulate the flexural creep behavior. Isochronous sustained stress-creep strain curves may be used as a constitutive nonlinear relationship for creep analysis in flexure. Creep deflection of beam-columns is much less than that of beams, mainly because axial compressive loads tend to retard cracking of concrete and tensile creep of the FRP tube. The axial stiffness ratio of the FRP tube with respect to the concrete core has a pronounced effect on creep deflection of CFFT beam-columns. As the stiffness ratio increases, creep deflections



(a) Cyclic loading test setup



(b) Cyclic loading test results

Fig. 9.13—Cyclic loading test of CFFT (Shao 2003).

decrease. There exists, however, a threshold beyond which stiffer tubes do not provide additional benefit.

CHAPTER 10—MASONRY APPLICATIONS

10.1—Introduction

"Building Code Requirements for Masonry Structures" (ACI 530/ASCE 5/TMS 402) covers the design and construction of masonry structures. The code provides requirements for design and construction of new structures. Repair, retrofitting, and rehabilitation of masonry structures are not included in the document.

FRP composites using various polymer and cementitious matrixes, and fiber reinforcement of treated and untreated glass, carbon, and aramid fibers have all been applied for strengthening of masonry. This chapter summarizes work that has been focused on FRP composite systems for strengthening of masonry structures.

Potential advantages of retrofitting masonry using FRP composites include low installation costs, flexibility of use, and minimum changes in the member size after repair. Disturbance to occupants and loss of usable space are also minimized. From a structural point of view, the dynamic properties of the existing structure remain unchanged because there is little addition of weight. If stiffness change is required, it may be engineered on a case-by-case basis by properly designing the composite retrofit.

Even though most of the research on FRP composites and field applications has focused on strengthening reinforced concrete members, available literature shows high potential for reinforcing and strengthening masonry. A research project between the U.S. Army Corps of Engineers and the Market Development Alliance (MDA) of the FRP Composites Industry (Marshall et al. 1999) tested over 100 clay and concrete masonry walls under in-plane loading, and produced a wealth of data on the increased strength and ductility of these walls. The efforts ended with a seismic simulation test in which four FRP composite systems were used to seismically retrofit a half-scale two-story brick building with dimensions of 3.66 x 3.66 x 3.66 m (12 x 12 x 12 ft). The main conclusions were:

- FRP composites can be applied to increase the strength of masonry walls in shear;
- FRP composites enable greater wall drift before failure occurs;
- For shear, glass fiber is preferred over carbon fiber because of the lower stiffness of glass; and
- The failure mode of masonry wall sections can be changed by the application of FRP. By the proper placement and selection of FRP composites on an unreinforced or a lightly reinforced masonry wall, failure modes such as x-cracking can be prevented while transferring the failure to a more ductile mode such as bed joint sliding or rocking before toe crushing.

The U.S. Army Corps of Engineers has published guidelines on the specification and construction of masonry repaired with FRP composites (UFGS 2004a,b). Furthermore, ACI Committee 440 and the Existing Masonry Committee of The Masonry Society (TMS) have established a joint task group for the development of design provisions.

10.2—FRP strengthening techniques

FRP composite products, in the form of externally bonded laminates and grids, and NSM bars are the typical approaches used to strengthen masonry structures. FRP composites have been primarily investigated for enhancing the structural capacity of masonry walls and columns (Masia and Shrive 2003).

10.3—FRP repair and strengthening of masonry

10.3.1 *Flexural strengthening*—Many research projects have been conducted to study FRP systems for flexural strengthening of masonry walls. Ehsani and Saadatmanesh (1996) investigated the flexural behavior of small-scale unreinforced masonry (URM) walls strengthened with GFRP sheets and found that the flexural capacity was increased up to 24 times compared with the unreinforced control specimen. According to the test results, the effect of the mortar strength appeared to be negligible, and both specimens failed by crushing of the masonry.

Velazquez-Dimas et al. (2000) reported test results of half-scale URM walls tested under out-of-plane cyclic loading. Two of the walls were strengthened on both faces with GFRP strips. Substantial increases in strength and deformation capability were achieved. The retrofitted walls resisted pressures up to 24 times the weight of the wall and deflected as much as 5% of the wall height. To avoid very stiff behavior and improve the hysteretic response, the authors recommended limiting the reinforcement ratio to two times the balanced condition. The balanced condition is defined as the point at which failure of the masonry in compression and rupture of the composite in tension occur at the same time. Although the brittle URM walls were retrofitted with a linear elastic material, the combination resulted in a system capable of dissipating some energy representing system nonlinearity.

Hamilton and Dolan (2001) investigated the flexural behavior of small-scale URM walls strengthened with different composite materials. Strengthening with high-strength composite materials such as CFRP and AFRP (with vertical fiber orientation) led to modes of failure such as delamination and shear in the masonry. To use the composite material more efficiently, two alternatives were recommended: first, to increase the spacing of the material until rupture of the laminate governed failure, and second, to use less expensive materials such as GFRP. These more efficient alternatives resulted in four failure modes: debonding, laminate rupture, masonry shear, and face shell pullout. They reported that debonding from the masonry substrate caused the failure of most of the test specimens.

The successful use of NSM bars for improving the flexural capacity of reinforced concrete members (De Lorenzis et al. 2000) led to extending this technique to URM walls. As an example, masonry panels of concrete blocks were tested by Tumialan et al. (2002). One specimen was strengthened with one No. 3 GFRP bar (9.5 mm [0.375 in.] nominal diameter), the second with two No. 3 GFRP bars, and the third was strengthened with an externally bonded GFRP laminate (width = 76 mm [3 in.]). For comparison purposes, specimens one and three had an equivalent axial stiffness $E_{FRP} \times A_f$ (modulus of elasticity × area) to each other. The wall strengthened with one GFRP bar failed due to debonding of the paste from the masonry. Initial flexural cracks formed at the mortar bed joints perpendicular to the reinforcement, and caused secondary cracks at the epoxy paste-masonry interface resulting in debonding and subsequent wall failure. The wall strengthened with two bars failed due to masonry shear, while the specimen with the GFRP laminate failed due to debonding. This experimental program was used as a validation for the strengthening of two URM concrete walls at an educational facility in Kansas City, Missouri, where the walls exhibited cracking in the bed joints at the midheight region.

By using epoxy strengthened with short fibers, Bajpai and Duthinh (2003) were able to prevent debonding of NSM GFRP bars and consistently rupture the bars in flexural tests of masonry walls. This method resulted in higher wall strength and a more brittle behavior.

The capacity of flexural walls strengthened with FRP laminates is a function of the axial load level (Triantafillou 1998b). Moreover, FRP composites are highly effective in the case of walls that can be treated as simply supported (that is, walls exhibiting a large slenderness ratio). For a wall with a low slenderness ratio built between rigid supports, FRP is less effective because arching action of the wall dominates over the effect of the FRP because crushing of the masonry units at the boundary regions controls ultimate behavior (Tumialan et al. 2002; Galati 2002).

In summary, available literature indicates that URM walls strengthened with FRP exhibit the following modes of failure: 1) debonding of the FRP laminate from the masonry substrate; 2) flexural failure (that is, rupture of the FRP laminate in tension or crushing of the masonry in compression); or 3) shear failure in the masonry. Of these three modes of failure, the literature has shown that the controlling mode is mostly debonding of the FRP laminate. Thus, the quantity of FRP reinforcement should be balanced against the masonry shear strength; if a large amount of FRP reinforcement is provided, a brittle masonry shear failure may result. Proper masonry design philosophy dictates that brittle shear failure should be avoided by ensuring masonry flexural capacity is exceeded by its shear capacity.

Debonding is directly related to substrate surface characteristics such as roughness, soundness, and porosity. For instance, Roko et al. (1999) observed that absorption of epoxy is limited in extruded brick units as compared with that in molded bricks, leading to a reduction of the bond strength at the FRP laminate-masonry interface.

Tumialan et al. (2002) suggested that, rather than attempting to predict bond failure, the strain in the FRP laminates could be limited. The effectiveness of the FRP reinforcement depends on the bond of the FRP laminate to the masonry substrate. Because the flexural capacity is dependent on the strain developed in the laminate, effective strain in the laminate ε_{fe} can be expressed as the product $\kappa_m \varepsilon_{fu}$, where κ_m is the bond-dependent coefficient, and ε_{fu} is the design rupture strain of FRP. Tumialan et al. (2002) concluded that for nonputtied surfaces, κ_m can be assumed to be 0.45, and for puttied surfaces, κ_m can be 0.65.

Luciano et al. (2001) investigated the possibility of reinforcing masonry arches using FRP composite materials and found that the FRP laminates greatly enhanced the capacity of masonry arches.

To enhance the out-of-plane seismic resistance of the facades of historic masonry buildings, unobtrusive FRP rehabilitation techniques that incorporate intermittently bonded NSM carbon fiber rope and unbonded and intermittently bonded NSM carbon fiber composite cables (CFCC) were developed at McMaster University (Korany 2004). Ten full-size clay brick wall panels were retrofitted and tested under both monotonic loading and quasi-static cyclic loading using an airbag. Korany and Drysdale (2004) reported significant increases in ultimate capacities, energy absorption, and deformability compared with the behavior of the unreinforced walls.

10.3.2 *Shear strengthening*—Schwegler (1995) investigated strengthening methods for masonry shearwalls with FRP laminates. CFRP laminates were bonded diagonally to the masonry walls and mechanically anchored to the adjoining reinforced concrete slabs. The test results showed that the strengthened walls exhibited 50 and 300% increases in ultimate capacity and displacement, respectively, as compared with unstrengthened walls.

Cracked URM concrete block walls were repaired by Gergely and Young (2001) using CFRP laminates attached to both sides of the specimens and subjected to cyclic out-of-plane loads and in-plane loads. The symmetric laminates significantly increased the flexural and shear capacity of damaged walls. The specimens failed as a result of severe shear damage in the concrete masonry blocks.

Concrete masonry walls strengthened with FRP laminates in the horizontal direction only and tested with in-plane loading along the wall diagonal were observed to fail due to sliding shear along an unstrengthened joint (Tumialan et al. 2001; Morbin 2001). This mode of failure, which is undesirable if there are adjacent columns such as in the case of infill walls, may be controlled by placing FRP bars in the vertical direction to act as dowels.

As in the case of URM walls strengthened for flexure with FRP laminates, the type of masonry has been observed to be one of the factors influencing the in-plane wall behavior. Thus, in the case of clay brick masonry walls strengthened with laminates for shear, FRP strengthening has been observed to be more efficient than in the case of concrete masonry (Grando et al. 2003). This can be attributed to characteristics of the parent material such as height of masonry courses (that is, smaller in the case of brick masonry) and better mortar-masonry unit bond characteristics. Grando et al. (2003) also reported that the in-plane capacity of clay masonry walls strengthened on one and two faces doubled when the amount of FRP reinforcement was doubled.

Valluzzi et al. (2002) reported experimental results on small-scale clay brick masonry specimens using variables such as the type of FRP laminates (CFRP and GFRP) and strengthening configurations (single-side versus double-side strengthening, and square grid and diagonal). Double-sided strengthened specimens were more effective than single-sided specimens. In general, the diagonal strengthening configuration was observed to be more effective than the grid configuration; also, GFRP laminates were more effective than CFRP at increasing shear capacity.

Bastidas et al. (2002) investigated the strengthening with GFRP laminates of small-scale nonstructural masonry walls built with clay tiles. In addition, a full-scale wall was tested to validate the technology. The strengthening configurations included vertical and horizontal laminate strips, combinations of both, and diagonal laminates (cross-pattern). The test results showed the efficiency of the GFRP reinforcement for increasing the shear strength as well as the ductility of the system. The cross-pattern layout on both sides of the wall proved to be the most effective configuration. A significant global reduction of damage levels was observed for the strengthened masonry wall when compared with results reported by the same authors on similar URM walls. Also, global stability and overall seismic behavior were greatly improved with the GFRP reinforcement for in-plane loading.

Strengthening by FRP structural repointing can also remarkably increase the shear capacity of URM walls. Repointing is a technique used in masonry to repair and replace the mortar in the joint. With FRP repointing, the mortar is cleaned out, epoxy is placed into the groove, and an FRP bar is placed into the same groove into the epoxy. This was evident from the results of tests conducted on concrete masonry walls loaded along the diagonal (Tumialan et al. 2001; Morbin 2001). The maximum increase in shear capacity was 80% for walls strengthened with GFRP bars placed at every bed joint. Walls with reinforcement staggered on both wall faces exhibited the largest pseudoductility.

FRP structural repointing was also used to improve the in-plane structural performance of masonry infill walls (Tumialan et al. 2003a,b,c). Full-scale specimens were subjected to in-plane cyclic load. The specimens were surrounded by reinforced concrete frame and a stand-alone reinforced concrete support. The results indicated that FRP-strengthened specimens could reach lateral drifts of 0.7% without losing lateral carrying capacity, and, that for this drift level, the degradation of lateral stiffness in the strengthened walls did not implicate degradation of lateral carrying capacity.

With the objective to find alternative embedding materials to epoxy-based paste, Turco et al. (2003) investigated the in-plane behavior of concrete masonry walls strengthened with GFRP bars embedded in two different materials: epoxy-based paste and latex-modified cementitious paste. The in-plane test results showed that the performance of walls with both materials yielded similar results. The use of less expensive pastes, such as the latex-modified ones, makes the FRP structural repointing technique more appealing because the structural performance is not reduced and the appearance of the filled joints is similar to conventional mortar joints.

The effectiveness of increasing the shear strength of brick masonry by epoxy-bonding FRP overlays to the exterior surfaces was evaluated by Ehsani et al. (1997a). The variables in the test included the strength of the composite fabric, fiber orientation, and anchorage length. Specimens were tested under static loading. The results showed that both the strength and ductility of tested specimens were significantly enhanced. The orientation of the angle of fibers with respect to the plane of loading had a major effect on the stiffness of the retrofitted system, but did not affect the ultimate strength significantly.

The experimental results of three half-scale unreinforced brick walls retrofitted with vertical composite strips were presented by Ehsani et al. (1999). The specimens were subjected to cyclic out-of-plane loading. Five reinforcement ratios involving two different glass fabric composite densities were investigated. The mode of failure was controlled by tensile failure when wider and lighter composite fabrics were used and by delamination when stronger ones were used. The strengthened specimens were able to support a lateral load up to 32 times the weight of the wall. A deflection as much as 2% of the wall height was measured.

Avorio and Borri (2001) studied the problem of seismic strengthening of monumental arches and vaults. The interest in this technique came about from the examination of the types of collapse involving arches and vaults and from problems shown by structures strengthened with traditional methods. In formulating the criteria, attention was paid to the behavior of the vaults according to their constructional type and the type of texture and pattern.

Moon et al. (2003) and Moon (2004) tested, under lateral loads, a full-size two-story URM brick building that was strengthened using several FRP techniques. On one three-wythe wall, GFRP was epoxy-bonded vertically on the inside face, while NSM glass rods were epoxy-bonded into horizontal bed joints on the exterior face. This two-way retrofit increased the lateral strength, caused cracks to be well distributed, and produced a ductile-type failure mode with broad hysteresis loops and considerable energy dissipation. The four FRP systems used in this project included: precured structural grids embedded in trowel-applied epoxy adhesive (Fig. 10.1); wet lay-up unidirectional glass fabrics with an epoxy matrix (Fig. 10.2); epoxy adhesive-applied NSM GFRP rods (Fig. 10.3); and glass grids in cementitious trowel-applied matrix (Fig. 10.4). Application of GFRP systems on the other multiwythe walls worked well in in-plane shear retrofit because header bricks every sixth course generally maintained continuity between wythes.

In-place tests were performed by Corradi et al. (2002) on FRP retrofitted masonry walls damaged by recent earthquakes. Both CFRP and GFRP unidirectional laminates were used to retrofit the masonry panels, followed by in-plane tests. The tests confirmed that the shear capacity of the masonry panels was significantly increased by the FRP materials.

10.3.3 *Settlement repair*—Hartley et al. (1996) tested two full-sized 200 mm (8 in.) concrete block walls, 2.4 m (8 ft) high and 6.0 m (20 ft) long to investigate the feasibility of using unidirectional CFRP sheets to repair settlement damage. In the study, settlement loads were first applied to induce characteristic step cracking. CFRP was then applied to one surface, and the wall retested. Strength gains of over 50% were recorded. The results suggested that CFRP was suitable for rehabilitating concrete block walls damaged by foundation settlement.

10.4—Design and application considerations

10.4.1 *FRP system selection requirements*—Several suitable FRP systems are currently available to repair or retrofit masonry structures. To select the proper FRP system for a particular project, several factors have to be considered by the design engineer and building owner. Some of these factors are:

- Types of masonry construction: non-load bearing, load bearing, or retaining walls, and parent material (that is, concrete or clay masonry unit);

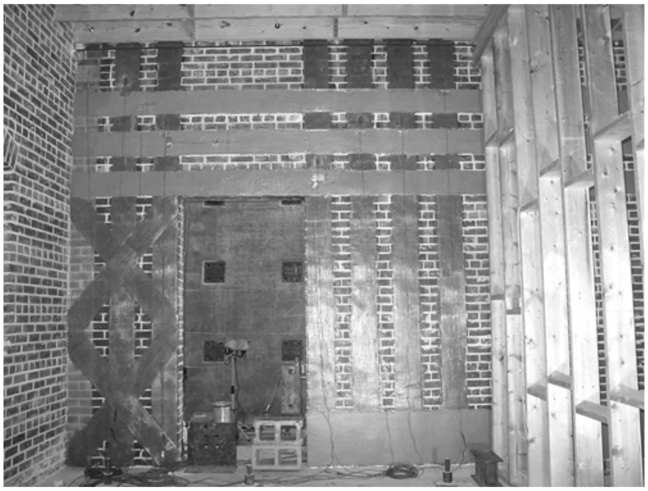


Fig. 10.1—Precured structural grids embedded in trowel-applied epoxy adhesive (Moon 2004).

Fig. 10.2—Wet lay-up unidirectional glass fabrics with epoxy matrix (Moon 2004).

Fig. 10.3—Epoxy adhesive-applied NSM GFRP rods (Moon 2004).

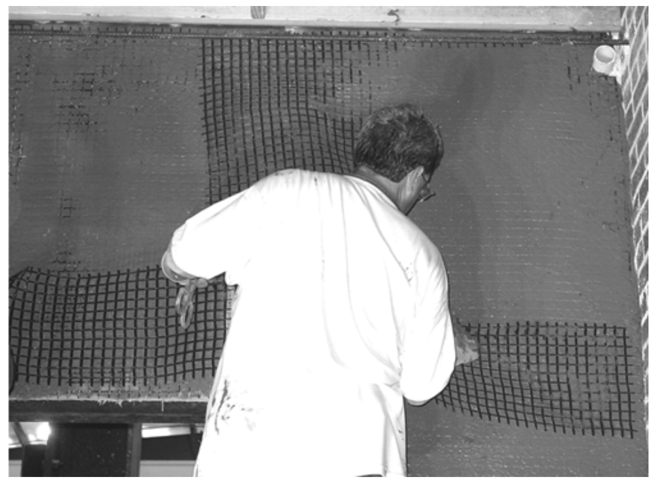


Fig. 10.4—Glass grid in cementitious trowel-applied matrix (Moon 2004).

- Overall building condition (damage level, presence of cracks, surface coatings, accessibility), with particular emphasis on the condition of the substrate. The condition and strength of the masonry substrate is an important parameter for bond-critical applications, including flexure or shear strengthening. The existing masonry substrate should possess the necessary strength to develop the design stresses of the FRP system through bond;
- Repair or retrofit impact on building operation;
- Building occupancy and use;
- Architectural considerations; and
- Building code and fire code requirements. Coatings can be used to limit smoke and flame spread.

Many of these factors have been addressed in several publications. Saadatmanesh (1994) provided an overview of the FRP applications for existing structures, including seismic retrofit of URM buildings. Christensen et al. (1996) studied the architectural implications of reinforcing existing masonry walls with FRP composite materials, the problems associated with the various substrates, and building code issues related to smoke and fire hazards. In the study, they also evaluated FRP-compatible architectural finishes.

10.4.2 *Detailing requirements*—In addition to strength and stiffness requirements, FRP retrofit or strengthening of masonry walls should also address application specific requirements. Among these are FRP composite reinforcement strength development, anchoring systems, and connections for the composite systems and between structural elements (Hamilton and Dolan 2001).

Proper FRP reinforcement detailing at wall boundaries is necessary to ensure expected performance and avoid premature failure due to debonding. For externally bonded laminates, this may be attained with anchorage systems that include steel angles, steel bolts, and FRP bars. Different systems offer their own advantages and disadvantages. Steel angles are easy to install, but aesthetically problematic. Angles in direct contact with the masonry surface may locally fracture the wall due to displacement and rotation restraint. Steel bolts have shown high effectiveness, but require a demanding installation effort (Schwegler 1995).

A technique similar to the one used for anchoring laminates in reinforced concrete joists strengthened in shear (Annaiah 2000) can be used for anchoring laminates in masonry applications. The installation technique consists of grooving a slot in the upper and lower boundary members. The ply is wrapped around an FRP bar that is placed and bonded in the slot with a suitable epoxy-based paste (Carney and Myers 2003a,b). For NSM or structural repointing construction, bars can be easily anchored into adjacent concrete members by drilling and embedding their extended terminations (Tumialan et al. 2003a,b,c).

NSM FRP bars can also be used to improve the anchorage of masonry walls to boundary reinforced concrete beams or foundations (Tumialan et al. 2002; Carney and Myers 2003a,b). Multiwythe steel reinforced masonry parapets built using clay units with standard dimensions were tested in-place. Several steel bars were missing or irregularly placed in the original construction. Before installing the GFRP bars, the holes in the reinforced concrete beam and slots in the masonry were filled with an epoxy-based paste. The design rationale was to provide enough flexural reinforcement to force the occurrence of shear failure. The masonry walls were loaded in-plane as cantilever walls. The control wall lost carrying capacity due to the crack growth caused by rocking. In the retrofitted wall, the opening of the horizontal crack in the strengthened side was controlled. Because of the eccentricity of the GFRP anchors, the wall tilted, preventing the development of the full flexural capacity. The improvement in capacity with respect to the unanchored parapet, however, was over 100%.

10.4.3 *Surface preparation*—Once the adequate FRP system has been selected and designed for the repair or retrofit project, the masonry surface to which the FRP system will be applied should be properly prepared. Surface preparation is necessary to adequately transfer the forces between masonry elements and surface-bonded FRP composite overlays. This preparation consists of complete removal of all mortar droppings, dust, dirt, oil, existing paint or coatings, and efflorescence from the masonry surface. Smooth-faced epoxy-coated or glazed units should first be roughened by grinding or sandblasting.

For unspoiled new clay or concrete masonry surfaces, wire brushing proved to be adequate to remove any loose particles or dust. The surface preparation of older clay or concrete masonry structural members, however, may require more intrusive techniques such as water blasting, grinding, or wire brushing with power tools. Concrete masonry units may be lightly sandblasted (Hamoush et al. 2001), but this method should be used with caution for clay units.

10.4.4 *Installation of FRP system*—For the installation of an FRP system, the recommendations of the system manufacturer should be precisely followed, with deviations requiring approval of the manufacturer and the design engineer. Typical masonry details such as weep holes, concrete masonry control joints, and clay masonry expansion joints should be maintained in their original condition. For example, no resin or FRP laminate should cover weep holes, and no FRP material should bridge masonry movement joints.

CHAPTER 11—DURABILITY OF FRP USED IN CONCRETE

A significant design issue for FRP composites is the consideration of overall durability of these materials, especially as related to their capacity for sustained performance under harsh and changing environmental conditions under load.

Although FRP composites have been successfully used in the automotive, marine, industrial, and aerospace sectors, critical differences exist in loading, environmental exposure, and the types of materials and processes used in these applications as compared with those likely to be used in civil infrastructure applications. Anecdotal evidence provides substantial reason to believe that, if appropriately designed and fabricated, these systems can provide longer lifetimes and lower maintenance than equivalent structures fabricated from conventional materials. Experimental data on durability, however, is sparse, not well documented, and not readily available. Additionally, some evidence has been found of rapid degradation of specific types of FRP composites exposed to certain environmental conditions found in civil engineering.

11.1—Definition of durability

In the context of this report, the durability of FRP or structural members using FRP is their ability to resist cracking, oxidation, chemical degradation, delamination, wear, fatigue, the effects of foreign object damage, or a combination of these for a specified period of time, under the appropriate load conditions, and under specified environmental conditions.

11.2—Durability of FRP composites

11.2.1 *Materials*—Without the protection of the appropriate resin, E-glass fibers are the most susceptible to degradation due to moisture and alkalinity. Similarly, aramid fibers are resistant to abrasion and impact, but show a propensity to creep, absorb moisture, and degrade under ultraviolet exposure. Carbon fibers are relatively inert to the environment. In composite design, however, the individual fibers are encapsulated in a suitable resin system to form the composite. Thus, the fibers are protected from the environment by the resin. The durability of the resin system is dependent on several factors, including the resin components and proportions as well as curing time and conditions. The composites industry has many resin systems that are designed for specific end-use applications and should be chosen based on the mechanical, physical, chemical, electrical, or other considerations in the operating environment. Properties of materials typically used in FRP composites used with concrete can be found Chapters 4 and 5 of this document or in 440.1R and 440.2R.

Both the fiber system and the resin system should be chosen based on requirements of structural performance, constructibility, and durability. Different fibers and resin systems provide differing degrees of resistance to environmental conditions such as moisture, alkaline solution, UV radiation, or extreme temperatures. Thus, the constituents need to be chosen based on both performance and durability requirements. The processing method and quality assurance and quality control used during processing are important predicators of quality and durability of the fabricated composite component. A summary of the significant factors affecting durability was presented by Porter at a SAMPE Symposium (Porter 1999).

11.2.2 *Overview of ASCE/CERF document*—The ASCE/CERF (Karbhari et al. 2003) document "Durability Gap Analysis for Fiber Reinforced Polymer Composites in Civil Infrastructure" provides the results of a gap analysis identifying critical areas in which data are needed. In this document, the use of FRP composites in the form of reinforcing bars, external reinforcement of concrete structures, seismic retrofit of concrete and masonry structures, replacement and new bridge deck systems, and wall panels and profiles were identified to be of the maximum interest. Therefore, the evaluation of durability issues with these applications was assigned the highest priority in the report. The following durability issues were addressed: moisture or aqueous solutions, alkaline environment, thermal effects, creep and relaxation, fatigue, and UV radiation. An analysis of the existing data was performed to rank the importance and availability of data in each of the topic areas.

11.2.3 *Environments*—The intent of this section is to outline the environments that FRP composites used with concrete and masonry are likely to encounter. Each section gives a brief summary of the effect that each of these environments has on the constituents usually found in FRP composites used with concrete.

11.2.3.1 *Moisture (water and salt solution)*—All resins absorb moisture, with the percentage of moisture sorption depending on the resin structure, degree of cure, and temperature. The two primary effects of moisture uptake are plasticization and a reduction in glass transition temperature. In general, moisture effects over the short term cause more pronounced degradation in strength as opposed to stiffness of the composite. Salt solutions can cause blistering due to osmotic effects. In some cases, moisture has been observed to wick along the fiber-matrix interface, resulting in a loss of structural integrity. Moisture can also affect the fracture toughness of FRP composites, with reported results being somewhat contradictory (Karbhari et al. 2003).

In the case of glass fibers, degradation is initiated by moisture-extracting ions from the fiber. Fibers need protection of the resin to avoid such deterioration. Aramid fibers absorb moisture, causing loss of transverse and compressive strength (Karbhari et al. 2003).

11.2.3.2 *Chemical solutions*—In most cases, the effect of chemical solutions is on the resin system, with the absorption following a diffusion-based process similar to that of water (Karbhari et al. 2003). The presence of specific salts or other chemicals in the solution can accelerate deterioration in the presence of inappropriate resin systems. A large number of specialty resin systems are available that are resistant to varying levels of chemical attack and exposure.

11.2.3.3 *Alkaline environment*—Alkaline solutions, such as the pore water of concrete, have a high pH and a high concentration of alkali ions (Neville 1995). Carbon fibers are resistant to alkaline solutions. Resin damage via alkali attack is generally more severe than that due to moisture. E-glass fiber systems should be properly designed and fabricated with the appropriate resin system to protect the reinforcement from alkali attack. Alkali-resistant glass fibers are available and can decrease the rate of deterioration substantially. A significant amount of extremely high pH testing has been conducted at Iowa State University, resulting in several investigations, such as by Mehus (1995). These tests show that, under a loading resulting in a stress of 40% of ultimate, some FRP reinforcing bars can fail while submerged in a solution with a pH of 12.8. Tests from the same source also show that the reinforcing bar that had been submerged in the high pH solution lost as much as 60% of its tensile strength. Later tests from the same location indicated that when improved resins were used, the results improved significantly; thus, durable resins need to be used for these environments (Boris and Porter 1999).

11.2.3.4 *Extreme temperature and thermal cycling*—The primary effects of temperature are on the viscoelastic response of the resin, and hence, of the composite. As temperature increases, the modulus of the resin will decrease. If the temperature exceeds the glass transition temperature T_g, FRP composite performance will decrease substantially. Thermal cycling below T_g generally does not cause deleterious effects, although extended thermal cycling of brittle resin systems can result in microcrack formation (Karbhari et al. 2003).

The coefficients of thermal expansion (CTEs) for FRP composites are generally quite different from those of steel and concrete. For the case of glass FRPs, the CTE is generally higher than that of steel and concrete. For carbon and aramid FRPs, the CTE is generally lower than that of steel and concrete in the direction of fibers (Hollaway and Leeming 1999). The CTE will vary considerably with fiber and resin type as well as fiber orientation and constituent volume fractions. The difference in CTE should be considered when composites are used in direct combination with steel and concrete systems.

11.2.3.5 *Low temperature and freezing and thawing*—In general, low temperature and freezing-and-thawing exposures do not affect fibers, although they can affect the resin and the fiber-resin interface. Polymeric resin systems are known to embrittle, resulting in increased strength and stiffness under sub-zero (but noncryogenic) conditions (Chawla 1998). Freezing-and-thawing effects can be more severe due to moisture-initiated effects, causing microcrack growth and coalescence because of cycling. The presence of road salts in wet conditions with subsequent freezing and thawing can cause microcrack formation and gradual degradation due to crystal formation and increased salt concentration.

11.2.3.6 *Creep and relaxation*—Polymer resins generally exhibit creep and relaxation behavior. The addition of fibers increases the creep resistance of the resins. Consequently, creep and relaxation behavior are more pronounced when load is applied transverse to fibers or when the composite has a low fiber volume fraction (Karbhari et al. 2003). Typically, thermosetting resins (unsaturated polyesters, vinylesters, epoxies, and phenolics) are more resistant to creep than are thermoplastics (polypropylene, nylons, and polycarbonates).

Carbon fibers are the least susceptible to creep rupture; aramid fibers are moderately susceptible; and glass fibers are most susceptible to creep rupture (Hollaway and Leeming 1999). Extrapolations of short-term creep data to longer service lifetimes in room-temperature air suggest rupture strengths of 29 to 55%, 47 to 66%, and 79 to 93% of the initial strength for essentially unidirectional GFRP, AFRP, and CFRP materials, respectively (Yamaguchi et al. 1997; Ando et al. 1997; Seki et al. 1997; Greenwood 2002).

E-glass fibers are also susceptible to environmental stress corrosion cracking. This is a delayed brittle fracture effect that is caused by synergism between stress and the environment (Jones 1999).

Stress limits for FRP composites under sustained load to avoid premature failure due to stress rupture can be found in ACI 440.1R and 440.2R.

11.2.3.7 *Fatigue*—The fatigue performance of FRP composite materials depends on the matrix composition and, to some extent, on the type of fiber (Curtis 1989). The individual fibers within unidirectional composites have few defects, and are consequently resistant to crack initiation. Additionally, any crack that does form travels through the matrix and is not transmitted through adjacent fibers. These toughness and crack-arresting properties contribute to the good fatigue performance of FRP materials.

11.2.3.8 *UV radiation*—Polymeric materials undergo degradation when exposed to UV radiation between 290 and 400 nm due to dissociation of chemical bonds (Karbhari et al. 2003). The subsequent reaction with oxygen can lead to oxidation, chain scission, or cross-linking. In general, effects are rarely severe in terms of mechanical performance, although some resins can show significant embrittlement and surface erosion. The most deleterious effect of UV exposure is generally not the UV-related damage, which is surface limited, but the potential for increased penetration of moisture and other agents via the damaged region. In some cases, degradation at the surface has been found to affect mechanical properties disproportionately because flaws can serve as stress concentrations (Chawla 1998). FRP composites can be protected from UV-related degradation with appropriate additives in the resin, appropriate coatings, or both.

11.3—Internal reinforcement

11.3.1 *Introduction*—This section covers the degradation process and mechanisms affecting hygro-thermo-mechanical properties of FRP reinforcing rods under exposure to alkaline environments, alternate wet and dry cycles (in corrosive and noncorrosive mediums), freezing-and-thawing conditions, temperature and humidity variations, and loads (creep and fatigue).

11.3.2 *Moisture*—Sen et al. (1998) conducted a 45-month study on the long-term performance of AFRP and CFRP pretensioned elements used to reinforce piles driven in tidal waters, based on destructive tests. Results indicated that bond degradation adversely affected the ultimate capacity of AFRP-reinforced piles, but the CFRP-reinforced piles were largely unaffected.

11.3.3 *Alkaline environment*—The reaction of FRP composites to alkaline conditions in concrete is a major design consideration. The internal concrete environment initially has high alkalinity, with the pH between 12 and 13. This alkaline environment can have an effect on fibers, such as glass and aramid, as discussed in the previous section.

Although an appropriate resin matrix (vinylester, epoxy) provides a high level of protection to fibers from this degradation, migration of high pH solutions and alkali salts through the resin (at voids, cracks, and interface between the fiber and matrix) to the fiber surface is possible (ACI 440.1R). In addition, the application of special surface coatings or fillers, selection of suitable chemistry, and improvement in manufacturing processes can all improve the durability of FRP composite reinforcement in an alkaline environment. For instance, Shah et al. (2002) observed a 50% reduction in the room-temperature diffusion rate of water into vinylester resin filled with montmorillonite nano-clay, although the equilibrium moisture content increased compared with the same resin without filler.

Aqueous solutions with high pH are known to degrade the tensile strength and stiffness of GFRP bars (Porter and Barnes 1998; Rostasy 1997; Sen et al. 1998; Takewaka and Khin 1996; Sheard et al. 1997; GangaRao and Vijay 1997). On the other hand, Devalapura et al. (1998) concluded that GFRP reinforcement exposed to both alkaline and acidic environments retained significant load-bearing capacities for extended life cycles under conditions harsher than expected in field service. Al-Dulaijan et al. (1996) detected a considerable reduction in bond strength of bars immersed in a high-pH solution for 28 days. This reduction appeared to be a result of degradation of the resin.

Arockiasamy and Sandepudi (1994) concluded from experimental studies that the Young's modulus of CFRP composite reinforcement in a combined seawater and alkaline solution with sustained tension was reduced by approximately 12% over exposure periods from 3 to 9 months. The same exposure, however, did not affect the ultimate strength.

In 2004, ISIS Canada approved a project to study the performance of GFRP reinforcement that has been used in many demonstration concrete structures across Canada (Mufti et al. 2005). The objective of the study was to provide the engineering community with the results of the performance of GFRP reinforcing bars that have been exposed to a concrete environment in built structures and to calibrate the Canadian Highway Bridge Design Code (CHBDC) performance factors on the GFRP reinforcement. Core specimens of GFRP reinforcement were collected from five field demonstration projects across Canada. Analytical methods, such as scanning electron microscopy and energy-dispersive x-ray, optical microscopy, differential scanning calorimetry, and infrared spectroscopy, were used to determine the degradation of GFRP in concrete structures.

Based on the results of the aforementioned analyses described, Mufti et al. (2005) found no visible degradation of the GFRP reinforcement (rods and grids) in the concrete environment in real engineering structures exposed to natural environmental conditions for 5 to 8 years. The results from scanning electron microscopy and x-ray analyses suggest no degradation of the GFRP reinforcement materials in the demonstration concrete structures. The x-ray analyses indicate no alkali ingress in the GFRP reinforcement from the concrete pore solution. The conclusion of the project was that the GFRP reinforcement is durable and highly compatible with the concrete material. Also, the team concluded that the CHBDC is conservative; therefore, the material performance factors of the GFRP bars should be increased. This change has been incorporated in the new addendum for the CHBDC.

11.3.4 *Low temperature and freezing and thawing*—At low temperatures, complex residual stresses arise within FRP composites as a result of matrix stiffening and mismatch of CTEs of matrix and resin as well as FRP and concrete (Chawla 1998). Residual stresses can cause microcracks in the matrix and fiber-to-matrix interface, which can grow under low-temperature thermal cycling and coalesce to form transverse matrix cracks and debonding between the fibers and the matrix.

In general, the reported literature shows that unidirectional tensile strength decreases when exposed to temperatures between –10 to –40 °C (14 to –40 °F), whereas the off-axis and transverse strengths may increase due to matrix hardening. Increasing freezing-and-thawing cycles have been shown to accentuate residual stresses, resulting in increased severity and density of cracks. An apparent increase in matrix brittleness and decrease in tensile strength has also been reported (Lord and Dutta 1988).

Mashima and Iwamoto (1993) determined that bond strength of GFRP and CFRP rods was not influenced by freezing and thawing, but AFRP (both braided and coiled) rods showed a gradual reduction of bond strength up to about 20% with continued freezing and thawing.

11.3.5 *Temperature*—Researchers reported that extremely elevated temperatures (above the glass transition temperature) have a detrimental effect on bond, probably because of lower shear stiffness in the FRP. The GFRP achieved the highest residual bond strength, while the AFRP achieved the lowest, but slip increased dramatically with increases in temperature in all the materials (Nanni et al. 1995; Katz et al. 1999).

11.3.6 *Creep and relaxation*—Odagiri et al. (1997) investigated relaxation characteristics of 6 and 7.4 mm (0.24 and 0.29 in.) diameter AFRP tendons with anchorages. Overall, the relaxation rates for AFRP rods were found to be approximately 11% at 1000 hours, and 15% at 17,700 hours (2 years).

Creep rupture is another important concern when FRP reinforcing bars are subjected to long-term loading. Creep-rupture is the tensile rupture of a material subjected to sustained high stress levels over a period of time. The creep-rupture behavior of 3 mm (0.12 in.) diameter FRP composite circular rods of glass, aramid, and carbon fiber was evaluated by Dolan et al. (1997) by nonaccelerated techniques. Aramid specimens were more susceptible to stress concentrations at anchor points. Substantial decay was found in the long-term resistance of the glass tendons, especially when in direct contact with cementitious material.

Nikurunziza et al. (2002) conducted stress rupture tests on GFRP bars. After 60 days of exposure, the loss of tensile strength was 4% for a water exposure and 11% for alkaline exposure at temperatures of 65 to 75 °C (149 to 167 ºF).

Almusallam et al. (2002) conducted stress rupture tests on GFRP bars embedded in concrete beams and exposed to water. The strength losses were less than 5% for unstressed specimens, but as high as 30% for stressed specimens.

Den Uijl (1991) predicted the long-term performance of aramid bars in an alkaline environment using temperature as the varying parameter. The tests indicated that the time to failure under constant load at 60 °C (140 °F) was between 10 and 15 times shorter than when tested at 20 °C (68 °F). Ando et al. (1997) found similar results.

Sheard et al. (1997) studied durability of FRP-reinforced concrete in aggressive alkali and wet and dry environments at different temperatures and stress levels. Based on the test results, the authors suggested a 100-year life threshold stress limit of about 25% for GFRP, 50% for AFRP, and 75% for CFRP.

Creep experiments performed by Apinis et. al. (2000) showed that carbon fibers do not creep at strain levels as high as 0.69% for 17,700 hours. Aramid fibers, starting at a strain level of 1.38%, creep by 69% after 16,800 hours. Glass fibers creep by 5% from a starting strain level of 0.78% after 16,600 hours. Experiments and analysis performed by Tamužs et. al. (2001) concluded that hybrid composites, CFRP+AFRP and CFRP+GFRP, have a considerably higher creep resistance compared with pure aramid and glass composites. If brittle components such as carbon fibers are used in the hybrid composite, however, a certain risk of overloading the component exists due to the large creep of aramid fibers. After the breakage of carbon fibers, the creep will accelerate and may lead to the failure of the whole composite if the load level for the remaining fibers is high.

11.3.7 *Fatigue*—As described in Section 11.2.3.7, FRP composites generally have very good resistance to fatigue. In the case of internal reinforcement, fatigue loading is more likely to affect the concrete and the bond between the concrete and the FRP reinforcement than the FRP itself. It should, however, be noted that cracking in concrete could cause abrasion related damage to the FRP surface, which could result in higher environmental exposure to moisture and solutions.

11.3.8 *UV exposure*—In the case of internal FRP reinforcement, there is no direct exposure of the FRP to UV and hence this is not of concern except through any deterioration that UV may cause to concrete at the systems level.

11.4—External reinforcement

This section is devoted to the performance of external FRP reinforcement for concrete that has been adhesively bonded to the surface of the concrete. Included in this classification are both confinement systems such as column wrapping and flexural and shear reinforcement. External reinforcement is likely to see a more varied environmental exposure than that of internal reinforcement including moisture cycling, chemical solutions, and UV radiation.

In adhesively bonded FRP systems, one face of the material is adhered to the concrete and one is exposed to the environment. Consequently, the exposure conditions (related to moisture) for the FRP composite are affected simultaneously by the local environment and the underlying concrete. Highly porous concrete will readily transmit any available moisture to the bond line. Accumulation of salts at the bond line may cause large pore pressures to develop, thus reducing the bond capacity.

Bonded FRP systems that are not anchored rely entirely on the bond between the FRP composite and the underlying concrete and masonry. The nature of the bond between the substrate and FRP composite is tied to the FRP strength and durability as well as that of the concrete.

Failure of the external systems is generally governed by the tensile strength of the concrete. Also, cracking in the beam (due to flexure, shear, or other causes) will have a profound effect on the failure load of beams plated with FRP (Teng et al. 2002). The consequence of this key structural aspect of external FRP reinforcement is that the durability of the system may have little to do with the durability of the components of the FRP reinforcement, and have much to do with the durability of the underlying substrate.

11.4.1 *Moisture (water and salt solution)*—Prolonged exposure of FRP composites to moisture can cause problems. Moisture can gain access to the resin and fibers from the exposed face. In addition, cracks and porosity in the underlying concrete or masonry can allow moisture transport by diffusion and capillary action. Thus, both faces of externally applied composites may be exposed to moisture and salt solution.

Karbhari et al. (1997) studied the effect of water, seawater, and freezing and thawing on bonded FRP systems using a peel test. They found that the nature of the bond changed and reduced in strength when exposed to water. Wan et al. (2003) and Wan (2002) found similar results even though the FRP was not directly exposed to moisture. In the same study, the presence of moisture on the FRP substrate during FRP application was also investigated. Once again, bond behavior deteriorated with increased moisture present during application.

Chajes et al. (1995) found similar results for 48 small-scale beams strengthened with glass, aramid, or carbon fiber composites. The beams were subjected to wetting and drying or freezing and thawing, and then tested in flexure. The results indicated that the beams strengthened with carbon fibers maintained their strength better than glass or aramid fiber composites. They also found that the wetting and drying caused slightly greater degradation than that of freezing and thawing. They also found that the exposure changed the failure mode to debonding, indicating a loss in adhesion due to exposure.

Moisture damage can occur either in the adhesive at the bond line or just below the bond line in the concrete and masonry substrate (cracking). Beaudoin et al. (1998) tested beams that had been strengthened with CFRP laminates and then subjected to moisture cycling. Although there was strength reduction noted in one series, the loss of capacity was attributed to the adhesive and not to failure of the substrate.

Murthy et al. (2002) conducted environmental tests combined with sustained loading and found significant decreases in the bond capacity.

In summary, it appears that long-term exposure of externally bonded FRP composites to moisture can cause deterioration of the adhesion between the composite and concrete or masonry substrate. Furthermore, the bond may initially appear adequate, and yet deteriorate with time and exposure to moisture, freezing and thawing, or both.

11.4.2 *Alkaline environment*—Concrete has a pore water pH level as high as 13.5. Research has shown that alkaline solutions and ions can cause severe degradation to bare glass fibers and some polymer systems (Karbhari et al. 2003). Resins are known to be more or less susceptible to alkali attacks because alkaline solutions can accelerate the degradation of bond and of resins themselves, especially if they are not fully cured. An important role of the resin, in addition to transferring load between the fibers, is to protect the fibers against alkali and other harmful agents.

In addition, a number of FRP systems may be in contact with soil, which could also have a high alkaline content. Thus, the determination of durability of FRP composite systems in contact with alkalis is essential. Although a number of these effects could be in the form of solutions and could conceivably be considered under the first topic, this condition is considered separately due to its relationship to concrete.

Alkali environments can also have adverse effects on the fibers, with glass fibers being the most susceptible to damage. Aramid fibers are generally considered more resistant than glass to alkali attacks. Carbon fibers are noted to have high resistance to alkali attack to the point of being almost unaffected (Meier 1995).

11.4.3 *Extreme temperature and thermal cycling*—The effects of extreme temperature and thermal cycling on FRP materials in general is discussed in Section 11.2.3.4. An additional concern for externally bonded FRP is the effect on the bond of the FRP to the substrate. Not only may the adhesive layer be affected by temperature, but a mismatch of CTE between the FRP, adhesive, and substrate may result in unexpected thermally induced stresses. In general, stresses due to differential thermal behavior are not likely to be significant for GFRP, with a CTE relatively close to that of concrete and steel. CFRP and AFRP, on the other hand, need more consideration because their CTE values differ significantly from those of concrete and steel.

11.4.4 *Freezing and thawing*—The effects of freezing and thawing on FRP materials in general is discussed in Section 11.2.3.5. Again, the bond to the substrate may also be affected by freezing and thawing. Experience has shown that voids—regions between the concrete and FRP not completely filled with adhesive and or resin—exist in most external FRP applications. The expansive nature of most resin systems tends to minimize these voids, but poor surface preparation, latent material, or moisture presence during FRP application, among other factors, can create significant voids along the FRP-substrate interface. If liquid water can get to these voids and freeze, a lensing effect is likely. The interface region will deteriorate, and the voids will grow with continued freezing-and-thawing cycles. Good surface preparation techniques and quality-control procedures should minimize this concern. Some specific research on freezing-and-thawing effects is summarized in this section.

Karbhari and Eckel (1993) tested FRP-wrapped cylinders at a low temperature (–18°C [0 ºF]) and found increased brittleness of FRP fibers at a low temperature. Soudki and Green (1996) found good performance of CFRP-strengthened columns when subjected to up to 200 cycles of freezing and thawing consisting of 16 hours of freezing (–18 °C [0 °F]) followed by 8 hours of thawing in a water bath (+15 °C [59 °F]). Toutanji and Balaguru (1998) exposed FRP-wrapped concrete cylinders to 300 freezing-and-thawing cycles. They found some deterioration due to freezing and thawing with CFRP performing better than GFRP.

Chajes et al. (1995) tested FRP-strengthened concrete beams exposed to 100 freezing-and-thawing cycles. They found strength losses of 21% for CFRP and 27% for GFRP. In similar tests, Bisby and Green (2002) and Green et al. (2003) found little strength loss and no noticeable bond deterioration with FRP-strengthened beams exposed to up to 200 cycles of freezing and thawing. Lopez et al. (1999) tested small-scale beams strengthened with two types of CFRP and subsequently exposed to up to 300 freezing-and-thawing cycles. Their results showed, for one type of FRP, that precracking of the beams before strengthening could result in significant deterioration (up to 40% loss in strength) due to freezing and thawing. This result indicates that the combination of load and cold region exposure may be more severe than cold region exposure alone.

Arockiasamy and Thayer (1998) and Lopez et al. (2003) considered fatigue loading of CFRP-strengthened beams at low temperature and found no observable difference in behavior caused by the exposure to low temperature.

11.4.5 *Creep*—Most externally bonded FRP systems are intended to strengthen an existing structure. Consequently, the level of sustained load carried by the FRP will likely be minimal. As long as the sustained load is kept below the limits prescribed in ACI 440.2R, the remaining capacity of the FRP system will still be available to resist live load. If, however, a change of use is proposed for a structure previously strengthened with FRP, then creep rupture limits should be re-examined as a potential limiting factor.

Plevris and Triantafillou (1994) analytically and experimentally studied the time-dependent behavior of concrete beams strengthened with FRP plates. Three 1.5 m (4.9 ft) span beams were tested under sustained load. The analytical model was based on the age-adjusted effective-modulus method for concrete. Their model predicted the test results of deflection to within 15%. From parametric studies, they found that CFRP and GFRP laminates bonded to concrete beams improved the long-term behavior of the strengthened beams by controlling creep strain. AFRP was not as effective because of significant creeping of the AFRP itself.

11.4.6 *Fatigue*—Relatively little investigation of the fatigue behavior of plain or reinforced concrete beams with bonded FRP composites has been conducted. Meier et al. (1993), Heffernan (1997), Barnes and Mays (1999), Shahawy and Beitelman (1999), Papakonstantinou et al. (2001), Masoud et al. (2001), Breña et al. (2002), Aidoo et al. (2004), and Quattlebaum et al. (2005) all report fatigue testing of concrete beams with bonded FRP. In all cases, the eventual failure was fatigue fracture of the longitudinal steel reinforcement, which was similar to that of the unstrengthened companion specimens. Increases in fatigue life observed in these tests were all attributed to the applied FRP reducing the stress range in the existing steel reinforcement.

In some studies, some extent of debonding of the FRP material from the concrete substrate was observed. Once debonding occurred, stress carried by the FRP is redistributed back to the internal reinforcing steel. Thus, improvement in fatigue performance was only attained as long as the FRP was adequately bonded to the concrete (Aidoo et al. 2004).

Meier (1995) performed fatigue tests on a 6.0 m (19.7 ft) T-beam strengthened with a CFRP laminate. The fatigue loading caused stresses in the FRP to vary from 102 to 210 MPa (14.8 to 30.5 ksi). The laminate survived 14 million cycles and failed in shear peeling after failure of the internal reinforcing steel.

Muszynski and Sierakowski (1996) carried out fatigue tests on reinforced concrete beams with external CFRP reinforcement. Two million cycles of load were applied, and fatigue damage was measured using two nondestructive methods. The endurance limit of beams reinforced with CFRP was greater than 250% of their static flexural strength, which showed the excellent fatigue properties of CFRP strengthening.

Toutanji et al. (2006) conducted fatigue tests on reinforced concrete beams externally strengthened with CFRP reinforcement. The beams, with a span of approximately 2 m (6.5 ft), were subjected to 2 million cycles. Results show that, due to FRP strengthening, the fatigue strength increased by 55%.

Larson et al. (2005) tested five CFRP-strengthened pretensioned prestressed concrete T-girders to examine the effect of increased live loads on fatigue. CFRP strengthening was designed to allow for stress ranges of 124 and 248 MPa (18 and 36 ksi) in the steel strands under the upgraded live load levels. They found that the girders successfully sustained 1 million fatigue cycles with no loss of ultimate capacity, and 3 million cycles with 19% loss of ultimate capacity.

11.4.7 *UV exposure*—The effects of ultraviolet exposure on externally bonded FRP materials in general is discussed in Section 11.2.3.8. Generally, when UV exposure is a concern, an appropriate admixture in the resin or barrier coating may be specified.

11.4.8 *Condition of existing structural members*—In some cases, FRP reinforcement is applied externally to replace strength that has been lost due to deterioration of the existing structure. Common elements include corrosion-damaged piers, piles, and beams. Corrosion damage may be caused by chlorides available from either a marine environment or deicing salts. ACI 440.2R clearly addresses "corrosion-related damage" and that the "cause of corrosion should be addressed" before application of the FRP system. This statement implies that the chloride-contaminated concrete should be replaced with sound concrete or other repair material before application of an FRP system. ACI 440.2R further indicates that FRP systems should not be applied over concrete that is suspected of containing corroded reinforcement.

For bonded FRP composites to remain effective over the remaining service life of the structure, the underlying concrete substrate should remain sound and capable of transferring load to the externally applied composite. This is particularly important in the case of bond-critical applications (flexural and shear strengthening).

Placement of any externally bonded FRP system will greatly affect the movement of moisture both into and out of the underlying concrete. For interior applications, this can serve to enhance the overall durability of the structure by preventing the ingress of moisture and other deleterious chemicals. If, however, the structure is exposed to a humid or wet environment, then the FRP can actually trap unwanted moisture inside the structure and cause damage to the underlying concrete (fib 2001). Before the placement of the FRP repair system, the existing moisture flow characteristics of the structure should be evaluated and then maintained. For example, external shear reinforcement can be applied in thin strips rather than full coverage to allow moisture to move freely across the concrete surface.

Another potentially damaging effect that an FRP system can have on a structure relates to corrosion and galvanic action. Aramid and glass fibers are considered strong insulators, and do not allow for movement of electrical current, but carbon is electrically conductive and can potentially facilitate the corrosion process. As a result, carbon fiber-based FRP systems should never be placed in direct contact with reinforcing steel (fib 2001).

Notwithstanding these comments, recent research has suggested that, under the right conditions, FRP wraps can slow, or even stop, steel reinforcement corrosion in chloride-contaminated concrete columns (Debaiky 2002; Debaiky et al. 2002; Pantazopoulou et al. 2001; Teng et al. 2003; Sen 2003; Chauvin et al. 2000). Pantazopoulou et al. (2001) subjected small concrete columns containing steel reinforcement to accelerated corrosion conditions and then repaired them with GFRP wraps. The repaired specimens were then subjected to continued accelerated corrosion conditions, and there appeared to be some reduction in the corrosion rates with the application of the wraps.

Debaiky et al. (2002) conducted similar tests on small concrete cylinders; corrosion data were also taken, including half-cell potential and linear polarization. They found that under strict control and monitoring, CFRP wraps reduced the corrosion activity in the reinforcing steel. This protection did not vary with the number of layers, so it was concluded that the resin provided the protection (not the fibers). They also found that partial wrapping led to macrocell corrosion in the steel reinforcement at the extremities of the wrap. These results were also confirmed with tests on larger-scale columns (Debaiky 2002). The researchers recommended more research into the effect of partial coverage on full-scale columns. Teng et al. (2003) found similar results.

For FRP-wrapped beams undergoing active corrosion, improved structural performance may be exhibited by a combination of the following two mechanisms: 1) confinement of the concrete section, thereby lessening corrosion cracking and bond splitting cracks; and 2) increased structural resistance.

Sherwood and Soudki (1999) studied the effects of reinforcing bar corrosion on the behavior of CFRP-strengthened reinforced concrete beams. During corrosion, test results showed that the CFRP laminates adequately confined corrosion cracking up to only 6% corrosion. Beyond this level, extensive cracking and CFRP laminate delamination were observed. To provide better corrosion confinement, it was suggested to extend the transverse laminate up to the full height of the beam instead of half the beam height. Load test results indicated that the strengthened corroded specimen retained 92% of the control beam strength, while the unstrengthened corroded beam retained only 79% of the strength.

The bond enhancement of FRP repaired and concrete members with corroded reinforcing bars were investigated by Soudki and Sherwood (2003). Following corrosion, the specimens were tested by bar pullout to determine the bond strength versus slipping of the reinforcing bars. The bond strength of the reinforcing bars in the concrete prisms increased at low levels of corrosion, but decreased as the degree of corrosion increased. The failure mode was typically bond splitting in unwrapped specimens. Strengthened specimens exhibited increased bond strength; they also exhibited failure by bar pullout due to the confining effects of the CFRP strengthening.

Masoud et al. (2001) investigated the structural behavior of corroded beams strengthened by CFRP sheets. The results showed that the use of CFRP sheets for strengthening reinforced concrete beams experiencing steel reinforcement corrosion was an efficient technique that could maintain the structural integrity and enhance the structural behavior of such beams. The fatigue life of the CFRP strengthened-corroded specimens was increased within a range of 2.5 to 6.0 times that of a similar unstrengthened-corroded specimen.

Although under strictly controlled conditions, FRP wraps and jackets have been demonstrated to mitigate corrosion damage; the opposite has also been observed to be true (Pantazopoulou et al. 2001). In particular, some field studies have shown accelerated corrosion rates under a wrap or jacket. Until further evidence becomes available that bonded FRP composites will effectively slow corrosion rates of existing steel reinforcement, it is advisable to repair existing corrosion damage and mitigate the underlying cause of the corrosion. An alternative would be to monitor the continuing corrosion of the wrapped column.

11.5—Structurally integrated stay-in-place (SIP) forms

Stay-in-place (SIP) FRP forms have shown promise in a variety of applications, such as piles (Mirmiran 1995; Saafi et al. 1999), girders (Zhao 1999), and bridge decks (Thompson and Iyer 2001). This type of construction has its specific durability issues, primarily because FRP is exposed and acts as the first line of defense against environmental factors. Moisture absorption, UV degradation, and freezing and thawing are the most critical issues with these systems. The durability issues are thus similar to those for externally bonded FRP.

Durability studies on SIP systems, however, are rare and limited. Lopez-Anido et al. (1999) studied the fatigue of SIP deck systems on steel girders at high temperatures, and found no significant damage after 1 million fatigue cycles at high temperature.

Fam et al. (2002) studied the effect of moisture absorption on the behavior of concrete-filled GFRP tubes of two different laminate structures submerged in fresh water for a period of up to 1 year. The ends of specimens were not sealed; therefore, moisture was allowed to penetrate the specimens from both ends. The study showed strength degradations on the order of 13 to 20% for the two different laminate structures within 1 year. The rate of degradation was reduced over time, and strength recovery was observed for one of the specimens after 1 year. The authors noted that the gain in strength of submerged concrete might have offset the loss of strength in the FRP tube. Additional studies, however, are needed to verify these findings.

CHAPTER 12—FIRE AND BLAST EFFECTS

12.1—Introduction

Recent research into applications of FRP materials for reinforcement and strengthening of reinforced concrete infrastructure has highlighted the need for information in two specific and specialized research areas: fire and blast. This chapter discusses fire and blast effects on FRP systems for concrete separately in two sections.

12.2—Fire

To date, FRP reinforcement and strengthening systems have been used predominantly in structures either where fire resistance considerations are typically not critical factors in design or where the existing structure is capable without FRP on supporting design loads during a fire event. In cases where strict fire code regulations apply, FRP systems are not being used to their full potential, largely because of their unknown behavior in the event of fire. This gap in knowledge is a primary factor preventing the widespread application of FRP materials in buildings, and it has thus been recognized as a critical research need (Sorathia et al. 2001; Harries et al. 2003; Karbhari et al. 2003). This chapter presents a brief overview of the state of the art with respect to the high temperature and fire performance of FRP reinforcement strengthening and systems for concrete structures. More complete discussions are available elsewhere (Bisby et al. 2005a).

12.2.1 *Fire safety in structures*—Fire safety engineering is concerned with limiting the likelihood of loss of life, injury, and property damage in an unwanted fire (ASCE 1992; Buchanan 2001). Most current codes focus their attention on life safety objectives. Prevention of property loss is left largely to building owners. In general, life safety is protected by preventing fires, alerting people to the presence of a fire, and providing means of escape for an adequate period of time. Design for fire safety should also ensure that building occupants are not overcome by flames, heat, or smoke, that the fire is contained to as small an area as possible, and that the structure does not collapse.

Structural fire protection is concerned with providing adequate structural fire endurance (preventing collapse) and ensuring that fire barriers remain intact (compartmentalizing the fire). These goals are typically accomplished using proven structural materials and systems, such as reinforced concrete, masonry, or structural steel, whose ability to withstand fires has been demonstrated through full-scale standard fire tests, such as ASTM E 119 guidelines. Before new construction materials and systems, such as FRP, can be used in buildings, their ability to meet fire resistance criteria set out in building codes should be demonstrated.

12.2.2 *FRPs and fire*—While the structural performance of FRP reinforced or strengthened concrete members during fire has only recently begun to receive significant research attention, a relatively large amount of research has been performed to characterize the overall behavior of FRP materials when subjected to fire (Sorathia et al. 1992; Sorathia 2004; Davies et al. 2004). Because fire safety concerns not only structural performance, but also issues of fuel load, flame spread, smoke generation, and smoke toxicity, many factors have been studied with a view to FRP applications in various specific structural and nonstructural applications (mostly in marine, aerospace, and offshore conditions). Nevertheless, because a wide variety of different FRP products and formulations are currently available, properties can vary widely, and generalizations regarding their fire performance are difficult to make.

When thick FRPs are exposed to fire, pyrolysis of the polymer matrix near the exposed surface will occur, leading to the formation of a protective (insulating) char. The thickness of the char layer increases with increasing fire exposure and forms a thermal barrier that insulates the interior of the FRP component from the fire. The char eventually degrades after prolonged exposure to high temperatures (for example, above approximately 500 °C [932 °F] for a typical polyester resin), and beyond this point, only the fibers, which are much less sensitive to elevated temperatures, remain (Davies et al. 2004). Thus, compared with unreinforced polymers, thick FRPs have advantages with regard to their behavior in fire in that the noncombustible fibers displace polymer resin, making less fuel available for the fire. Also, when the outermost layers of a composite lose their resin because of combustion, the remaining fibers act as an insulating layer for the underlying material, thus providing thermal protection to the FRP (Sorathia et al. 1992).

The aforementioned advantages are not particularly beneficial when FRP materials are used as reinforcement or strengthening for concrete or masonry because charring is not effective with thin laminates or internal reinforcement. In internal FRP reinforcing applications, where the FRPs are protected against combustion and insulated by the concrete cover, loss of bond at temperatures exceeding the glass transition temperature of the polymer matrix is likely to be the critical factor during fire. Concerns associated with flame spread and smoke generation, while important, are obviously less critical for internal reinforcement because the concrete protects the bars. In typical strengthening applications involving externally bonded FRP materials, the FRPs are generally too thin for a protective char to form, and thus the bond between the substrate and the FRP would likely be lost very early in a fire, well before any protective char would have

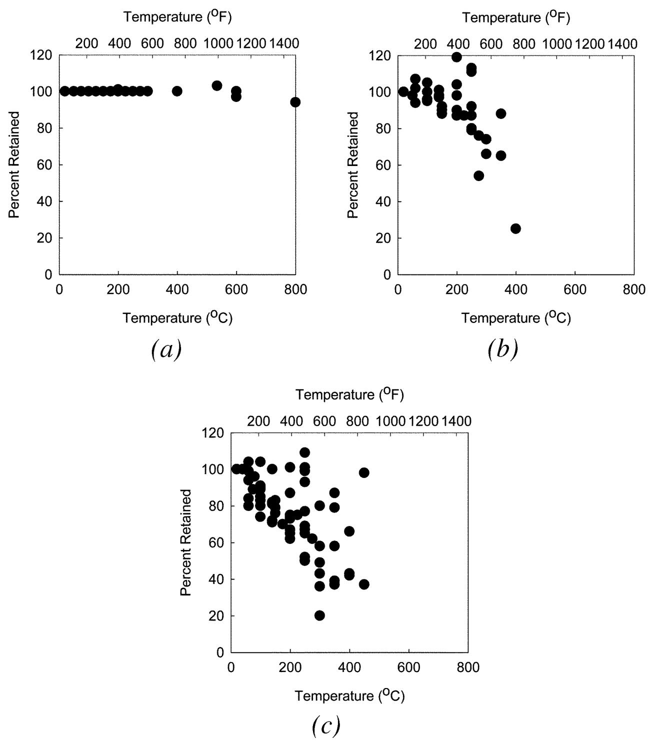


Fig. 12.1—Variation in tensile strength for: (a) carbon fibers; (b) unidirectional carbon/epoxy FRP; and (c) variation in tensile elastic modulus for unidirectional carbon/epoxy FRP. All plots show data assembled by Bisby (2003) and Williams (2004) from a variety of sources and on a variety of different fiber-matrix combinations, some of which may have incorporated elevated temperature cure matrix materials.

time to form. Flame spread, smoke generation, and smoke toxicity are clearly serious concerns that should be addressed when unprotected externally bonded FRP wraps are used in buildings.

The various high temperature properties are clearly important for the fire performance of FRP-reinforced or strengthened concrete and masonry members. The following sections present overviews of the strength, stiffness, bond, and combustion properties of FRP during fire.

12.2.2.1 *Strength and stiffness at elevated temperature*—Virtually all structural materials display reduced strength and stiffness properties at elevated temperature, and FRPs are no exception. Reductions in strength and stiffness at elevated temperature are important considerations when FRP materials are used to reinforce or strengthen structures. Relatively little information is available in the literature on the high temperature performance of FRP materials used in concrete or masonry reinforcing or strengthening applications. Data have been presented by several authors studying the variation in both strength and stiffness of various specific FRP formulations (used in applications ranging from aerospace to infrastructure).

The available test data have been assembled by Blontrock et al. (1999), Bisby (2003), and Williams (2004), and used to develop generalized semi-empirical models to describe strength and stiffness degradation with temperature for various types of FRP materials used in concrete reinforcing applications. As an example of the data available in the literature, Fig. 12.1 shows the assembled data for the variation in strength and stiffness of carbon/epoxy FRPs with temperature. While the carbon fibers themselves are relatively insensitive to the effects of elevated temperatures (Fig. 12.1(a)), the strength and stiffness of the composites are significantly degraded at temperatures of more than about 200 °C (392 °F) (Fig. 12.1(b) and (c)). The aforementioned data is provided for the purposes of illustration only, and tests should be conducted on any specific FRP material being contemplated for use, because FRP formulations vary widely. Plots similar to Fig. 12.1 for glass/epoxy and aramid/epoxy FRPs have been presented previously by Bisby et al. (2005a).

12.2.2.2 *Bond properties at elevated temperature*—The bond between concrete or masonry and FRPs is a critical factor in the performance of most FRP-reinforcing or strengthening systems for concrete and masonry. The bond, which relies heavily on the mechanical (shear) properties of the polymer matrix or adhesive, can be expected to be severely reduced at temperatures exceeding the glass transition temperature T_g of the matrix or the adhesive. Indeed, for the case of internal FRP reinforcement, testing by Katz et al. (1999) has shown that bond strength reductions in the order of 80 to 90% can be expected at temperatures between 100 and 200 °C (212 and 392 °F) (that is, in the range of T_g). Very little information is available on the post-heating residual strength of the bond between FRP bars and concrete, and further research is needed in this area.

Essentially no information is currently available on the specific behavior of the bond between unprotected externally bonded FRP materials and concrete or masonry at high temperature. The bond will likely be lost very quickly under fire exposure, because externally bonded systems are typically very thin (<2 mm [0.08 in.]), and the T_g of the resin will thus be exceeded almost instantaneously during a standard fire exposure. For insulated FRP systems, however, it is not clear exactly how long the bond between externally bonded FRPs and substrate can be maintained during fire. This is due in part to the type of concrete or masonry used, the effectiveness and thickness of the insulation, and various other factors. In addition, post-fire residual behavior of these systems is unknown. Further research is needed.

12.2.2.3 *Flame spread, smoke generation, and toxicity*—As stated previously, various important concerns, namely fuel load, flame spread, smoke generation, and smoke toxicity, are associated with combustion of organic polymer matrixes for FRPs. The likelihood of polymer ignition and combustion is dependent on the specific polymer formulation, and can vary widely from polymer to polymer (Nelson 1995). Sorathia et al. (1992) studied the smoke generation and toxicity characteristics of a variety of FRP formulations and demonstrated that thermoset resins commonly used in structural FRPs generate unacceptable quantities of smoke and have relatively poor flame spread characteristics. Furthermore, burning these materials generates varying quantities of potentially harmful gases such as carbon monoxide, hydrogen fluoride, hydrogen chloride, hydrogen sulfide, and hydrogen cyanide. While resin additives exist that can enhance the ability of polymer materials to resist ignition and combustion, these tend to unacceptably diminish mechanical properties and, in some cases, to increase the production of smoke upon eventual combustion (Nelson 1995).

When FRPs are contemplated for interior building applications, some means of preventing ignition and minimizing their involvement in fire is required. The ignition and flame spread characteristics of conventional FRP systems can be significantly improved by applying barrier treatments or coatings. Ceramic fabrics, ceramic or latex coatings, and intumescent coatings have been used to improve the ignition temperatures and flame spread characteristics of FRP systems (Sorathia et al. 1992; Apicella and Imbrogno 1999). These thin coatings, however, cannot be expected to maintain the structural effectiveness of FRP materials for significant durations of fire exposure. Thicker spray-applied and board insulation systems have also been used to protect fire-exposed FRPs, with varying degrees of success (Deuring 1994; Blontrock et al. 2000, 2001; Bisby et al. 2004a; Sorathia 2004; Williams 2004).

12.2.3 *Fire tests on FRP-reinforced or strengthened concrete*—The behavior in fire of conventional construction materials such as reinforced concrete, steel, timber, and masonry has been researched extensively, and is now reasonably well documented and understood. Data are available from numerous full-scale fire tests on conventional building systems and structural members, and, in many cases, complex numerical fire simulation models are also available to conduct parametric studies and consider specific fire scenarios on a case-by-case basis. This is not true for structural members that incorporate FRP materials, because relatively few fire tests have been reported in the literature (Deuring 1994; Blontrock et al. 2000, 2001; Bisby et al. 2004a; Williams 2004).

12.2.3.1 *FRP-reinforced concrete or FRP prestressed structural concrete members*—Several test programs have been reported in the literature that present data from fire tests on concrete members internally reinforced with FRP materials. The available data, however, are from tests using specific FRP-reinforcing systems, and the conclusions drawn are not necessarily applicable to other FRP materials or systems. Early work was performed in Japan to demonstrate the adequacy of specific FRP-reinforced curtain wall elements (Fujisaki et al. 1993; Nakagawa et al. 1993) and FRP-reinforced or prestressed beams and slabs (Okamoto et al. 1993; Sakashita 1997; NEFCOM Corp. 1998). More recently, fire tests and numerical studies on FRP-reinforced beams and slabs have been performed in North America (Kodur and Baingo 1998; Saafi 2002; Kodur and Bisby 2005) and the United Kingdom (Abassi and Hogg 2004). Essentially, it has been shown that FRP-reinforced concrete structures behave similarly to steel-reinforced concrete structures in fire, but that larger concrete covers are required to maintain FRP temperatures below acceptable limits. FRP bars appear to be more sensitive than steel bars to elevated temperature.

12.2.3.2 *FRP-strengthened structural concrete members*—Very few tests on concrete members reinforced with externally bonded FRP materials are available in the literature. Initial research in this area was performed in Europe by Deuring (1994), who demonstrated the extreme susceptibility of externally bonded FRP flexural strengthening systems to fire. Deuring also demonstrated the need for thermal insulation of the externally bonded FRPs to maintain their effectiveness during fire. Fire tests followed on FRP-strengthened concrete beams (Blontrock et al. 2000) and slabs (Blontrock et al. 2001) using a variety of supplemental fire insulation schemes. These tests confirmed the need for thermal insulation of the externally bonded FRP, but also demonstrated that the bond between the FRP and the concrete was lost in less than 1 hour even for well-insulated systems.

More recently, an extensive experimental and numerical research program aimed at investigating the fire performance of externally bonded FRP strengthening systems for concrete has been conducted through a joint research study between ISIS Canada, the National Research Council of Canada (NRC), Queen's University, and industry partners. To date, intermediate and full-scale fire tests have been performed on FRP-strengthened concrete columns (Bisby et al. 2004a), slabs (Williams et al. 2006), and beam-slab assemblies (Williams 2004). In addition, numerical models have been developed to simulate the fire behavior of these types of members (Bisby et al. 2005b), and these models have been used to conduct parametric studies (Bisby et al. 2004b). This research program has demonstrated that while FRP materials are sensitive to elevated temperatures, appropriately designed and insulated FRP-strengthened reinforced concrete beams, slabs, and columns are capable of achieving fire ratings of greater than 4 hours according to ASTM E 119. The specimens in these tests were designed in accordance with 440.2R. External coatings of gypsum-based or cementitious fire protection were applied (Kodur et al. 2006).

Preliminary studies have also been presented on the residual performance of FRP-confined concrete cylinders after exposure to elevated temperatures by Saafi and Romine (2002) and Cleary et al. (2003).

12.2.4 *Current treatment in codes and guidelines*—Thanks to an enormous research effort over the past 15 years, design codes and guidelines now exist for the application of FRP materials as internal (Rizkalla and Mufti 2001; CSA S6 and S806; ACI 440.3R and 440.4R) and externally bonded (Neale 2001; ACI 440.2R; CSA S806; ICC Evaluation Service 2003a,b; Concrete Society 2004) reinforcement for concrete structures. All of these codes and guidelines comment on the susceptibility of FRP materials to elevated temperatures. Because information on the fire performance of FRP-reinforced or strengthened concrete remains scarce, few of the existing design documents provide specific guidance or procedures for safe fire design of FRP systems. Instead, existing design documents tend to limit the use of FRP systems for reinforcing and strengthening concrete members.

With respect to fire-safe design of internal FRP reinforcement for concrete structures, CSA S806 provides guidance through a series of illustrative design charts (based on the work of Kodur and Baingo (1998)) that can be used to estimate the required concrete cover to achieve a specified fire resistance for FRP reinforcement. The difficulty in using these charts in

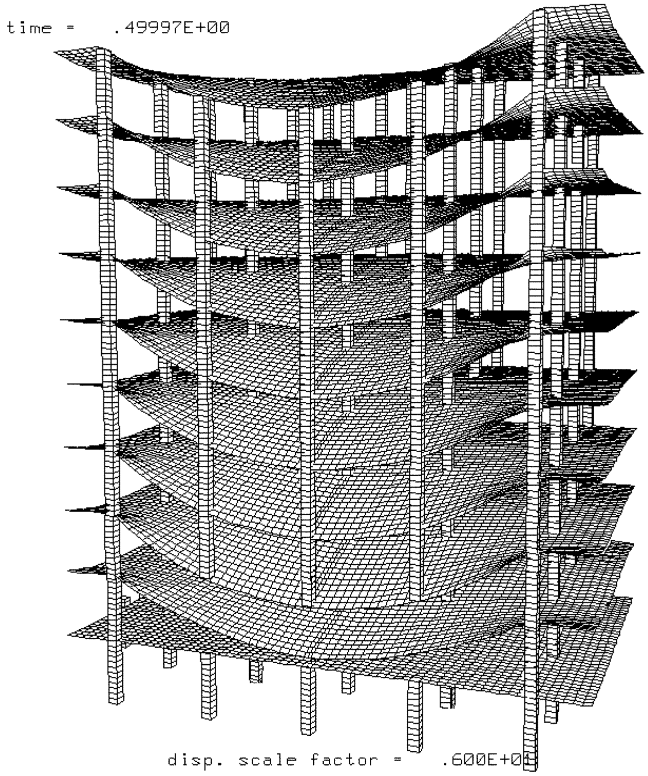


Fig. 12.2—Progressive collapse from loss of first-floor columns (Crawford et al. 1997).

practice is that they require that the critical temperature for the FRP reinforcement be known. The critical temperature for conventional steel reinforcement is 593 °C (1100 °F) in North America, which is the temperature at which it loses about 50% of its room temperature yield strength (ASCE 1992). Critical temperatures for most currently available FRP bars and tendons are not known, and additional research is required in this area. It is worth noting that the critical temperatures for FRP reinforcements will likely be governed not by tensile strength, but by bond strength, because bond strength is severely reduced at only mildly elevated temperatures (as discussed in Section 12.2.2.2).

With respect to fire-safe design of externally bonded FRP-strengthening systems for concrete, a common approach taken in existing codes and guidelines is to require that FRP materials be completely ignored during fire. This requirement is based on the assumption that unprotected FRPs will be rendered completely ineffective within minutes of fire exposure. Strengthening limits are also often imposed (ACI 440.2R).

A similar rationale is used by The Concrete Society's *Technical Report* TR55 (Concrete Society 2004). Because fire is an accidental load, *Technical Report 55* states that the safety factors used in design can be reduced and, in some cases, the FRP system can fail completely without risking failure of the structure.

The effect of the FRP composite system on fire-resistance construction is evaluated according to the appropriate section of the governing building code (ICC Evaluation Service, Inc. 2003b).

12.3—Blast effects

Structural components, such as columns, beams, and slabs, are vulnerable to the large dynamic loads from blasts. Such loads can lead to the shear failure of main load-bearing columns and result in the collapse of whole buildings, as in the case of the Oklahoma City Alfred P. Murrah building, resulting in a large number of casualties. In this building, approximately 87% of the people in the collapsed portion of the building died (153 out of 175); whereas only 5% of the people in the uncollapsed portion died (10 out of 186) (ASCE 1996). Thus, had the structure resisted the blast loads, the casualties could have been reduced significantly. Even if the columns survive, such loads can also lead to the disintegration of walls, creating secondary fragments that could cause fatalities in adjacent rooms. FRP composite materials can be used to reinforce concrete and masonry structures to prevent such disasters.

12.3.1 *Blast strengthening of reinforced concrete columns*—Numerical models can predict the shear failure of main load bearing columns, and the resulting progressive building collapse (Fig. 12.2) (Crawford et al. 1995, 1996, 1997). The models also predicted that composite wraps, or steel jackets, could increase the shear capacity of the column, thus preventing column failure and building collapse. This concept had also been reported by the Federal Emergency Management Agency (ASCE 1996).

Once the potential to resist blast loads using FRP composites had been demonstrated, it was necessary to develop both a simple design procedure and a test series to demonstrate the expected capabilities. Such a program was undertaken by the Defense Threat Reduction Agency, which sponsored the development of a simplified single-degree-of-freedom model to assess the survivability of the columns, and sponsored a series of full-scale tests to demonstrate the concepts (Morrill et al. 1999, 2004). A retrofit design procedure was developed using steel jackets or FRP wraps that were able to prevent column damage and building collapse, for circular, square, and rectangular columns. This simple column blast analysis and retrofit design procedure was based on first-principle calculations, and has been verified and validated against many full-scale blast tests (Crawford et al. 2003; Morrill et al. 2004). This code has been validated for reinforced concrete columns against several full-scale blast tests, using full-scale buildings (Fig. 12.3), or full-scale components in a testing frame that provides the proper end restraints (including the vertical load from the building) (Fig. 12.4).

Figure 12.3(a) and 12.4(a) show that blast loads will often cause unretrofitted columns to fail in diagonal shear, whereas Fig. 12.3(b), 12.3(c), and 12.4(b) show that composite wraps or steel jackets can provide sufficient diagonal shear enhancement to make the column survive. After blast testing, if the column had been properly retrofitted, its remaining axial capacity typically significantly exceeded its required axial resistance. Retrofitted and unretrofitted column deflection predictions using the column blast analysis and retrofit design software compared well with test results.

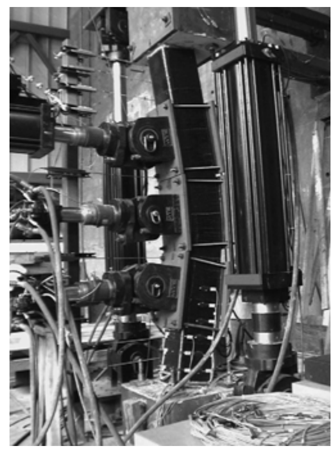
In addition, quasi-static laboratory tests with loading conditions simulating blast loads were conducted to measure

(a) Unretrofitted column (b) Steel jacketed column (c) Wrapped column

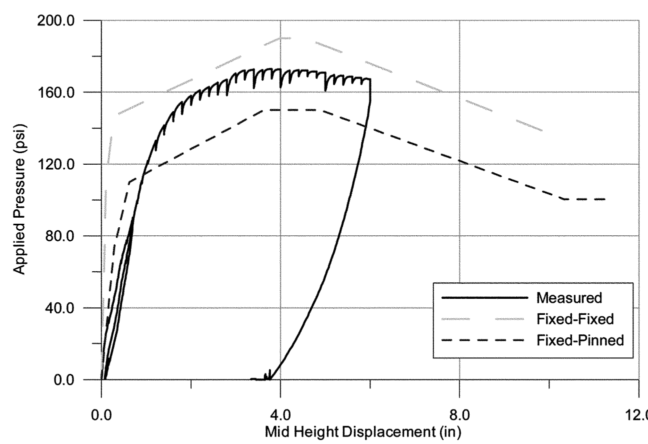
Fig. 12.3—Full-scale blast testing of reinforced concrete column in building (Crawford 2002b).

(a) Unretrofitted column (b) Composite wrapped column

Fig. 12.4—Full-scale blast testing of reinforced concrete column in loading frame (Morrill et al. 2001).



(a) Quasi-static test setup



(b) Measured-versus-predicted resistance

Fig. 12.5—Quasi-static laboratory tests under simulated blast loads (Morrill et al. 2001).

the flexural resistance of the columns (Fig. 12.5(a)). The measured flexural response is shown in Fig. 12.5(b) (Morrill et al. 2001).

While retrofitting columns with steel jackets and composite wraps is reminiscent of seismic retrofits, there are many basic differences. Besides the highly transient dynamics included into the single-degree-of-freedom design procedure, the deformed shape of first-floor building columns under blast loads usually includes two inflection points versus a single one for seismic loading (Morrill et al. 2001). This is an important difference because the deformed shape with two inflection points allows for the development of a compression membrane, which can increase the lateral resistance of the column tenfold. In turn, this requires increased shear capacity, often to be provided by composite wraps.

These building column retrofit concepts are currently being included into Department of Defense Unified Facilities Criteria to harden structures and to prevent progressive collapse (Malvar 2005). This technology has also been applied to bridge columns (Lan et al. 2003).

12.3.2 *Blast strengthening of walls*

12.3.2.1 *Reinforced concrete walls*—Research conducted by Muszynski (1998) investigated the performance of unreinforced and CFRP- or AFRP-reinforced concrete walls subjected to blast loadings at two different distances from the blast source. The reinforced walls were damaged due to either delamination of the composite material, or composite failure in tension at wall midheight. The CFRP- or AFRP-strengthened walls had less residual displacement than the bare control walls. In a follow-up test series, Muszynski et al. (2003a,b) used an autoclave-cured, three-ply, carbon fiber epoxy laminate and a knitted biaxial E-glass fabric, and later a Kevlar/glass FRP to reinforce concrete walls. The externally reinforced walls suffered high displacements, but did not fail.

Knox et al. (2000) sprayed the back of concrete walls with various unreinforced polymers, particularly polyurethane and polyurea. These polymers showed exceptional ductility, and were able to prevent deadly wall fragmentation.

12.3.2.2 *Unreinforced masonry walls*—Muszynski (1998) used CFRP and Muszynski et al. (2003b) used CFRP and Kevlar/glass FRP to strengthen masonry walls. The reinforcements reduced the residual displacements (compared with the displacements of the unstrengthened walls). The Kevlar/glass FRP seemed to provide additional ductility and further reduced the displacements.

Hammons et al. (2002) expanded the application of polyurea to concrete masonry walls and found it very successful in preventing deadly wall fragmentation. These polymers can be further strengthened by including continuous fibers (Crawford 2002a,b). Tests were recently conducted to further enhance wall response by including aramid fiber meshes within the unreinforced polymer (Morrill et al. 2005).

Baylot et al. (2002) conducted experiments on concrete masonry unit walls using quarter-scale models of typical 200 mm (8 in.) wide concrete masonry units. Three types of retrofitting schemes were evaluated for ungrouted and partially grouted walls. The first retrofit method consisted of 1 mm (0.04 in.) thick GFRP fabric bonded to the back face of the wall. The second retrofit technique consisted of a two-part sprayed polyurea product applied to the back face of the wall. The third retrofit method used galvanized steel sheet metal attached to the back of the wall. All the walls failed during the blast testing, but the GFRP retrofitted walls were successful in preventing loose fragments from penetrating within the structure, and were, therefore, considered successful retrofits.

Myers et al. (2003) tested eight masonry walls to investigate the feasibility of using FRP laminates and rods to improve the blast resistance of masonry walls. Two series of walls with different slenderness ratios and different strengthening schemes were tested at varying blast charge weights and standoff distances. Results indicated that shear capacity controlled the blast behavior of the strengthened walls, emphasizing that any retrofit scheme should not only examine the flexural strength of the system, but also the shear strength. The use of composites in the form of bonded laminates also demonstrated that the failure mode of the system was different from the control walls; in the case of bonded laminates on the interior face of the wall, very limited scatter of debris was noted—an important safety factor for building occupants.

In a follow-up study by Carney and Myers (2003b), the continuity between infill walls strengthened with FRP and concrete boundary elements was studied. Twelve walls were tested in the lab using a pressure bag with various strengthening schemes and connection detailing. Two validation blast tests in the field were performed to verify the observed static test results. Two connection details were examined, including an embedded NSM bar and an embedded sheet with an NSM bar in addition to a control without continuity. Increases in strength and strain energy on the order of two to three times were observed for the FRP-strengthened walls with the boundary connection compared with the control walls. The study again highlighted the control of debris scatter using FRP retrofits.

Crawford et al. (2002) and Crawford and Morrill (2003) used numerical models to study various retrofit techniques for stud and masonry walls.

12.3.2.3 *Wall design*—As for the case of columns, a simple software program was developed to assess the capacity of existing walls and determine the effectiveness of the retrofit (Morrill et al. 2004). Compression membrane action was also shown to provide walls with a significant lateral capacity that was often overlooked.

CHAPTER 13—FIELD APPLICATIONS

Remarkable progress has been made in field applications of FRP for internal reinforcement, prestressing and post-tensioning tendons, external reinforcement, and piles. Many reinforced concrete structures have been built and strengthened with FRP reinforcement in North America, Europe, Japan, and several other countries. The following projects have been selected to demonstrate the wide range of applications of FRP in new construction and in retrofitting and rehabilitation of existing structures.

13.1—FRP as internal reinforcement

13.1.1 *FRP for concrete bridge decks*—The Morristown Bridge (Benmokrane et al. 2006) is a single span, slab-on-girder bridge that spans over Ryder Brook in Vermont. The bridge has a span of 43.0 m (141 ft). The 230 mm (9 in.) thick concrete slab is supported by five girders spaced at 2.4 m (7.8 ft) on center, with a 915 mm (3 ft) cantilever on each side. This concrete bridge deck slab is totally reinforced with GFRP bars. The top and the bottom mesh consist of No. 6 GFRP bars that are spaced at 100 and 150 mm (4 and 6 in.) in the transverse and the longitudinal directions, respectively. The bridge was opened to traffic in 2002.

Other bridges have been constructed in Quebec (Benmokrane et al. 2004; El-Salakawy et al. 2005), and Texas (Bradberry and Wallace 2003) (Fig. 13.1).

13.1.2 *Hall's Harbour Wharf, (2000) (Nova Scotia, Canada) GFRP bar reinforcement*—Hall's Harbour (originally constructed in 1904) is a small fishing harbor in rural Nova Scotia. The Fundy shore, where the wharf is situated, is known for its large tidal range that averages about 10 m (33 ft). The tidal surge, when whipped up by high winds, can have devastating effects on the wharf. A large section of the wharf collapsed after a severe winter storm in 1998. The destroyed wharf needed to be replaced with a new and durable structure (Fig. 13.2).

The rehabilitation involved the construction of a pile-supported concrete deck structure. The concrete deck panels were precast archpanels (Newhook et al. 2000) that used a top layer of GFRP reinforcement to resist the effects of uplift caused by wave action on the soffit of the panels. The panels were supported on deep reinforced concrete beams spaced at 4.0 m (13.1 ft) centers and supported by steel piles. The beams were reinforced with a hybrid scheme of an internal layer of minimal steel longitudinal bars with an outer layer of longitudinal and transverse GFRP reinforcement. Many sensors, consisting of conventional electric resistance strain gauges and fiber optic sensors, were embedded within the critical parts of the wharf to monitor its performance (Newhook et al. 2000).

13.1.3 *FRP barrier walls*—Several slab-on-girder bridges have been constructed in Canada employing steel-free bridge deck technology that takes advantage of the internal arching action that develops in a well-confined deck slab. In these cases, FRP bars (glass and carbon) are used as tensile reinforcement in select locations, including barrier walls and cantilever sections. Four such projects and their respective usage of FRP are: Salmon River Bridge (1996) (Nova Scotia,

Fig. 13.1—GFRP deck slab reinforcement during concrete placing of Sierrita de la Cruz Creek Bridge (Bradberry and Wallace 2003).

Fig. 13.2—Wharf at Hall's Harbour.

Canada), GFRP grid in concrete curbs and parapet walls; Crowchild Trail Bridge (1997) (Alberta, Canada), GFRP bars in cantilever slabs and as longitudinal continuity reinforcement over piers, and GFRP grid in the concrete barrier walls (Tadros et al. 1998); Chatham Bridge (1996) (Ontario, Canada), CFRP grid in cantilever section of slabs and GFRP grid reinforcement in barrier walls; and Waterloo Creek Bridge (1996) (British Columbia, Canada), GFRP reinforcement in barrier walls and as continuity reinforcement with external abutments. A view of the cantilever reinforcement terminating over the interior girders for the Chatham Bridge is shown in Fig. 13.3.

13.1.4 *Specialty applications*—Conventionally reinforced underground hydroelectric chambers are prone to deterioration due to several factors, including attack from the deicing salts and the chemicals present in the ground water. Many chambers have to be replaced annually. Concrete chambers reinforced with GFRP reinforcement have been constructed and installed in the field as part of a joint study undertaken by

Fig. 13.3—Cantilever reinforcement for Chatham Bridge (photo courtesy of B. Bakht).

Fig. 13.4—GFRP reinforcement cage (ISIS Canada 2006).

Universite de Sherbrooke and Hydro Quebec (ISIS Canada, 2006; Fig. 13.4). The chambers have been instrumented and monitored for their performance.

FRP reinforcement is often chosen for its magnetic transparency. One example is the expansion of the magnetic resonance imaging center in Halifax, Nova Scotia, Canada. One of the challenging aspects of the performance criteria was that only a very minimal amount of steel was permitted within a predetermined distance from the magnetic resonance imaging unit. Because of the close proximity of the magnetic resonance imaging unit to its cast-in-place concrete foundation, GFRP reinforcing bars were used as the primary reinforcement for the foundation. The FRP reinforcement (Fig. 13.5) allowed the foundation to meet the structural requirements, as well as the magnetic transparency needed to meet performance criteria.

Toll highways use loop detectors to detect the characteristics of the vehicles passing through a toll plaza. These detectors are embedded within the continuously reinforced concrete pavement of the ramps and the main lanes. Metallic reinforcement can introduce noise to the signals from the loop detectors and lead to unreliable information about the passing vehicles. For loop detectors to function properly, the concrete in their immediate vicinity should be free of any metallic reinforcement up to a depth of 180 mm (7 in.). GFRP reinforcement was

Fig. 13.5—Reinforcement for magnetic resonance imaging base.

(a) Parking garage

(b) Grids in double tee flange

Fig. 13.6—Precast double tees containing CFRP grids, Milwaukee, WI (Carboncast 2006).

applied in a concrete pavement for Loop 1-SH 45 Williams County toll plaza in Texas (Bradberry and Wallace 2003). The design of the deck slab in the in the area housings the toll plaza was amended to a 254 mm (10 in.) thick slab with GFRP bars as top reinforcement and conventional steel bars as bottom reinforcement.

13.1.5 *Applications of CFRP grids in precast concrete structures*—Carbon fiber grids (Fig. 13.6) are routinely used to reinforce precast concrete members (Carboncast 2006). Their strength and resistance to corrosion make them ideal

(a) New Jersey Aquarium

(b) Budweiser Cold Storage

Fig. 13.7—Carbon fiber grids for shear connectors in sandwich walls (Carboncast 2006).

for reducing the thickness and, hence, the weight of precast concrete structures. They have been used in a variety of precast concrete products including double tees, architectural cladding panels, and insulated concrete sandwich panels. To date, over 5 million square feet of carbon grid reinforced concrete products have been erected or are being produced (TechFab LLC 2006).

The new parking garage for General Electric Healthcare in Wisconsin was constructed of precast concrete double tees. The flanges of the double tees for the roof were reinforced with a single layer of carbon fiber grid. The fire rating required for this project was satisfied by full-scale assembly testing.

Carbon fiber grids have also been used as shear connectors in a variety of insulated sandwich wall panel projects (Carboncast 2006). The carbon fiber grids resist horizontal shear between the concrete wythes of a sandwich panel, making the panel behave as a fully composite member. In addition, the relatively low thermal conductivity of the carbon fiber grids help to increase the thermal and energy efficiency of the panels. The New Jersey Aquarium and a Budweiser cold storage distribution facility are shown in Fig. 13.7.

13.1.6 *Applications of CFRP grids in concrete repair*—Carbon fiber grids are also applied for repair and strengthening of concrete structures. The restoration of the Naumburg Bandshell in Central Park (Fig. 13.8) used carbon grids in rebuilding the roof shell. The first phase of restoration involved repairing the leaking concrete roof. After removing the old roof, a new waterproofing membrane was installed, ribs constructed, and a thin concrete shell formed with three layers of carbon grid as reinforcement for the thin lifts of concrete placed by hand troweling. The carbon grid easily conformed to the hemispherical shape of the roof.

(a) Overview of bandshell

(b) Carbon grid reinforcement

Fig. 13.8—Restoration of the Naumburg Bandshell in Central Park, New York (Carboncast 2006).

Fig. 13.9—Beddington Trail Bridge (ISIS Canada 2006).

Fig. 13.10—Taylor Bridge, Manitoba, Canada (ISIS Canada 2006).

13.2—Prestressing applications

13.2.1 *Internal pretensioned reinforcement*—Two bridges were constructed in Canada in 1993 and 1998 with this technology. The Beddington Trail Bridge is a two-span, continuous skew bridge (Fig. 13.9). Each span consists of 13 bulb-T precast, prestressed concrete girders. Carbon fiber cables were used for four girders, while two other girders were prestressed using two rod tendons. Prestressing of CFRP was adapted to the practice by coupling the carbon fiber composite cables and Leadline™ rods to conventional steel strands. Couplers helped to minimize the length of CFRP tendons, and were staggered to allow use of same spacing for the conventional steel-reinforcing tendons. The girders using CFRP prestressing cables and rods have been instrumented with Bragg grating fiber optic sensors and are being monitored by ISIS Canada (2006).

The Taylor Bridge (Fig. 13.10) is situated in Winnipeg, Manitoba, Canada. This is a multispan bridge with a total length of 165 m (542 ft). Four, from a total of 40 prestressed precast concrete girders, were prestressed with CFRP reinforcement. FRP stirrups were also provided for shear resistance. This field application was one of the first in which bridge girders were reinforced totally with FRP (ISIS Canada 2006).

13.2.2 *Post-tensioning applications*—A continuous concrete box girder bridge over the Verdasio River in Switzerland (Keller 2003) had two spans of 31.4 and 37.6 m (103 and 123 ft), with a width of 6.0 m (19.7 ft). Both the webs of the box girder were fully prestressed. The concrete in the web had accumulated excessive amounts of chlorides due to the effects of deicing salts. Some of the prestressing cables were corroded, and there was up to 100% loss of cross section of non-prestressed reinforcement. The external prestressing system consisted of 19 pultruded carbon wires (5 mm [0.20 in.] diameter), with an outer diameter of 32 mm (1.3 in.). The cables were stressed to 65% of their nominal ultimate capacity to restore the initial structural resistance.

The BASF bridge in Ludwigshafen, Germany is a two-lane prestressed concrete bridge with a total length of 85 m (279 ft). Four CFRP cables were used in conjunction with 16 conventional steel cables as internal unbonded post-tensioned cables (Zoch et al. 1991). Each cable consisted of 19 CFRP strands of 12.5 mm (0.50 in.) diameter, with a prestressing force of 70 kN (15.7 kips) applied on each strand. All cables were guided through tubes embedded in the concrete. Steel tendon tubes were filled with concrete after tensioning, while the CFRP tendon tubes were not filled to allow inspection, exchange of cables, and data collection.

The Bridge Street Bridge in Southfield, Michigan (Fig. 13.11), completed in 2001, is a three-span prestressed concrete bridge, 63 m (69 yd) long and 17 m (18.6 yd) wide. Both bonded and unbonded CFRP tendons were used in the longitudinal as well as transverse directions (Grace and Abdel-Sayed 2003).

13.2.3 *CFRP tendons for lighting poles*—Prestressed concrete lighting poles were used for a parking garage in Switzerland (Terrasi and Lees 2003). The overall height of

(a) Bridge

(b) External Tendons

Fig. 13.11—Bridge Street Bridge (Grace et al. 2004).

Fig. 13.12—CFRP strip application to girder (Labossière et al. 2000).

the poles was 9.2 m (30.2 ft), from which 1.2 m (4 ft) was buried in the ground, leaving a clear height of 8.0 m (26.2 ft). The poles had an outer diameter of 120 mm (4.7 in.) at the tip and 220 mm (8.7 in.) at the base. These poles were made from centrifugally cast, high-strength concrete. Each pole contained six CFRP tendons that were prestressed to 60% of their ultimate tendon stress. The concrete had a minimum 28-day cube strength of over 90 MPa (13 ksi), a minimum tensile strength of 5 MPa (725 psi), and a Young's modulus of 38,550 MPa (5590 ksi). The poles were tested successfully under torsional and bending loads.

13.3—External reinforcement

13.3.1 *Beam and girder repair*—Sainte-Emile-de-l'Energie Bridge is a single-span bridge originally built in 1951. The four-girder bridge has a span of 21.3 m (70 ft). The vehicle weights have increased significantly since the bridge was constructed. Analysis showed that to sustain

Fig. 13.13—Maryland Street Bridge CFRP wrap on external girder (ISIS Canada 2006).

the increased loads, the flexural and shear strength of the bridge had to be increased by 35 and 20%, respectively. Carbon fiber reinforcing strips were used for increasing the flexural strength (Labossière et al. 2000) (Fig. 13.12). Glass-fiber reinforcement strips were used for shear strength enhancement. The performance of the strengthening scheme was verified by load testing.

The City of Winnipeg implemented an application of CFRP sheets as a first step in upgrading the shear capacity of the Maryland Street Bridge in Winnipeg (Fig. 13.13). The twin five-span continuous precast, prestressed concrete structures were designed and constructed in 1969. Analysis using current codes, however, indicated that the shear strength of the I-shaped girders was not sufficient to withstand the increased truck loads. An experimental study was conducted to examine the use of CFRP sheets on this particular girder shape. Four girders have since been strengthened using the sheets that were placed vertically with a horizontal layer placed across the top and bottom of the web for anchorage (Hutchinson et al. 2003).

The Farm-to-Market 1927 Bridge in Texas was originally constructed in 1964. This simply supported bridge has two 13.7 m (45 ft) end spans and two 18.3 m (60 ft) interior spans, with four prestressed concrete girders spaced 2.4 m (8 ft) center in each span. In January 2002, an overheight vehicle struck the inner span of the bridge, severely damaging an external girder between two diaphragms. The bottom flange and the web of the girder were fractured into several pieces. The damage, however, was mainly limited to concrete. The prestressing strands were found to have retained most of their preimpact tension. The concrete deck, the top flange, and the rest of the girder were in good shape. The restoration work consisted of repairing the damaged concrete by conventional methods and then wrapping with CFRP composite wrap (Bradberry and Wallace 2003). The CFRP wrap was provided to enhance the integrity of the

Fig. 13.14—Damaged and rehabilitated girder, Farm-to-Market Bridge, TX (Bradberry and Wallace 2003).

Fig. 13.15—CFRP application to girders, Sue Creek Bridge, TX (Bradberry and Wallace 2003).

Fig. 13.16—Portage Creek Bridge (ISIS Canada 2006).

repair work. The concrete was repaired using rapid-set, non-shrink grout and concrete, and epoxy injection (Fig. 13.14). The composite wrap was unidirectional carbon-fiber fabric and a compatible epoxy adhesive.

The Sue Creek Bridge was originally constructed before 1964. This simply supported bridge has two 9.1 m (30 ft) spans. The planned widening of the bridge necessitated the strengthening of the bridge. The two spans of the bridge were strengthened with two different types of CFRP strengthening systems (Bradberry and Wallace 2003). The first span was strengthened with longitudinally oriented CFRP fabric as the primary strengthening reinforcement and transverse CFRP fabric straps as the secondary reinforcement to control debonding of the longitudinal reinforcement. The second span was strengthened with longitudinally oriented CFRP pultruded laminates as the primary strengthening reinforcement and transverse CFRP fabric straps as the secondary reinforcement to control the debonding of longitudinal reinforcement (Fig. 13.15).

Nine two-story single-bay frames of a five-story beach resort in Egypt were found to be structurally deficient. Each bay spanned 12 m (39.4 ft) and had a 1.2 m (4 ft) deep beam. The frames lost between 20 and 30% of their flexural reinforcement and between 10 and 15% of their shear reinforcement because of corrosion of the reinforcement. The preparatory work involved removal of the rust and the repair of the concrete substrate. The flexural strengthening was applied as four 100 mm (4 in.) wide CFRP laminates bonded to the bottom fiber of the girder, and two 100 mm (4 in.) wide laminates attached to the sides of the beam (Mahfouz and Rizk 2003). U-shaped CFRP sheets, 100 mm (4 in.) wide, were used to enhance the shear capacity of the beam as well as to prevent the delamination of CFRP laminates.

13.3.2 *Column wrapping*—The Portage Creek Bridge (Fig. 13.16) in Victoria, British Columbia, Canada, was built before current seismic design codes and construction practices and would not resist potential earthquake forces as required by the current standards. Analysis showed that two short columns could fail catastrophically in shear during a large earthquake. The seismic upgrade included a retrofit scheme using FRP wraps to strengthen the short columns and a system for structural health monitoring (ISIS Canada 2006).

The structural system of the Faisal Bank's newly acquired office building in downtown Cairo, Egypt was analyzed to ascertain its adequacy for its intended use by the bank. The load-carrying capacity of 121 of its columns had to be increased by 25%. The columns, circular in cross section, with diameters varying from 300 to 750 mm (12 to 30 in.), were all 3 m (10 ft) high. Because of the premium on the office space in Cairo, steel or concrete jacketing was not an option. In addition, larger columns in the basement would have made parking of cars impossible. From the earthquake point of view, the steel or concrete jacketing would also have resulted in increase in the stiffness of the columns. The time

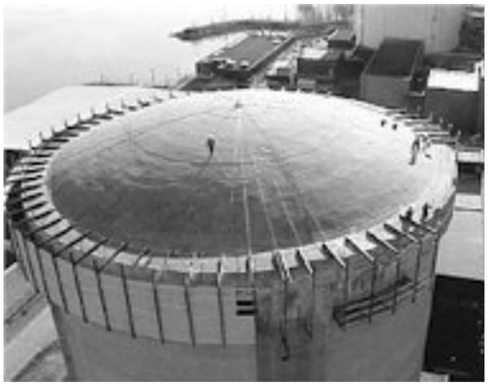


Fig. 13.17—Ring beam during rehabilitation (Demers et al. 2003).

(a) Koerschtal Bridge, Germany

(b) Sung San Bridge, Seoul, Korea

(c) Main Street Bridge, MB, Canada

Fig. 13.18—Site applications with prestressed CFRP plates or sheets: (a) the Koerschtal Bridge, Germany (photo courtesy of Sika Services AG, Basler et al. 2004); (b) The Sung San Bridge, Seoul, Korea (photo courtesy of Sika Services AG, Basler et al. 2004); and (c) the Main Street Bridge, Manitoba, Canada (Kim et al. 2006).

required to complete the upgrade was also an important consideration. Wrapping of the columns was done with one or more layers of CFRP with the orientation of fibers being circumferential, thus avoiding any increase in the flexural stiffness (Mahfouz and Rizk 2003). The columns were strengthened either by continuous or discontinuous CFRP wrapping, depending on the strength requirements.

13.3.3 *Nuclear reactor containment structure*—Severe deterioration of the concrete in a containment structure occurred due to harsh environmental conditions. The deteriorated concrete was repaired, and the ring beam of the containment structure was strengthened (Fig. 13.17). The repair scheme involved the removal and replacement of deteriorated concrete, and the application of over 700 m^2 (7500 ft^2) of GFRP fabric (Demers et al. 2003).

13.3.4 *Prestressed FRP plates or sheets*—The application of prestressed FRP sheets or plates is mentioned in Section 8.2, including its structural benefits and techniques for prestressing. Basler et al. (2004) reported three reinforced concrete bridges strengthened with prestressed CFRP plates, including particular attention on various anchor systems. The first North American site application using prestressed CFRP sheets was conducted in the Main Street Bridge Overpass No. 4, Winnipeg, Manitoba, Canada (Kim et al. 2006). The bridge, 56 m long and 26 m wide (183 x 85 ft), was frequently stuck with heavy trucks, resulting in rupture of internal prestressing strands. The insufficient flexural capacity was fully recovered after strengthening. Serviceability was also improved to meet code requirements. Figure 13.18 shows photos of some of these applications.

13.4—Masonry applications

The cathedral church of Arequipa, an example of one of the most important neoclassical monuments of Peru, was first built in 1629. Its two 28 m (92 ft) high slender towers were damaged as a result of an earthquake in 2001. The upper part of one of the towers collapsed, and the second tower was severely damaged. The rehabilitation scheme involved CFRP strips to strengthen the towers (Tumialan et al. 2003d). The reinforcing consisted of 250 mm (10 in.) wide CFRP strips, spaced at 550 mm (22 in.) centers, around the circumference of the member. A concrete ring beam was provided in the second segment where the vertical CFRP laminate strips from the first segment were anchored. The second segment was strengthened by providing circumferential CFRP laminates. Because of the porosity of the masonry surface, the high-viscosity saturant was directly applied to the masonry surface instead of the primer conventionally used on concrete members. A masonry wall was later built over the CFRP laminates to retain the original appearance of the towers.

Masonry domes at El-Aini, Egypt have diameters of 12 m (39 ft) and rest over cylindrical brick walls (Mahfouz and Rizk 2003). The domes in the cracked condition behaved as a set of isolated arches connected at the apex and supported by the cylindrical walls. First, the cylindrical walls were strengthened providing 200 mm (8 in.) wide circumferential CFRP strips spaced at 500 mm (20 in.). Subsequently, the domes were also strengthened by providing CFRP circumferential wraps. The surface of the wraps was coated with brick powder to blend with the existing brickwork.

13.5—Stay-in-place FRP forms

Stay-in-place FRP forms that are structurally integrated in the system have been used in a number of applications such as end-bearing or friction piles, fender piles, and structural supports for pier decks over the last 15 years. FRP forms provide a solution for deteriorated piers and waterfront structures subjected to the harsh marine environment. The population growth of marine borers, strict environmental laws that limit toxic treatments for wood, and prohibitions on traditional maintenance practices (lead-based primers, sandblasting, and solvent-based paints) have resulted in higher maintenance and replacement costs for traditional materials. Composite materials offer both long-term and low maintenance solutions. In this section, three available systems and

Fig. 13.19—New Castle Pier.

Fig. 13.20—Hardcore monopile.

their case studies or field applications are summarized. In addition, a novel bridge girder application using stay-in-place forms is also described.

13.5.1 *Marine pile systems and applications*—New Castle Pier in Delaware was reconstructed using 300 and 450 mm (12 and 18 in.) diameter hollow FRP composite tubes, driven over the remains of the original wood piling, in 2002. The hollow tubes were then filled with concrete and used to anchor the new pressure-treated timber deck as shown in Fig. 13.19.

Another field application was a monopile of composite tube as a single 24.4 m (80 ft) long, 1.5 m (5 ft) diameter pile, weighing only 6350 kg (13,990 lb). The pile was installed for the Delaware River and Bay Authority at the Lewes, Delaware, ferry terminal as shown in Fig. 13.20. The pile driving was performed using a vibratory hammer, and was faster than a typical steel monopile because the composite pile was fabricated in one continuous length, eliminating the need for field welding or splices. The pile was driven hollow and was not filled with concrete. The monopile was tested under a force of 1070 kN (240 kips), with a deflection of 790 mm (31 in.).

In a recent TEA-21 project, the Federal Highway Administration and Virginia DOT used CFFT piles in one bent of Route 40 Bridge over Nottoway River, as shown in Fig. 13.21 (Pando et al. 2003). The tubes were 14 m (46 ft) long, with a 0.6 m (2 ft.) diameter and a wall thickness of 6.4 mm (0.25 in.). The piles had a design capacity of 690 kN (155 kips).

Fig. 13.21—Bridge over Nottoway River (Pando et al. 2003).

Fig. 13.22—Naval pier at Ingleside (Fam et al. 2003d).

At the U.S. Naval Facility pier in Ingleside, TX, CFFT composite piles with 330 to 355 mm (13 to 14 in.) diameters were used as vertical and batter piles for fender applications as shown in Fig. 13.22 (Fam et al. 2003d). The piles were designed with a factor of safety of 10 at a design capacity of 178 kN.

As part of a pilot study for the Florida DOT, empty or concrete-filled FRP tubes, with or without field splices, were installed in March 2000 as shown in Fig. 13.23 (Mirmiran and Shahawy 2003). Field tests showed composite piles to be a feasible alternative for bridge substructure. No damage was observed at the top of the filled tube, nor in the tube-concrete interface, under more severe driving conditions than those allowed by current specifications for concrete piles (Mirmiran et al. 2002).

13.5.2 *Bridge girders*—The research and development efforts on concrete-filled CFRP tubes at the University of California, San Diego, have been implemented in a bridge demonstration project by the California DOT (Caltrans). One of the two parallel structures for the Kings Stormwater Channel Bridge (Fig. 13.24) of Route 86 in Riverside County was designed with the modular concrete-filled carbon shell system (CSS) construction concept (Karbhari et al. 2000). The bridge consists of a two-span, 20 m (66 ft) long and 13 m (43 ft) wide continuous system with a beam and slab superstructure and a multicolumn intermediate pier. The superstructure is composed of six carbon epoxy girders with 10 mm (0.40 in.) wall thickness and 0.34 m (13.4 in.)

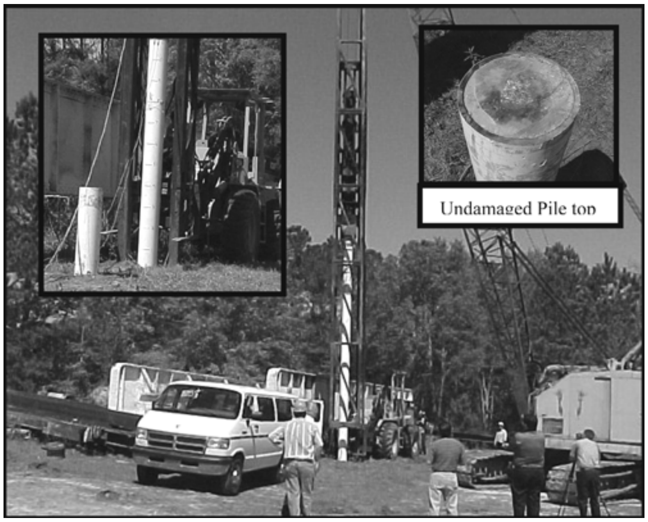


Fig. 13.23—Pile being driven as load-bearing pile in Florida (Mirmiran and Shahawy 2003).

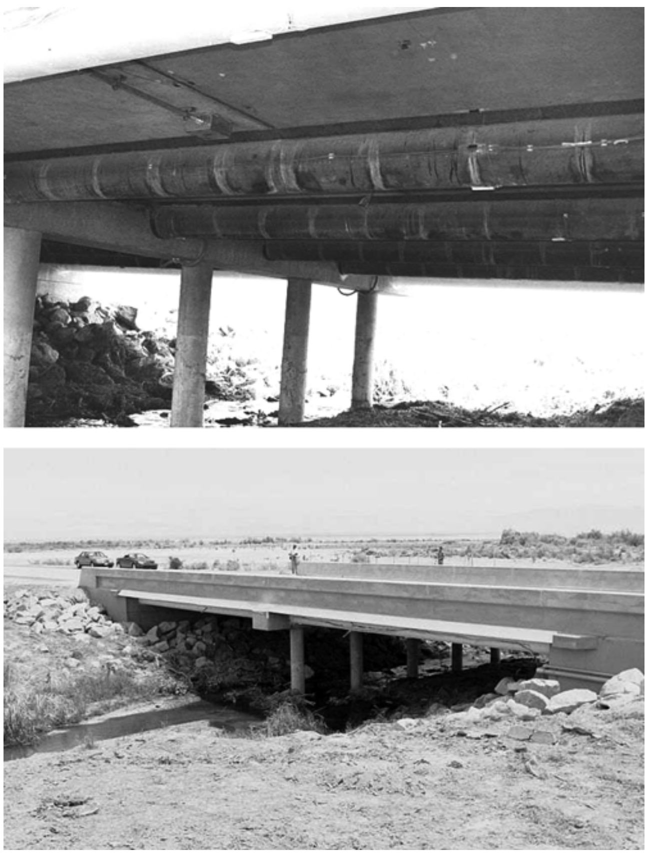


Fig. 13.24—Kings Stormwater Channel Bridge, San Diego, CA (Karbhari et al. 2000).

inside diameter filled on site with lightweight concrete. The girders are connected compositely with cellular E-glass vinylester FRP deck panels (Seible et al. 1999b). Continuing the carbon shell into the reinforced concrete elements and providing a conventional steel reinforcement anchorage allows connection of the carbon CFFT girders to the abutment end diaphragms and the center cap-beam. The pier columns are conventionally reinforced concrete pile extensions. After proof testing, the bridge was placed in service in July 2000. Long-term health monitoring of the Kings Stormwater Channel Bridge is being conducted by using permanent field monitoring instrumentation and a remote communication and data collection system (Zhao et al. 2001).

CHAPTER 14—RESEARCH NEEDS

14.1—Introduction

The research community has made great progress identifying and quantifying the characteristics of FRP composite materials for infrastructure applications. A growing number of demonstration projects and commercial applications contribute to the knowledge base in this area. The use of FRP materials in infrastructure is still in its infancy, however, and a number of issues remain to be adequately addressed. In some cases, the lack of understanding or design guidance represents a significant limitation to the broader implementation of FRP materials in infrastructure applications. This chapter identifies research areas and issues that industry, practitioners, and academia have identified as requiring further attention to improve the understanding of the behavior of FRP materials and their applications to concrete structures.

The following discussion of research needs should not be construed as representing an impediment to the broader adoption of FRP materials for concrete applications. Rather, this discussion of research needs acknowledges that, as with any emerging technology, limitations and gaps in knowledge exist.

As the various ACI Committee 440 subcommittees develop their respective reports and guidelines, five general needs are continually repeated. These critical needs are identified as:

- Data on the durability of FRP materials and FRP-reinforced concrete systems;
- Design and construction guidelines and specifications;
- Standardized material test methods;
- New materials and systems; and
- Future research directions.

These five themes will be repeated in more specific terms in the following sections.

14.2—Key research needs

In October 2004, the National Science Foundation sponsored a workshop (Porter and Harries 2005), and the primary objective was to identify critical research needs affecting the implementation of FRP composites in construction applications and to develop a consensus on the priority of these needs. This workshop was developed out of a 2002 survey of research needs (Harries et al. 2003). This section is based on the consensus of the workshop.

To address the immediate needs of the use of FRP in concrete construction, the following research topics are highly recommended for investigation. These topics address needs based on widely accepted applications of FRP in civil infrastructure.

14.2.1 *Durability and performance-related topics*

14.2.1.1 *Identification of appropriate environments for durability testing*—Considerable disagreement exists among researchers and practitioners as to exactly what environmental parameters need to be considered when using FRP materials.

Additionally, the intended use, regional climates, and maintenance practices will significantly impact which parameters affect a particular application. Such research studies should include participation from all primary climatic regions.

14.2.1.2 *Durability studies of externally bonded FRP repair or retrofit measures*—Time-dependent effects and factors affecting the durability of concrete structures with FRP (including fatigue) need to be identified. Clearly, the durability of the adhesive bond or interface between the FRP and the substrate is of primary concern.

14.2.1.3 *Durability studies of internal FRP reinforcement*—The same time-dependent effects identified for externally bonded FRP apply equally to internal FRP reinforcement. The critical issue for internal reinforcement is the behavior of FRP in-place; thus, studies should account for the concrete environment in which the FRP is embedded, the expected behavior (such as cracking) of that environment (which may differ from steel-reinforced members), and the environmental factors of importance (which also differ somewhat from those of importance for steel-reinforced members).

14.2.1.4 *Service life prediction of structures using FRP*—Models need to be developed to extrapolate short-term test results to long-term service life predictions. This topic also deals primarily with parameters related to durability, and should involve models of degradation processes. Fatigue life of bonded FRP has been shown to be of particular concern, and predictive models of this behavior are required.

14.2.1.5 *Fire resistance and protection of FRP*—The behavior of FRP materials, whether embedded in concrete or externally applied, subject to fire loading is largely unknown. Modeling techniques should be developed and verified for predicting the performance of structures in fire.

14.2.1.6 *Seismic and blast resistance of FRP systems*—FRP is very often used for structural retrofit, including efforts to mitigate the effects of earthquake or blast loads. Beyond pseudostatic testing, which does not capture strain rate effects, little is known of the performance of these retrofit systems under such extreme loads. Methods of assessing the appropriateness of existing and innovative FRP systems for mitigating the effects of extreme loading need to be developed.

14.2.2 *Development of standardized test methods*—A consensus on accelerated environmental conditioning techniques and subsequent durability test methods is required. Methods are required for both external FRP applications and internal FRP reinforcement applications.

14.2.3 *Design and construction guidelines and specifications*—Reduction and synthesis of data and development of analysis techniques and design guidelines, which are verified by experimental data, should form a major portion of future research programs. The inherent differences between the behavior 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 is also necessary. These philosophies and axioms can then be modified to become suitable for application to FRP-reinforced and prestressed concrete structures. Furthermore, with the continued acceptance of limit states design, reliability and probabilistic studies should be conducted to develop appropriate load and resistance factors for FRP materials and systems for incorporation into codes and design specifications. Construction guidelines and specifications are also needed for implementation in the field.

14.2.4 *New materials and systems*

14.2.4.1 *Innovative and hybrid materials*—FRP materials are often not mechanically or hygrothermally suited to applications in concrete infrastructure. Research aimed at developing new and hybrid FRP materials with properties better matched to concrete is necessary. Such systems may be as simple as combined CFRP, GFRP, and AFRP products, or as innovative as polymer-free chemically prestressed systems.

14.2.4.2 *Innovative reinforcing schemes*—Taking proper advantage of the FRP materials should involve more than simply replacing steel with FRP, and should move toward the development of innovative reinforcing schemes. Such advancement should make both FRP reinforcement and concrete construction more cost effective. One role that concrete plays in reinforced concrete systems is to protect the reinforcing system. If FRP systems can be made more robust and durable, this role for concrete becomes obsolete, and should result in savings.

14.2.4.3 *Self-sensing FRP structural health-monitoring systems*—FRP materials are unique in terms of their properties and their fabrication, which lends itself well to the development of integrated sensor systems. Such systems facilitate improved structural health monitoring and may be developed to allow the structure to interact with its occupants.

14.2.5 *Future research directions*—FRP materials represent a new paradigm in structural materials, and thus, innovation in this direction is required. Research aimed at developing innovative and appropriate applications of FRP should be encouraged.

14.2.5.1 *Leveraging properties of FRP in infrastructure*—Many properties of FRP materials are not currently exploited in the construction industry. For example, glass transmits light and carbon conducts electricity. Additionally, FRP materials may be tailored to retain stiffness or exhibit particular desired stiffness properties throughout their loading history. The inherent properties of FRP materials, as described, could be leveraged in smart structures that would be able to interact with their occupants, or offer stabilizing or self-centering behavior, respectively.

14.2.5.2 *Innovative material properties*—The tailorable nature of FRP materials makes them well suited to developing a wide spectrum of unique material properties. As an example, chemical or shape memory prestressing systems may represent an appropriate use of FRP materials. The exceptional fatigue performance of CFRP materials may make them well suited for active structural systems.

14.2.5.3 *FRP in sustainable construction*—FRPs are believed to represent a very sustainable construction tool. FRP materials have a relatively large impact-to-cost ratio (that is, few resources are engaged to result in a significant structural benefit). Furthermore, FRP materials and systems may offer otherwise obsolete structures a prolonged useful life. Integrating FRP materials into Leadership in Energy and

Environmental Design (LEED) certification is a recommended beginning in this regard.

14.2.5.4 *Research partnerships*—FRP materials and systems are not uniform, and are typically proprietary. For seminal research to be conducted, a close collaboration between academe and industry is essential. The corollary to this, of course, is the need to make trade secrets available. For these competing reasons, innovative industry-academe-government partnerships should be developed to facilitate seminal research. Such partnerships are even more critical considering the large number of disparate FRP systems being developed. Nonetheless, research involving fundamental interaction between industry and academe should take the highest priority, and is likely to yield the most fruitful results.

As has been made clear, long-term performance and durability of FRP materials and systems are considered to be of critical importance. Because of the great differences in environmental exposure of civil infrastructure in North America, a need exists for a coordinated effort involving research institutions located in different climatic regions to develop standardized environmental exposure test methods and protocols.

Educational partnerships are equally important. Few civil engineering departments teach courses addressing FRP materials. Broader impact criteria for assessing research proposals should help to address this need by favoring activities that will also incorporate FRP materials into the university classroom.

Finally, although FRP is becoming accepted in infrastructure engineering, it is still not well understood by the lay engineer. For example, the 2004 roof collapse at Charles de Gaulle airport in Paris, France was first blamed on an FRP detail that was later cleared of having any contribution to the collapse. For this reason, site visits with the intent of forensic investigation of structural failures of FRP systems should be supported. The findings of such visits could help to address concerns associated with FRP and to significantly augment the existing knowledge base of its use.

14.3—Conclusions

As various organizations and institutions, including ACI Committee 440 Subcommittees, develop their respective reports and guidelines, five general needs are continually repeated. These critical needs are identified as:

- Data on the durability of FRP materials and FRP-reinforced concrete systems;
- Design and construction guidelines and specifications;
- Standardized material test methods;
- New materials and systems; and
- Future research directions.

The presentation of extensive research needs herein is not an indication of an immature technology, but rather a technology that is rapidly maturing. FRPs for applications in concrete are limited only by the innovation of the practitioners and researchers involved in its development. This discussion presupposes that FRP materials offer a number of advantages to the concrete construction community. A key responsibility, however, is to apply this new technology in appropriate ways; this requires a more thorough case-specific understanding of FRP materials and their interaction with concrete materials and structures than is currently available. Thus, this chapter is intended as a guide to researchers and funding agencies alike to help them to prioritize their activities.

CHAPTER 15—REFERENCES

15.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

116R Cement and Concrete Terminology

209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

215R Considerations for Design of Concrete Structures Subjected to Fatigue Loading

318 Building Code Requirements for Structural Concrete

440.1R Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars

440.2R Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures

440.3R Guide Test Methods for Fiber-Reinforced Polymers (FRPs) for Reinforcing or Strengthening Concrete Structures

440.4R Prestressing Concrete Structures with FRP Tendons

530 (ASCE 5/TMS 402) Building Code Requirements for Masonry Structures

ASTM International

A 944 Standard Test Method for Comparing Bond Strength of Steel Reinforcing Bars to Concrete Using Beam-End Specimens

D 618 Standard Practice for Conditioning Plastics for Testing

D 695 Standard Test Method for Compressive Properties of Rigid Plastics

D 2101 Test Method for Tensile Properties of Single Man-Made Textile Fibers Taken from Yarns and Tows (Withdrawn 1995)

D 3410/D 3410 Standard Test Method for Compressive Properties of Polymer Matrix Composite Materials with Unsupported Gage Section by Shear Loading

D 3518/D 3518M Standard Test Method for In-Plane Shear Response of Polymer Matrix Composite Materials by Tensile Test of a ±45° Laminate

D 4255/D 4225M Standard Test Method for In-Plane Shear Properties of Polymer Matrix Composite Materials by the Rail Shear Method

D 4476 Standard Test Method for Flexural Properties of Fiber Reinforced Pultruded Plastic Rods

D 5379/D 5379M Standard Test Method for Shear Properties of Composite Materials by the V-Notched Beam Method

E 119 Standard Test Methods of Fire Test of Building Construction and Materials

Canadian Standards Association

A23.3 Design of Concrete Structures
S6 Canadian Highway Bridge Design Code
S478 Guideline of Durability in Buildings
S806 Design and Construction of Building Components with Reinforced Polymers

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www.astm.org

Canadian Standards Association
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Toronto, ON
M9W 1R3 Canada
www.csa.ca

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