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North American design guidelines for concrete reinforcement and strengthening using FRP: principles, applications and unresolved issues

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Abstract

This paper reports on the North American state-of-the-art in the use of FRP composites in concrete structures. For new construction, FRP bars have been used as the internal reinforcement in concrete members to replace conventional steel rebars for a host of reasons. The principles for design and construction have been established and proposed to industry by the American Concrete Institute (ACI). For repair and upgrade, strengthening of concrete members with externally bonded FRP laminates or near surface mounted (NSM) bars has received remarkable attention. The design and construction principles for use in practice are being finalized by ACI. On the application side, FRP materials have been used in some multi-million dollar projects for strengthening parking garages, multi-purpose convention centers, office buildings and silos. The drivers for this technology are several, but perhaps the most relevant one is the ease of installation. In the repair/upgrade arena (as well as new construction), perhaps one of the most important unresolved question remains that of durability (including fire resistance). Further research and validation is necessary to increase confidence in FRP technology for concrete construction.
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1. Design and construction of concrete reinforced with FRP bars

In this section, reference is made to key features in the recently published guidelines document by the American Concrete Institute (ACI) [1], which provides recommendations for design and construction of FRP-reinforced concrete (RC) structures as an emerging technology. The ACI document only addresses non-prestressed FRP reinforcement. Only notations critical to the understanding of the section are defined and equations are expressed in US customary and SI units.

1.1. Design principles

1.1.1. General

FRP materials are anisotropic and are characterized by high tensile strength with no yielding only in the direction of the reinforcing fibers. This anisotropic behavior affects the shear strength and dowel action of FRP bars, as well as their bond performance. Design procedures should account for a lack of ductility in concrete reinforced with FRP bars. Both strength and working stress design approaches are considered according to the provisions of ACI 318-95 [2] (ACI 318 from hereon). An FRP-RC member is designed based on its required strength and then checked for serviceability and ultimate state criteria (e.g. crack width, deflection, fatigue and creep rupture endurance). In many instances, serviceability criteria may control the design.

Design values: The design tensile strength that should be used in all design equations is given as $f_{fu}=C_E f^*_{fu}$, where: f_{fu} = design tensile strength of FRP considering reductions for service environment; C_E = environmental reduction factor given in Table 1 (column labeled 'Int.' = New construction/internal) for various fiber types and exposure conditions; and f^*_{fu} = guaranteed tensile strength of an FRP bar defined as the mean tensile strength of a sample of test specimens, $f^*_{u,ave}$, minus three times the standard deviation σ, ($f^*_{fu}=f^*_{u,ave}-3\sigma$).

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Table 1
Environmental-reduction factor C_E for various FRP systems and exposure conditions

Exposure condition	Carbon		Glass		Aramid	
	Int.[a]	Ext.[b]	Int.[a]	Ext.[b]	Int.[a]	Ext.[b]
Interior exposure	1.0	0.95	0.8	0.75	0.9	0.85
Exterior exposure	0.9	0.85	0.7	0.65	0.8	0.75
Aggressive environment	n/s	0.85	n/s	0.50	n/s	0.70

[a]New construction/internal; [b]Strengthening/external; n/s: Not specified.

The design rupture strain should be determined similarly, whereas the design modulus of elasticity is the same as the average value reported by the manufacturer. Design parameters in compression are not addressed since the use of FRP rebars in this instance is discouraged.

1.1.2. Flexure

Behavior and failure modes: If FRP reinforcement ruptures, failure of the member is sudden and catastrophic. However, there would be some limited warning of impending failure in the form of extensive cracking and large deflection due to the significant elongation that FRP reinforcement experiences before rupture. The concrete crushing failure mode is marginally more desirable for flexural members reinforced with FRP bars since the member does exhibit some plastic behavior before failure [3]. In conclusion, both failure modes (i.e. FRP rupture and concrete crushing) are acceptable in governing the design of flexural members reinforced with FRP bars provided that strength and serviceability criteria are satisfied. To compensate for the lack of ductility, the suggested margin of safety against failure is, therefore, higher than that used in traditional steel-RC design.

Φ factor: When concrete-crushing controls, a strength-reduction factor of 0.70 is adopted. Furthermore, a Φ factor of 0.50 is recommended for FRP rupture-controlled failure. While a concrete crushing failure mode can be predicted based on calculations, the member as constructed may not fail accordingly. For example, if the concrete strength is higher than specified, the member can fail due to FRP rupture. For this reason and in order to establish a linear transition between the two values of Φ, a section controlled by concrete crushing is defined as a section in which the reinforcement ratio, ρ_f, is greater than or equal to 1.4 times the balanced reinforcement ratio, ρ_{fb},($\rho_f \geq 1.4\rho_{fb}$) and a section controlled by FRP rupture is defined as one in which $\rho_f < \rho_{fb}$.

Minimum reinforcement: If a member is designed to fail by FRP rupture, $\rho_f < \rho_{fb}$, a minimum amount of reinforcement, $A_{f,\min}$, should be provided to prevent failure upon concrete cracking (that is, $\Phi M_n \geq M_{cr}$ where M_{cr} is the cracking moment). The minimum reinforcement area is obtained by multiplying the existing ACI 318 limiting equation for steel by 1.8 (i.e. 1.8=0.90/0.50 which is the Φ ratio).

Crack width: For FRP-reinforced members, the crack width, w, can be calculated from the expression shown in ACI 318 with the addition of a corrective coefficient, k_b, for the bond quality. The k_b term is a coefficient that accounts for the degree of bond between the FRP bar and the surrounding concrete. For FRP bars having bond behavior similar to steel bars, k_b is assumed equal to one. When k_b is not known, a value of 1.2 is suggested for deformed FRP bars.

Creep rupture and fatigue: Values for safe sustained and fatigue stress levels are given in Table 2. These values are based on experimental results with an imposed safety factor of 1/0.60.

1.1.3. Shear

When using FRP as shear reinforcement, one needs to recognize that: FRP has a relatively low modulus of elasticity; FRP has a high tensile strength and no yield point; tensile strength of the bent portion of an FRP bar is significantly lower than the straight portion; and FRP has low dowel resistance.

According to ACI 318, the nominal shear strength of a steel-RC cross section, V_n, is the sum of the shear resistance provided by concrete, V_c, and the steel shear reinforcement, V_s. Similarly, the concrete shear capacity $V_{c,f}$ of flexural members using FRP as main reinforcement can be derived from V_c multiplied by the ratio between the axial stiffness of the FRP reinforcement ($\rho_f E_f$) and that of steel reinforcement ($\rho_s E_s$). The equation for $V_{c,f}$ is that shown in Eq. (1) (noting $V_{c,f}$ cannot be larger than V_c).

$$V_{c,f} = \frac{\rho_f E_f}{\rho_s E_s} \; V_c \tag{1}$$

The ACI 318 method used to calculate the shear contribution of steel stirrups, V_s, is applicable when using FRP as shear reinforcement with the provision that the stress level in the FRP shear reinforcement, f_{fv},

Table 2
Creep rupture and fatigue stress limits in FRP reinforcement

Fiber type	Glass FRP	Aramid FRP	Carbon FRP
Creep rupture stress limit, $F_{f,s}$	0.20 f_{fu}	0.30 f_{fu}	0.55 f_{fu}

Fig. 1. Top GFRP mat placement.

should be limited to control shear crack widths, maintain shear integrity of the concrete, and avoid failure at the bent portion of the FRP stirrup, f_{fb}. The stress level in the FRP shear reinforcement at ultimate for use in design is given by $f_{fv}=0.002E_f\leq f_{fb}$. An expression for f_{fb} is given in ACI 440.1R-01 [1].

1.1.4. Development length

The development length of FRP reinforcement can be expressed as shown in Eq. (2). This should be a conservative estimate of the development length of FRP bars controlled by pullout failure rather than concrete splitting.

$$\ell_{bf}=\frac{d_b f_{fu}}{2700} \tag{2}$$

$$\ell_{bf}=\frac{d_b f_{fu}}{18.5} \quad \text{for SI units} \tag{3}$$

Manufacturers can furnish alternative values of the required development length based on substantiated tests conducted in accordance with available testing procedures. Reinforcement should be deformed or surface-treated to enhance bond characteristics with concrete.

1.2. Application for internal FRP reinforcement

1.2.1. GFRP-reinforced concrete bridge decks

The use of GFRP bars for reinforcing concrete bridge decks has captured some interest, particularly for the case of the replacement of the top steel mat [4]. The idea is to eliminate one of the major causes of deterioration (i.e. the steel reinforcement embedded in the concrete region more exposed to chlorides) without significantly increasing cost of construction and without totally removing steel reinforcement (see Fig. 1).

1.3. Unresolved issues

As pointed out in ACI 440.1R-01 [1], future research is needed to provide information in areas that are a real or perceived disadvantage, are still unclear, or are in need of additional evidence to validate performance. A list of research topics is shown below.

Materials: behavior of FRP-reinforced members under elevated temperatures; minimum concrete cover requirement for fire resistance; fire rating; effect of transverse expansion of FRP bars on cracking and spalling of concrete cover; creep rupture behavior and endurance limits of FRP bars; end treatment requirements of saw-cut FRP bars; and strength and stiffness degradation of FRP bars in harsh environment.

Flexure/axial force: behavior of FRP-RC compression members; behavior of flexural members with tension and compression FRP reinforcement; design and analysis of non-rectangular sections; maximum crack width and deflection prediction and control; minimum member depth for deflection control; and long-term deflection behavior.

Shear: concrete contribution to shear resistance; failure modes and reinforcement limits; and use of FRP bars for punching shear reinforcement in two-way systems.

Detailing: standardized classification of surface deformation patterns; effect of surface characteristics of FRP bars on bond behavior; lap splices requirements; and minimum FRP reinforcement for temperature and shrinkage cracking control.

Structural systems/elements: behavior of FRP-reinforced concrete slabs on ground.

Test methods: bond characteristics and related bond-dependent coefficients; creep rupture and endurance limits; fatigue characteristics; coefficient of thermal expansion; durability characterization with focus on alkaline environment and determination of related environmental reduction factors; strength of the bent portion; shear strength; and compressive strength.

2. Design and construction of FRP systems for strengthening

In this section, reference is made to the key issues of a technical document under development at ACI [5], which provides guidance for the selection, design, and installation of FRP systems for externally strengthening concrete structures as an emerging technology. Information on material properties, design, installation, quality control, and maintenance of FRP systems used as external reinforcement is presented in this ACI document. This information can be used to select an FRP system for increasing the strength and stiffness of RC beams or the ductility of wrapped columns. Conditions are also identified where FRP strengthening is beneficial and where its use may be limited. The ACI document does not address masonry walls.

2.1. Design principles

2.1.1. General

It is recommended that the increase in load-carrying capacity of a RC or prestressed concrete (PC) member strengthened with an FRP system be limited. The philosophy is that a loss of FRP reinforcement should leave a member with sufficient capacity to resist at least 1.2 times the design dead load and 0.85 times the design live load. Design recommendations are based on limit-states-design principles. This approach sets acceptable levels of safety against the occurrence of both serviceability and ultimate limit states (i.e. deflections, cracking, failure, stress rupture, fatigue). In determining the ultimate strength of a member, all possible failure modes and resulting strains and stresses in each material should be assessed. For evaluating the serviceability of an element, engineering principles, such as modular ratios and transformed sections, can be used.

FRP-strengthening systems should be designed in accordance with current ACI 318 strength and serviceability requirements. The strength-reduction factors required by ACI 318 should also be used. Additional reduction factors applied to the contribution of the FRP reinforcement are recommended to reflect the limited body of knowledge of FRP systems compared with steel RC and PC. For the design of FRP systems for the seismic retrofit of a structure, it may be appropriate to use established capacity design principles, which assume a structure should develop its full elastic capacity and require that members be capable of resisting the associated shear demands. The environmental-reduction factor, C_E, to determine the FRP design strength and strain was given in Table 1 for different fiber types and exposure conditions (see column labeled 'Ext.' = Strengthening/external). A similarity with the corresponding values adopted for internal FRP reinforcement should be noted.

2.1.2. Flexure

Failure modes: Guidance is given on the calculation of the flexural strengthening effect of adding longitudinal FRP reinforcement to the tension face of a rectangular RC member (concepts could be extended to cover T-sections and I-sections as well as PC). The nominal flexural capacity can be computed as per ACI 318. An additional reduction factor, $\psi_f = 0.85$, is applied to the flexural-strength contribution of the FRP reinforcement to account for lower reliability of the FRP reinforcement.

The strain level in the FRP reinforcement at the ultimate-limit state needs to be determined and limited to an upper value equal to the product $\kappa_m \varepsilon_{fu}$. The term κ_m is a factor no greater than 0.90 that is meant to limit the strain in the FRP reinforcement to prevent debonding or delamination. This term recognizes that laminates with greater axial stiffness (i.e. the product elastic modulus by cross sectional area) are more prone to delamination. The κ_m term, that was primarily developed to address interfacial debonding at intermediate cracks, is only based on a general recognized trend and on the experience of engineers practicing the design of bonded FRP systems.

Φ factor: The strength-reduction factor depends on the strain in the steel at ultimate, ε_s. Φ is set equal to 0.90 for ductile sections ($\varepsilon_s \geq 0.005$) and 0.70 for brittle sections where the steel does not yield. A linear transition for the reduction factor between these two extremes is then established.

Stress limits: To avoid plastic deformations, the existing steel reinforcement should be prevented from yielding at service load levels. The stress in the steel at service should be limited to 80% of the yield stress. Similarly, to avoid failure of an FRP-reinforced member due to creep rupture of the FRP, stress limits for these conditions should be imposed on the FRP reinforcement. Limits on sustained and fatigue stresses are those listed in Table 2 and are identical to those for internal FRP reinforcement.

2.1.3. Shear

The nominal shear capacity of an FRP-strengthened concrete member can be determined by adding the contribution of the FRP reinforcing to the contributions from the reinforcing steel and the concrete. An additional reduction factor ψ_f, is applied to the contribution of the FRP system. The additional reduction factor, ψ_f, should be selected based on the known characteristics of the application but should not exceed 0.85 for two- and three-sided wrapping schemes and 0.95 for completely wrapped elements.

The shear strength provided to a member by the FRP system should be based on the fiber orientation and an assumed crack pattern [6]. It can be determined by calculating the force resulting from the tensile stress in the FRP along the assumed crack with an expression similar to that of steel reinforcement. To compute the tensile stress in the FRP shear reinforcement at ultimate, it is necessary to calculate the effective strain, f_{fe}, in the FRP.

FRP systems that do not enclose the entire section (two and three-sided wraps) have been observed to delaminate from the concrete before the loss of aggregate interlock of the section. For this reason, bond stresses should be analyzed to determine the usefulness of these systems and the effective strain level that can be achieved. The effective strain is calculated using a bond-reduction coefficient, κ_v, so that $\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004$ (for U-wraps or bonding to two sides).

The bond-reduction coefficient is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the laminate [6]. The methodology for determining κ_v has been validated for members in regions of high shear and low moment, such as simply supported beams. The methodology has not been confirmed for shear strengthening in areas subjected to simultaneous high shear and moment loads, such as continuous beams. In such situations, conservative values for κ_v are recommended.

2.1.4. Compression members

The axial compressive strength of a non-slender member confined with an FRP jacket is calculated using the conventional expressions of ACI 318 substituting for f'_c the factored confined concrete strength $\psi_f f'_{cc}$. The additional reduction factor is set to $\psi_f = 0.95$. Vertical displacement, section dilation, cracking, and strain limitations in the FRP jacket can also limit the amount of additional compression strength that can be achieved with an FRP jacket. If the member is subjected to combined compression and shear, the effective strain in the FRP jacket should be limited based on the criteria given by $\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$.

At load levels near ultimate, damage to the concrete in the form of significant cracking in the radial direction occurs. The FRP jacket contains the damage and maintains the structural integrity of the column. At service load levels, this type of damage should be avoided so that the FRP jacket will only act during overloads that are temporary in nature. To avoid radial cracking under service loads, the transverse strain in the concrete should remain below its cracking strain at service load levels. This corresponds to limiting the stress in the concrete to $0.65f'_c$. In addition, the stress in the steel should remain below $0.60f_y$ to avoid plastic deformation under sustained or cyclic loads. By maintaining the specified stress in the concrete at service, the stress in the FRP jacket will be negligible. Because FRP jackets provide passive confinement, service load stresses in the FRP jacket should never exceed the creep-rupture stress limit.

2.1.5. Near surface mounted (NSM) reinforcement

Although not directly addressed by ACI 440 [5], the use of NSM FRP bars is a promising technology for increasing flexural and shear strength of deficient RC and PC members [7]. The advantages of NSM FRP bars compared with external FRP laminates are the possibility of anchoring the reinforcement into adjacent RC members, and minimal surface preparation work and installation time. For installation, a groove is cut in the desired direction into the concrete surface, the groove is then filled halfway with adhesive paste, and the FRP bar is placed in the groove and lightly pressed. This forces the paste to flow around the bar and fill completely between the bar and the sides of the groove. Finally, the groove is filled with more paste and the surface is leveled. As this technology emerges, the structural behavior of RC and PC elements strengthened with NSM FRP bars needs to be fully characterized.

Fig. 2. Typical stem cracking prior to strengthening.

2.2. Applications of strengthening

2.2.1. Strengthening with CFRP externally bonded laminates

A 90 000 m^2 (1 million $foot^2$) parking garage in Pittsburgh, Pennsylvania, was strengthened in order to address concerns regarding distress in the double tee beams supporting each elevated floor [8]. The short term parking garage at Pittsburgh International Airport is a three-story precast concrete structure. The two elevated levels (levels 2 and 3) consist of 2.8-m (9.2-foot) wide by 18.5-m (60.7-foot) long precast/prestressed double tees. The double tee units are generally 760 mm (2.5 fppt) in total depth and are dapped at the ends to bear on ledger beams. Cracking at the re-entrant corners of the dapped ends of the double tees was observed and initially gave cause for concern. Cracking was observed in nearly every double tee in the garage, and though the crack angle varied widely, most cracks were inclined between 0° (horizontal) and 45° (see Fig. 2). As part of a condition survey, four of the double tee beams were selected to be load tested in order to assess their structural adequacy. During load testing, flexural-shear cracking developed at a location approximately 1.5 m (4.9 foot) from the dapped ends. As loading increased, the flexural-shear cracking became so severe that the beams were deemed to have failed at roughly 75% of their design capacity.

In assessing various strengthening alternatives, externally bonded CFRP reinforcement was determined to be the most cost effective, the least disruptive to the operation of the parking garage, and virtually unnoticeable once the installation was complete. A load test of several tees after the installation of the FRP reinforce-

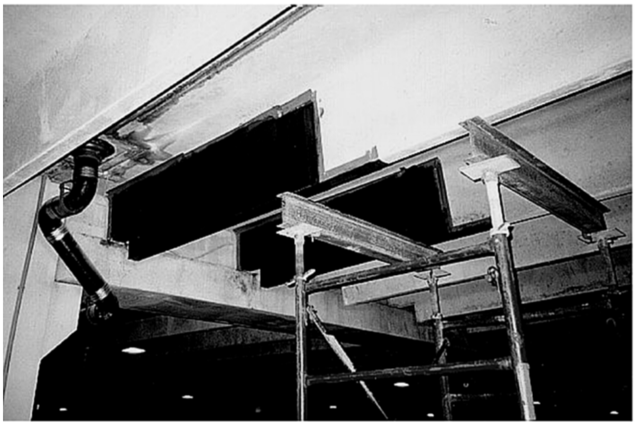

Fig. 3. Installed FRP U-wraps (0/90°).

ment demonstrated that the retrofitted tees could support loads over 100% of the design load. Externally bonded CFRP laminates were used as shear reinforcement at 0/90° combination on the stem of each double tee and as flexural reinforcement at 0° on each stem's soffit (see Fig. 3). The amount of FRP laminates provided was based on the level of deficiency of the member (i.e. one or two vertical 'U' wraps per stem; two horizontal plies per side of stem; and two horizontal plies on the bottom of each stem). The demand on the level 3 double tees is greater due to a thicker asphalt overlay and exposure to snow loading. The deficiencies in the double tees that needed to be addressed were both flexural and shear deficiencies at the ends of each tee.

The precast double tees have a nominal concrete strength of 31 MPa (4500 psi). The FRP material used utilizes a high strength carbon fiber fabric with the following characteristics: thickness, $t_f = 0.165$ mm (0.0065 in); elastic modulus, $E_f = 227$ GPa (33 Msi); and strain at rupture, $\varepsilon_f = 0.017$ mm/mm (in/in). The ends of the double tees were reinforced continuously with the FRP reinforcement (as opposed to reinforcing with strips of reinforcement spaced apart). The depth of the FRP reinforcement, d_f, varied based on variations in the effective depth of the section, d_p.

In order to accommodate any horizontal component of force and to provide additional anchorage for the vertical FRP reinforcement, horizontal plies of FRP reinforcement were added to the sides of the stems as well. In addition to the FRP shear reinforcement, two 100-mm (4-inch) wide CFRP strips were placed longitudinally on the soffit of the double tee stems. This reinforcement was used to limit the flexural strain level on the stem soffit to 0.005 mm/mm (in/in) under a positive bending moment of 493 kN-m (363 foot-kip). All of the existing cracks wider than 0.50-mm (0.02 inch) in the areas where the FRP was to be installed required epoxy injection. The surface to which the FRP was being installed also had to be profiled by water blasting.

In addition to buildings, a number of applications have included bridges. For example, three bridges in Boone County, Missouri were selected for strengthening with CFRP laminates in both shear and flexure [9]. The objective of the rehabilitation program was to remove the load posting that had been imposed on each of the bridges. The three bridges were all simply supported spans, ranging from 5.92 m (19.43 foot) to 11.84 m (38.84 foot), and composed of precast RC channel sections. The RC channel sections had a total depth ranging from 457 mm (18 inches) or 610 mm (24 inches).

The existing capacity of the bridges was determined using basic principals of RC theory. Based on material properties and geometry, the existing factored moment and shear capacities for a single RC channel section of one of the bridges were: $\phi M_n = 201$ kN-m (148 kip-foot) and $\phi V_n = 159.2$ kN (35.8 kip). Design loads were based on an HS20 truck load as given by AASHTO design guidelines [10] for precast multi-girder bridge decks. The moment and shear demand imposed by an HS20 for a single RC channel section of the reported bridge were: $M_u = 240$ kN-m (177 kip-foot) and $V_u = 191.3$ kN (43.0 kip).

The FRP material selected for this application had the same characteristics and installation process of that described in the previous parking garage application. Flexural strengthening was attained by applying strips of CFRP laminate to the soffit of the channel legs with the fibers oriented in the longitudinal direction (along the length of the bridge) (see Fig. 4). For the reported bridge, a 127-mm (5-inch) wide single ply strip is applied over the entire length of the span developing a new flexural capacity $\phi M_n = 247$ kN-m (182 kip-foot).

Shear strengthening the bridges was achieved by

Fig. 4. Completed installation of CFRP laminates.

Fig. 5. Horizontal and vertical grooves.

applying strips of CFRP laminate (with fiber perpendicular to the direction of the bridge) to the sides of the web on each channel e.g. (see Fig. 4). Due to the way sections are joined side-by-side, the CFRP strips could only be applied to one side of each channel leg. Each strip would extend from the channel web across the soffit of the leg, covering the longitudinal FRP strip. Shear strengthening for all three bridges consists of 610 mm (24 inches) wide single-ply CFRP strips spaced at 864 mm (34 inches) center-to-center developing a shear capacity $\phi V_n = 213.9$ kN (48.1 kip).

2.2.2. Strengthening with NSM carbon FRP bars

Six cement silos, located in the Boston, Massachusetts, were upgraded with the NSM FRP bar technology [11]. The silos are part of a larger complex composed of structures built in different periods. In 1962, a cluster of four silos (coded 1, 2, 3 and 4) was constructed; each silo is 45.7 m (150 foot) tall, with a diameter equal to 6.7 m (22 foot). In 1979, other four silos (coded 5, 6, 7 and 8) were added, each standing 39.6 m (130 foot) tall and 13.4 m (44 foot) in diameter. The repair was carried out on silos 1, 2, 3, 4, 7 and 8. They are characterized by raft foundations and a hopper independently supported by a ring beam and column system. Wall thickness is equal to 203 mm (8 inches) for the first four silos and 254 mm (10 inches) for the other two; for all of them, a single layer of vertical and horizontal steel reinforcement is placed close to the outside surface of the walls.

Initial signals of distress were observed in 1994, alerted by cracks and leakage of material through the silo walls. Inspections and analysis evidenced several problems. In particular, field measurements of reinforcing bar spacing showed inconsistency with construction drawings and revealed that the actual spacing of the vertical and horizontal steel bars was larger than specified in many regions. This resulted in significant levels of under-reinforcement. After an accurate economic evaluation and considering the peculiar cluster orientation of the silos, with common intersecting walls and partial access around their perimeter, the owner decided to repair the structures and, among different upgrade techniques, selected the use of NSM CFRP bars.

The design required that an 8-mm (5/16-inch) diameter CFRP bar be used in the vertical direction along the entire height of the silo. The spacing of these bars is such that one bar is located midway between each of the existing vertical steel bars (i.e. 460 mm (18 inch) o.c. spacing). In the horizontal (hoop) direction, an 11-mm (7/16-inch) diameter CFRP bar was used along the height of the silo starting from the level of the interior ring beam and continuing to the top of the silo. The spacing of these bars is also such that one bar is located midway between each of the existing horizontal steel bars (i.e. 200 mm (8 inches) o.c. spacing). The specifications for the CFRP bar were as follows: minimum elastic modulus = 148 GPa (21.5 Msi), and minimum strain at failure = 0.0105 mm/mm (in/in). The CFRP bar surface was roughened for improved bond with concrete by use of a peel ply during fabrication or by means of sand blasting after fabrication.

Installation of the bars began with the grooving operation (see Fig. 5). Customized grooving tools allowed technicians to cut the appropriate grooves in one pass. Where the groove intersected the common wall locations, a hole was drilled that was tangent to the curve of the silo to the depth determined by pull out test results. A two-component, high-viscosity, epoxy adhesive was injected into the deeper, vertical grooves. The bars, some 46 m (150 foot) long, could be handled in single pieces due to their lightweight properties. The vertical bars were then inserted into the groove and embedded in the adhesive (see Fig. 6). A second layer of adhesive was applied on top of the FRP bar. Upon completion of the vertical installation, the circumferential bars were installed. Adhesive was installed in the

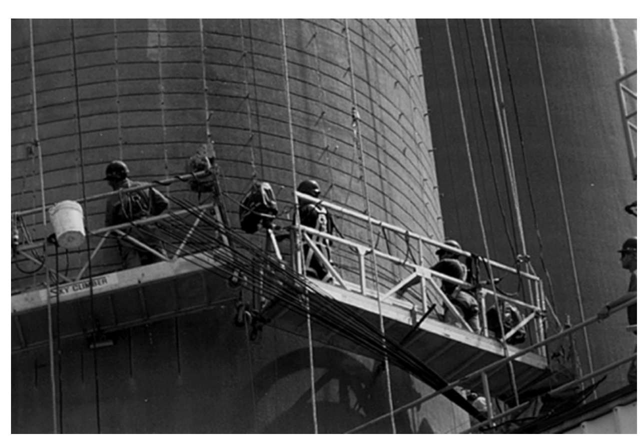

Fig. 6. FRP bar installation.

dowel holes in the two common wall intersections and in the horizontal grooves. Starting at the first dowel hole, a single length bar was inserted into the dowel hole and then placed into the horizontal groove around the circumference until it met the other common wall intersection. The remaining length was inserted into the second dowel hole. The second layer of adhesive on top of the horizontal bar was installed, and the material was then finished to create a surface appearance that could be easily hidden for aesthetics.

2.3. Unresolved issues

Several of the unresolved issues relative to internal reinforcement also pertain to the case of strengthening, for example, durability (including fire) and test methods, so that repetitions are not necessary. Further research into the mechanics of bond of FRP reinforcement is of critical importance. New experimental evidence and analytical tools should yield more accurate methods for predicting delamination at the interface and in the concrete. Furthermore, developments will likely account for the stiffness of the laminate, the stiffness of the member to which the laminate is bonded, and the influence of the adhesive thickness and properties. The interim recommendations to limit the strain in the FRP to prevent delamination need to be revisited and confirmed. In addition to delamination, it is critical to better understand other failure phenomena associated to external reinforcement such as peeling of the concrete cover. A list of critical issues follows:

Bond: effect of concrete surface profile, effect of adhesive thickness and properties, cover delamination, methods to improve anchorage, effect of distributed or damaged regions (holes, voids, etc.), detailing.

Seismic upgrade: develop special provisions for seismic rehabilitation of concrete structures using FRP.

Inspection and quality control: develop protocols and test methods for quality assurance.

NSM reinforcement: efforts need to be made to allow for other forms of FRP strengthening.

3. Conclusions

Even with some unresolved issues that should become a priority for future research, it can be concluded that the availability of design and construction guides developed by ACI for the use of FRP internal and external reinforcement for new and existing structures should allow the construction industry in North America to take full advantage of this emerging technology.

Applications for new construction where internal FRP reinforcement is used are slowly developing. It is expected that the use of glass FRP bars will dominate in this market. Applications for strengthening of existing structures with externally bonded laminates and NSM bars has already captured a significant market share with several multi-million dollar projects.

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