Carbon Fiber Post-Strengthening of Concrete Structures
An Alternative to Steel Retrofits

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Introduction
In the 1970s, exterior bonded steel reinforcement was used sporadically to reinforce bridges to attain higher service load capacities. This steel post-strengthening system consisted of steel straps bonded with epoxies to the exterior of concrete beams. An alternative to the steel retrofit is woven carbon fiber reinforced polymer fabric applied in the same manner as steel. Carbon fiber composite laminates, when compared with steel post-strengthening reinforcement, offer superior tensile strength, comparable modulus of elasticity, high corrosion resistance, lower weight, and ease of construction. CFRP shortcomings include high price, low ductility, and few in situ examples of its application.

CFRP fabric is commercially available in many forms and trade names. One type is a knitted fabric in which “yarns” of fiber are oriented in different directions, not always at 90 degrees, and stitched together. A second type of CFRP is a unidirectional matrix. Continuous carbon fibers are oriented in one direction and “glued” together by a resin matrix. The resin that binds individual fibers together to form the composite is usually a vinyl ester or epoxy. Vinyl esters are used when high corrosion resistance is needed and epoxy is used where high modulus and low creep are required.

Not all concrete structures are good candidates for post-strengthening using CFRP. Structures that have new dead loads higher than the capacity of the existing structure are poor candidates for reinforcing with fiber composites. Excellent choices for post-strengthening using CFRP include structures with small dead loads and transient live loads. Likewise, structures that are being seismically upgraded are excellent choices for post-strengthening using CFRP because they have no additional dead loads but must be reinforced to resist a transient seismic load.

One prevalent issue in using CFRP that may not be an issue with other post-strengthening materials is long-term creep. Long-term creep of the composite is directly influenced by the ratio of fiber material volume to overall volume (volume fraction). Resins tend to creep more than the fibers themselves, so a higher volume fraction results in composites that creep less than composites with lower volume fractions. After load is applied to CFRP, relaxation occurs, which is caused by a high creep rate. Soon after this the creep rate reduces until creep rupture begins. The allowable long-term stress level is a function of the time until creep rupture occurs. Little research on creep rupture of fiber composites has been completed. Careful design of retrofit of structures with carbon fiber laminates should involve using a laminate with a high volume fraction, i.e., a laminate that has more than 70 percent carbon fibers by volume.

Adhesives that are used to bond the carbon fiber to the concrete are also highly susceptible to creep with long load duration.
Design Procedure

The analysis of CFRP reinforced concrete beams is complicated by the fact that the stresses in the beam are not zero at the time of reinforcement. The dead load stresses, unless shoring is provided, should be considered. We can assume that if $f_{cDL}$ (stress in concrete due to dead load only) $< f_c/2$ the beam is elastically stressed, as in figure 1A. Therefore,

$$f_{sjDL} = \frac{M_{DL}}{A_s j d}$$

from equilibrium. The allowable stress in the steel caused by live load can be expressed as,

$$f_{sjLL} = f_y - f_{sjDL}$$

since the steel reinforcing should yield and the CFRP (non-yielding) should fail prior to the concrete crushing (under-reinforced). The traditional equations for concrete moment capacity apply to carbon fiber reinforced concrete, except that the Whitney stress block approximation has been shown, by experiment, as an invalid approximation for reinforcing beams with carbon fiber. The coefficients $\alpha$ and $\beta$, describing the non-linearity of compression in concrete, are calculated from empirically derived, piece-wise equations (as shown in notation). The sum of the moments about the resultant of the concrete stress results in $M_n = T_L(h-\beta c) + T_S(d-\beta c)$ (see figure 1B). Adding the dead load steel stress results in: $M_n = T_L(h-\beta c) + A_S(f_y - f_{SDL})(d-\beta c)$, which is the moment capacity of the CFRP reinforced section. The distance to the neutral axis can be calculated by the use of strain compatibility, for live loads only, (the strain in the concrete, steel, and carbon are all proportional) which yields

$$c = \frac{f_{Ld} A_L + f_y A_S}{abf_c}$$

(see figure 1C). This assumes the carbon and the steel simultaneously rupture and yield, respectively. The concrete strain or stress should be checked to verify that the concrete does not crush, i.e., concrete strain is less than 0.003 or concrete stress is less than 0.003Ec. The stress in the steel must be checked to verify that it does not rupture but has yielded. It has been suggested the nominal moment capacity must be checked against the factored actual moments:

$$\phi M_n = 1.4M_{DL} + 1.7M_{LL}$$

with $\phi = 0.7$. This approach uses the resistance factor of 0.7 instead of 0.9 because of the lower ductility of a carbon reinforced concrete beam and the workmanship variability of the application. A duration factor, depending on required useful life, should also be considered in this equation. It has been suggested for life spans of 100 years that the long-term load be limited to 30 to 40 percent of the CFRP ultimate capacity.

CFRP reinforced beams are more likely controlled by the bond between the concrete and the CFRP laminate instead of the flexural capacity. Experimental research conducted on steel plates bonded to concrete resulted in an equation for development length that can also be used for CFRP:

$$l_y = \frac{\left(\frac{T_{Ld}}{\phi}\right)^{2}}{b^2 k t_c \tau_k} \geq 20 \text{ inches.}$$

Where $\phi = 0.7$, $T_{Ld}$ equals the tensile force in the CFRP, $b_c$ equals the width of the laminate, $k$ is a constant equal to 4.35ksi, $t_c$ equals the width of an equivalent steel laminate, and $\tau_k$ equals the delamination stress. Delamination stress tests are conducted on laminate samples on in situ concrete by “peeling off” two-inch diameter laminate disks that are bonded to the concrete. A current ASTM standard test method has not been implemented, but this test can be carried out similarly to ASTM D 1876 (see...
The designer must use this equation with conservatism since no practical design rules are in place for specifically anchoring CFRP laminates. CFRP reinforcement should extend beyond the point at which it is no longer required to resist flexure for a distance equal to $l_1 + d/2$. In addition, horizontal shear at the interface of the laminate and concrete should be compared to the allowable adhesive bonding agent shear. The average horizontal shear stress can be written as

$$\tau = \frac{V Q_{cr}}{I_{cr} I_{1'}}.$$

**Conclusions**

The application of CFRP laminates to concrete beams is a viable means of post-strengthening structures that are slightly overstressed by transient loads. Designers should use it with extreme caution because experimental research is still in its infancy.

Structures with high permanent loads should be strengthened with CFRP using reduced resistance factors because creep rupture characteristics are not known. Building codes have not yet been adapted for CFRP, but in time a set of codes will be developed with the continuance of more construction applications.

**Notation:**

- $A_s$: Area Reinforcement Steel
- $A_t$: Area of Reinforcing Carbon
- $b$: Beam Width
- $c$: Distance to Neutral Axis
- $C$: Compression Force
- $d$: Beam Structural Depth
- $f_c$: Stress in Concrete
- $f'_c$: Concrete Compressive Strength
- $f_{DL}$: Steel Stress due to Dead Load
- $f_{LL}$: Steel Stress due to Live Load
- $f_{UL}$: Ultimate Carbon Stress
- $f_y$: Steel Reinforcing Yield stress
- $h$: Beam Height
- $I_{cr}$: Moment of Inertia of Cracked Section
- $k$: A constant
- $\alpha$: Concrete Compression Stress Factor
- $\epsilon_c$: Concrete Compression Stress Factor
- $\epsilon_{cLL}$: Strain in Concrete due to Live Load
- $\epsilon_{sLL}$: Strain in Steel due to Live Load
- $\epsilon_{LL}$: Strain in Carbon due to Live Load
- $\phi$: Resistance Factor
- $\rho$: Percent Steel