Fiber Reinforced Polymer (FRP) Strengthening Design for Concrete Bridge Strengthening: Codal Comparisons

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ABSTRACT

Fiber Reinforced Polymer (FRP) composite materials provide effective and potentially economic solution for rehabilitating and upgrading the existing reinforced and precast concrete bridge structures that have suffered deterioration. Each year, there are significant numbers of damaged bridge, mainly due to structural failure, reinforcing steel corrosion or vehicle collision. Using FRP materials has many advantages than any other strengthening method. This study consists of reviewing relevant guidelines, codes, standard practices and manufacturer's specifications that deals with FRP strengthening of damaged concrete bridges based on both U.S and international sources. Based on literature review, the available design guidelines are summarized and compared. Comparison includes calculation of flexural and shear strength based on reviewed code provisions for an example problem. Design code recommendations are made based on the comparative study.

Keywords: FRP strengthening, Bridge repair, FRP design guides, FRP design guide comparison

INTRODUCTION

Fiber Reinforced Polymer (FRP) is a composite material manufactured in the form of polymer matrix reinforced with fibers. Common available fibers are glass, carbon, or aramid, and polymers made up of epoxy, vinyl ester or polyester. FRP composite wrapping is a highly promising structural strengthening material and has been successfully used for strengthening of structures. FRP wrapping has more advantages than adding conventional reinforcement or steel plates to increase strength of structures; it is lighter in weight, non-corrosive in nature and has significant load capacity. The installation of FRP laminates is faster, simpler and less labor intensive, compared to adding structural steel or casting additional reinforced concrete. Use of FRP wrapping for in-service bridge repair or strengthening is economic, where prolonged construction time may lead to transportation difficulties.

The Texas Department of Transportation (TxDOT) manages over 50,000 bridges overall. This constitutes approximately 9% of the nation's entire inventory of bridges. Most of them are prestressed concrete type. The department handles a considerable number of bridges that are damaged due to vehicle or vessel collision, reinforcing steel corrosion or fire. TxDOT has been using FRP strengthening of repaired and damaged bridges since 1999. FRP wrapping improves flexural, shear, axial, and torsional strengths, also serviceability of existing or damaged bridges. There are several available design guides, standards, and manufacture's guidelines for FRP strengthening of concrete structures. This paper involves the comparison of the FRP wrap strengthening procedures from some of these available publications for concrete bridges. Various design parameters, failure modes, and debonding criteria are considered for comparison. Appropriate design criteria and suggestions are recommended.

Review of Current Practice

Several standards and guidelines for FRP strengthening of concrete structures from U.S and other countries were located after a through literature review and are listed below:

- ACI 440.2R-08, "Guide for the Design and Construction of Externally-Bonded FRP Systems for Strengthening Concrete Structures", (ACI 2008).
- AASHTO 2012, "Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements", (AASHTO 2012).
- ISIS Canada Design Manual, 2001, "Strengthening Reinforced Concrete Structures with Externally-Bonded Fiber Reinforced Polymers", (ISIS 2001).
- FIB Technical Report Bulletin 14, "Externally Bonded FRP Reinforcement for RC Structures", (FIB 2001).
- CNR 2004, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures – Materials, RC and PC Structures, Masonry Structures (CNR-DT 200/2004).
- NCHRP Report 655, "Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements", (Zureick et al., 2010).

- NCHRP Report 678, “Design of FRP systems for Strengthening Concrete Girders in Shear”, (Belarbi et al., 2011).
- TR55, 2012, “Design Guidance for Strengthening Concrete Structures Using Fiber Composite Materials”, (The Concrete Society, UK 2012).
- Egyptian Code of Practice for the Use of Fiber Reinforced Polymer in the Construction Fields Code No. ECP 208-2005 (Egyptian Code, 2005).

Initially, TxDOT used FRP manufacture’s guidelines to determine FRP system strengths, because there were no other existing codes. In 1998, the MBrace FRP strengthening design guide was developed by the BASF chemical company, and it has been used since then by TxDOT. BASF recently discontinued the MBrace guide and currently recommends the ACI 440 guidelines. In 2001, FIB published a technical report on design and use of externally bonded FRP for reinforced concrete structures (FIB 14 2001). In 2002, ACI published the first edition of its FRP strengthening design guide; it was developed based on the MBrace guide (ACI 440 2008). In 2008, ACI published the second edition of the FRP strengthening guide. Subsequently, other guides were published in Canada (ISIS Canada 2001), Italy (CNR-DT 200/2004) and Egypt (ECP 208-2005). In U.K., the TR55 technical report on FRP strengthening was published first in 2000, with subsequent upgrades (The Concrete Society, UK 2012). AASHTO published the first edition of its guide specifications in 2012, based on NCHRP 655, and NCHRP 688 reports (AASHTO 2012, Belarbi et al 2011, Zureick et al 2010)

All the located guide procedures consider minimum requirements necessary to provide for public safety. Each publication specifies its own partial factor of safety, characteristic values of material properties, design values of material properties, strength reduction factor It results in conservative design and does not allow the maximum utilization of material properties. For flexural design, most guidelines follow trial and error methods to predict the natural axis of the FRP strengthened structures, in the absence of any direct method. In AASHTO, the assumed maximum usable strain at the FRP/concrete interface is specified as0.005; there is no such assumption made in the ACI or other codes. All the publications considered specify different interpolation methods to calculate the compression stress block parameters, and this may result in differences in calculated strengths. The TR55 considers maximum FRP strain of 0.008; if this limit is exceeded, the publications states that the strengthened structure may fail due to separation of the FRP.

In ACI 440 2R-08, the design recommendations are based on limit state method and design based on ACI 318-05 strength and serviceability requirements. Additional load factors are applied to the contribution of the FRP reinforcement. These reduction factors were determined based on statistical evaluation of variability in mechanical properties, predicted versus full-scale test results, and field applications. FRP-related reduction factors were calibrated to produce reliability indexes, typically above 3.5. The moment and shear capacity equations from this code are shown in Eqns. 1 and 2. All parameters are defined in the “Notations” section in the latter part of this paper.

Moment capacity (ACI 440 2R-08)

$$M_r = \Phi \;\; A_{ps} f_{ps} \;\; d_{ps} - \frac{\beta_1 c}{2} \; + \Psi A_f f_{fe} \;\; h - \frac{\beta_1 c}{2} \qquad (1)$$

1. If ε_c is 0.003

β_1 - Stress block factor specified in Article 10.2.7.3 of ACI 318-11.

2. If ε_c is less than 0.003

β_1 – Stress block factor shall be calculated according to the following equation.

$$\beta_1 = \frac{4\varepsilon_c' - \varepsilon_c}{6\varepsilon_c' - 2\varepsilon_c}$$

Shear Capacity (ACI 440 2R-08)

$$\mathrm{V_r} = \Phi \;\; \mathrm{V_c} + \mathrm{V_s} + \Psi_\mathrm{f} \mathrm{V_f} \qquad (2)$$

In which:

$$\mathrm{V_f} = \frac{\mathrm{A_f f_{fe} d_{fv}} \;\; \sin\alpha_f + \cos\alpha_f}{s_f}$$

$$A_f = 2 n_f t_f w_f$$

In AASHTO 2012, the provisions are limited to concrete compressive strength not exceeding 8 ksi. The consideration of service limit states, strength limit states, Extreme-event limit states and fatigue limit state load combinations are considered as per AASHTO LRFD equations. The moment and shear capacities from this code are presented in Eqns. 3, 4 and 5:

Moment capacity (AASHTO)

1. If ε_c is 0.003

$$\mathrm{M_r} = \Phi \;\; \mathrm{A_{ps} f_{ps}} \;\; \mathrm{d_{ps}} - \frac{\beta_1 \mathrm{c}}{2} \;\; + \Phi_\mathrm{f} \mathrm{T_f} \;\; \mathrm{h} - \frac{\beta_1 \mathrm{c}}{2} \qquad (3)$$

β_1 – Stress block factor specified in Article 5.7.2.2 of AASHTO LRFD.

2. If ε_c is less than 0.003

$$\mathrm{M_r} = \Phi \;\; \mathrm{A_{ps} f_{ps}} \;\; \mathrm{d_{ps}} - \mathrm{k_2 c} \;\; + \Phi_\mathrm{f} \mathrm{T_f} \;\; \mathrm{h} - \mathrm{k_2 c} \qquad (4)$$

In which:

$\mathrm{T_f} = \mathrm{b_{frb} N_b}$

$$\mathrm{k}_2 = 1 - \frac{2 \ \frac{\varepsilon_c}{\varepsilon_o} \ - \ \tan^{-1} \ \frac{\varepsilon_c}{\varepsilon_o}}{\beta_2 \ \frac{\varepsilon_c}{\varepsilon_o}^{\ 2}}$$

$$\beta_1 = \frac{\mathrm{Ln} \ \ 1 + \ \ \frac{\varepsilon_c}{\varepsilon_o}^{\ 2}}{\frac{\varepsilon_c}{\varepsilon_o}}$$

Shear capacity (AASHTO)

$$\mathrm{V}_r = \Phi \ \ \mathrm{V}_c \ + \ \mathrm{V}_s \ \ + \ \Phi_f \mathrm{V}_f \qquad (5)$$

In which:

$$\mathrm{V}_f = \frac{\mathrm{A}_f \mathrm{f}_{fe} \mathrm{d}_{fv} \ \ \sin \alpha_f + \ \cos \alpha_f}{s_f}$$

$$A_f \ = 2 n_f t_f w_f$$

In FIB 14, design calculations are based on analytical or empirical models. Design procedure consists of a verification of both SLS and ULS. Material safety partial factors are used in this method to estimate structural strength. The SLS verification normally concerns stresses, creep, deformation and cracking. In ULS, the different failure modes that may occur need to be considered. The moment and shear capacities from this code are presented in Eqns. 6 and 7, respectively:

Moment capacity (FIB)

$$M_r = \ A_{ps} \, f_{ps} \ \ d_{ps} - \ k_2 c \ \ + \ A_f \, E_f \varepsilon_f \ \ h - \ k_2 c \qquad (6)$$

In which:

$$k_2 = \begin{array}{ll} \dfrac{8 - 1000 \ \varepsilon_c}{4 \ \ 6 - 1000 \varepsilon_c} & for \ \varepsilon_c \ \leq 0.002. \\ \cdot & \\ \dfrac{1000 \varepsilon_c \ \ 3000 \varepsilon_c - 4 \ \ + 2}{2000 \ \varepsilon_c \ \ 3000 \varepsilon_c - 2} & for \ 0.002 \ \leq \ \varepsilon_c \ \leq 0.0035 \end{array}$$

Shear capacity (FIB)

$$V_r = \min \ V_c + \ V_s + \ V_f \, , V_{max} \qquad (7)$$

In which:

$$V_f \ = 0.9 \ \varepsilon_f \ E_f \ \rho_f b_w \ d \ \ \ \cot\theta_f + \ \cot \alpha_f \ \ \ sin \ \alpha_f$$

In TR 55, the equation used for the design of FRP strengthening system is based on the parabolic-rectangular-stress-strain relationship for concrete in compression. Partial safety factor of concrete or reinforcement are calculated based on design situations. It is also possible in some situations for the ultimate strain in the FRP to govern failure of a strengthened structure. The moment and shear capacities from this code are presented in Eqns. 8 and 9, respectively:

Moment capacity (TR55)

$$M_r = M_{existing} + A_f \varepsilon_{fe} E_f \, z \tag{8}$$

In which:

ε_{fe} = Design strain value of FRP.

z = Prestressed steel lever arm.

Shear capacity (TR55)

$$V_r = \frac{A_{sw}}{s} z f_{ywd} \cot\theta + \frac{A_{fw}}{s_f} \; d_f - \frac{n_s}{3} l_{t,max} \cos\beta \;\; E_f \varepsilon_{fe} \; \sin\alpha_f + \cos\alpha_f \tag{9}$$

In CNR 2004, strength and strain properties of FRP materials used for strengthening, as well as those of existing materials, are described by the corresponding characteristic values. The flexural analysis of FRP strengthened members can be performed by using strain compatibility and force equilibrium. The stress at any point in a member must correspond to the strain at that point; the internal forces must balance the external load effects.

In ISISCanda, the initial strains are usually assumed to be negligible and the stress and strain distribution is approximated. The moment and shear capacities from this publication are shown in Eqns. 19 and 11, respectively:

Moment capacity:

$$M_r = \frac{1}{\gamma_{Rd}} \; \Psi \cdot b \cdot c \cdot f_{cd} \; d_{ps} - \lambda \cdot c \;\; + A_f \cdot \sigma_f \cdot d_1 \tag{10}$$

Shear capacity:

$$V_r = min \;\; V_c + V_s + V_f \, , V_{max} \tag{11}$$

In which:

For side bonding configuration

$$V_f = \frac{1}{\gamma_{Rd}} \cdot min \cdot \; 0.9d_{ps}\,, d_{fv} \;\; \cdot f_{fe} \; \cdot 2 \; \cdot t_f \; \cdot \frac{\sin\alpha_f}{\sin\theta_f} \cdot \frac{w_f}{s_f}$$

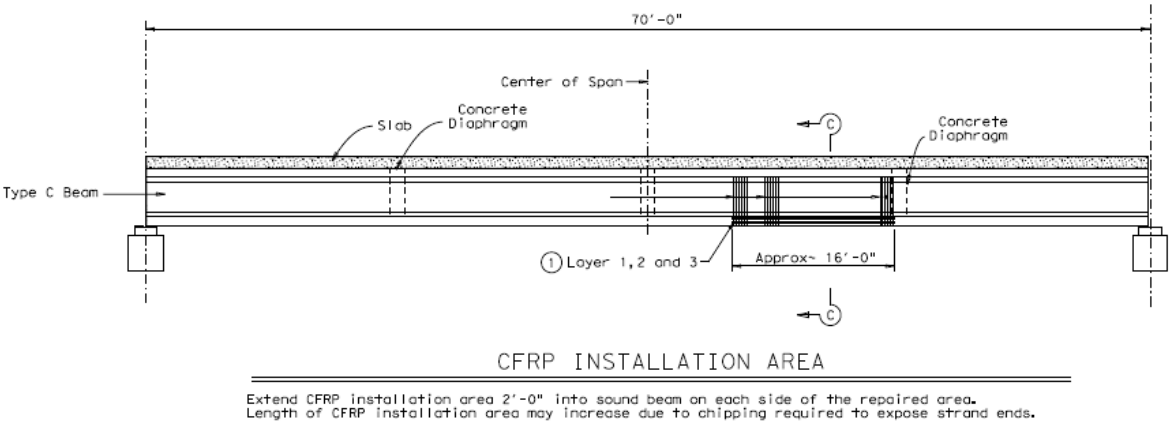
For U Wrapped or Completely wrapped configuration

$$V_f = \frac{1}{\gamma_{Rd}} \cdot 0.9d \;\cdot f_{fe} \cdot 2 \cdot t_f \cdot \;\cot\theta_f + \cot\alpha_f \;\; \cdot \frac{w_f}{s_f}$$

FLEXURAL STRENGTHENING

Bonding FRP reinforcement to the tension face of a concrete flexural member with fibers oriented along the length of the member will provide an increase in flexural strength up to 40%. To examine the flexural strength values predicted by the various codes, an example of an FRP strengthened I-girder was selected herein, as follows:

A prestressed simply supported TxDOT type C I-Girder has a depth of 47 in. from top of slab and effective top width of 87 in, as shown in Fig. 1a, 1b, and 1c. An analysis of the existing repaired beam indicates that the beam has strength of 3419 kip-ft. and additional 250 kip-ft. is required to avoid girder failure. Girder was strengthened with a 3 layer set up of FRP laminate. Various pertinent information on the girder are presented in Table 1.



Extend CFRP installation area 2'-0" into sound beam on each side of the repaired area.
Length of CFRP installation area may increase due to chipping required to expose strand ends.

Fig. 1a. TxDOT type C I-girder (Longitudinal view)

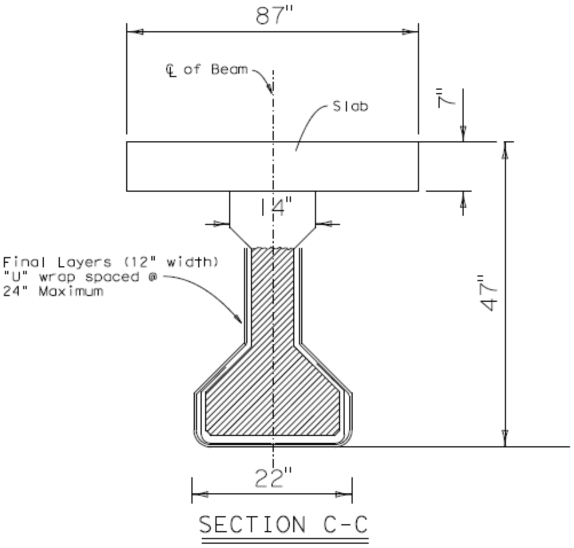


Fig. 1b. TxDOT type C I-girder (Cross section)

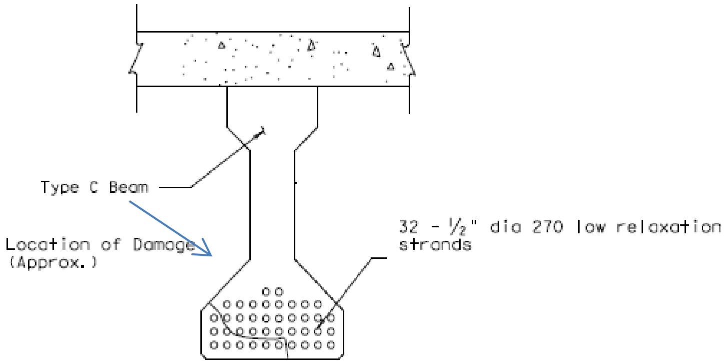


Fig. 1c. TxDOT type C I-girder (Cross section)

Table 1: Girder and FRP Information

Existing beam details	
Compressive strength of concrete, f'_c	5000 psi
Ultimate strength of strands	270 ksi
No. of 0.5 in diameter strands used	28
FRP Physical Properties	
Thickness per ply, t_f	0.0065 in
Ultimate tensile strength, f_{fu}*	550 ksi
Rupture strain, ε_{fu}*	0.017 in./in.
Modulus of elasticity of FRP laminates, E_f	33000 ksi

Table 2: Comparison of Calculated Moment Strengths

No.	**Code, Standard or Guideline**	**Flexure Strength (kip-ft.)**	**Variation (%) Based on ACI 440 value**
1	ACI 440 2R	3714*****	-
2	AASHTO/NCHRP	4012	+8.0
3	ECP	3612	-2.7
4	CNR	3401	-8.4
5	FIB	3523	-5.1
6	ISIS	3284	-12.0
7	TR	3718	+0.1
8	MBrace	3804	+2.5

**Percentage difference compare to ACI 440 2R*

The stress diagram varies from code to code, and this results in minor variation in final flexural strength values. Other reasons for this variation in design flexural strength are due to calculation of moment arm and partial safety factors. As shown in Table 2, the maximum flexural strength was obtained through the AASHTO 2012 code, and the minimum through the ISIS code.

SHEAR STRENGTHENING

Shear strengthening of reinforced concrete members using FRP wrap may be provided by bonding the external reinforcement with the principal fiber in the direction of maximum principle tensile stresses to maximize effectiveness of FRP reinforcement. Calculation of strain is based on shear crack and FRP laminate placed at an angle with horizontal. The nominal shear strength of an FRP strengthened concrete member can be determined by adding the contribution of the FRP external shear reinforcement to the contributions of the reinforcing steel and the concrete. The following example was used to compare shear strengths from the various codes. The TxDOT type C I-girder shown in Fig. 1 was considered herein, with pertinent information provided in Table 1. The girder has existing nominal shear strength of 205 kips (ΦV_n = 153 kips). Analysis of the beam indicates that it does not satisfy shear requirement of 180 kips. Addition external FRP shear reinforcement was provided to increase the shear capacity. The girder and slab dimensions are shown in Fig. 1.

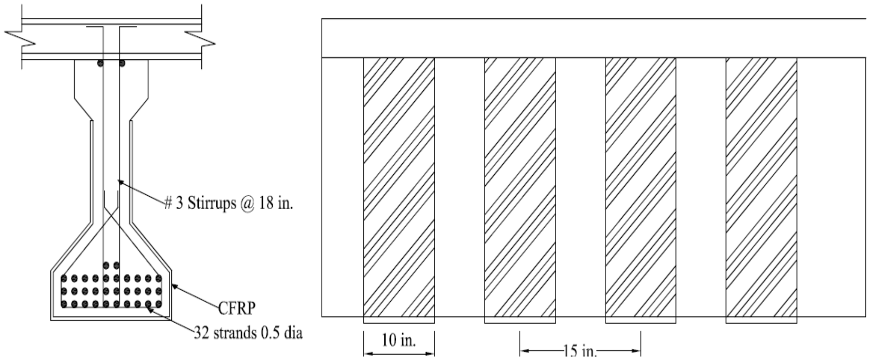


Fig. 2. TxDOT type C I-girder

Table 3. **Comparison of Calculated Shear Strengths**

No	**Code, Standard or Guideline**	**Shear Strength (Kips)** ΦV_n	**Variation (%) Based on ACI 440 value**
1	ACI 440 2R	183*	-
2	AASHTO/NCHRP	188	+2.7
3	ECP	178	-2.7
4	CNR	187	+2.2
5	FIB	196	+7.1
6	ISIS	182	-0.5
7	TR	182	-0.5
8	MBrace	183	0

**Percentage difference compare to ACI 400 2R procedure*

From the results in Table 3, it can be seen that the FIB code yields the maximum shear strength, while the Egyptian code results in the minimum shear strength. However, there is some minor variation among some of the calculated values.

Axial and torsional strengths predicted by the various codes were also evaluated. However, these results are not presented herein. There were minor variations among the various predicted values.

CONCLUSION AND RECOMMENDATION

The following conclusions and recommendations may be made based on the review results

- A number of design codes, standards and guidelines are available worldwide that deal with FRP strengthening of concrete structures. They present equations for the prediction of flexural, shear, axial and torsional strengths of such strengthened structures. Some of these documents contain different stress distributions for the flexural strength determination. All the flexural equations add a component caused by the FRP addition. There are only a few codes available for predicting torsional strengthening capacity.
- The maximum flexural strength is obtained through the AASHTO 2012 code, and the minimum through ISIS code. However, the variations in moment capacities are moderate (maximum 12%).
- The FIB code yields the maximum shear strength, while the Egyptian code resulted in the minimum shear strength. However, the variations in shear capacities are small (maximum 7%).
- It is recommended that the ACI 440 guidelines be followed for designing FRP strengthening systems for concrete bridges. The MBrace guidelines have been discontinued by the publishers. They are currently referring to the ACI 440 guidelines. The ACI guidelines are reasonable and predict various strength values that are consistent with other standards.

NOTATIONS

A_f - Area of FRP external reinforcement, in.^{2}
A_{ps} - Area of pre-stressed reinforcement in tension zone, in.^{2}
A_{sw} - Area of one stirrup leg, in.^{2}
b - Width of compression face of member, in.
b_w - Width of section web, in.
c - Distance from extreme comp. fiber to the neutral axis, in.
d - Effective depth of concrete section, in.
d_{fv} - Depth of FRP shear reinforcement, in
d_{ps} - Distance from extreme compression fiber to centroid of prestressed reinforcement, in.
E_f - Tensile modulus of elasticity of FRP, psi

f_{cd} - Design concrete compressive strength, psi
f_{fe} - Effective stress in the FRP; stress level attained at section failure, psi
f_{fu} - Design ultimate tensile strength of FRP, psi
f_{ps} - Stress in prestressed reinforcement at nominal strength, psi
f_{ywd} - Design yield strength of traverse steel reinforcement, psi
h - Overall thickness or height of a member, in.
k_2 - Multiplier for locating resultant of the compression force in the concrete
M_r - Factored moment capacity of the section, k-ft
n_f - Number of piles of FRP reinforcement
s_f - Center-to-center spacing FRP, in.
t_f - Nominal thickness of one ply of FRP reinforcement, in.
V_c - Nominal shear strength provided by concrete with steel flexural reinforcement, N.
V_f - Nominal shear strength provided by FRP stirrups, lb (N)
V_s - Nominal shear strength provided by steel stirrups, lb (N)
V_r - Factored shear capacity, kips
w_f - Width of FRP reinforcing plies, in.
α_f - Angle of inclination of FRP with respect to the longitudinal axis of the member, deg.
β_1 - Ratio of depth of equivalent rectangular stress block to depth of the neutral axis for concrete
θ - Angle of diagonal crack with respect to the member axis, assumed equal to 45 deg
γ_{Rd} - Partial factor for resistance models.
ε_c - Strain level in top surface of concrete, in./in.
ε'_c - Maximum strain of unconfined concrete corresponding to f'_c in/in.
ε_{fe} - Effective strain level in FRP reinforcement attained at failure, in/in.
ε_{fd} - Debonding strain of externally bonded FRP reinforcement, in./in.
ε_{fu} - Design rupture strain of FRP reinforcement, in./in.
ε_o - the concrete strain (in/in) corresponding to the maximum stress of the concrete stress-strain curve
λ - Resultant of the compression stress

ρ_f - FRP reinforcement ratio
σ_f - Stress in FRP reinforcement
Φ - Resistance factor
Φ_f - FRP resistance factor
Ψ - Resultant of the compression stress

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