FYFE Co. LLC
Tyfo® Fibrwrap® Systems

June 26, 2008

Mahmut Ekenel, Ph.D.
ICC EVALUATION SERVICE, INC.
Los Angeles Business/Regional Office
5360 Workman Mill Road
Whittier, California 90601

RE: Response to the ICC staff letter of June 2nd 2008
Proposed Changes to Section 7.3 (Design Criteria) of AC125

Dear Dr. Ekenel:

ICC Evaluation Services solicited public comments on proposed changes to Section 7.3 (Design Criteria) of AC125. This letter is a response from Fyfe Company.

General Comments:

- The proposed changes are based on the ACI 440.2R-02 guide. The design philosophy of AC125 is fundamentally different from that of ACI 440.2R-02. It is, therefore, our opinion that changes to AC125 should not be based on ACI 440.2R-02.
- The referenced ACI 440.2R-02 guide underwent significant changes and a new revision will be out soon (ACI 440.2R-08). It is not advised to make changes based on a soon-to-be an outdated guideline.

Comments on Item #1:

- The design criteria of AC125 are based on four main assumptions listed under section 7.3.2.1. The two key assumptions are "*b) the bond between the FRP and the substrate remains perfect; c) the maximum usable compressive strain in the concrete is 0.003*". The limiting ratio of 0.75 in Eq. 1 of Section 7.3.2 is not intended to specifically safeguard against bond failure. It is rather an overall safety factor to be used in combination with the IBC/UBC strength reduction factors. These assumptions have been validated by structural testing as per AC 125.
- ACI 440.2R-02 guide acknowledges that the κ_m factor is not based on sound bond mechanics and does not account for the stiffness of the member to which FRP reinforcement is bonded.
- The revised guide (ACI 440.2R-08) will replace limiting FRP strain using the κ_m factor with an expression (Eq. 10-2) for deponding strain (ε_{fd}) which accounts for the properties of the substrate namely, concrete compressive strength.
- Table 1 demonstrates the significant differences in strain limiting ratios between the 2002 "κ_m" factor and the 2008 "ε_{fd}" strain limit. Values in Table 1 are calculated for two of our most commonly used products, one is a primary carbon fiber system and the other is a primary glass fiber system. The properties of both composite materials are provided in Table 2. Concrete compressive strength of 3000 psi was used.

- For the above reasons, it is our opinion that the treatment of bond should either be left as is "designer to satisfy the assumption of plane sections remain plane and that the strains are compatible," or approaches similar to those described in Appendices A1 and A3 to Chapter 4 of the 2001 European Technical Report fib Bulletin 14 may be suggested.

Table 1: Comparison between bond limiting strains according to 2002 and 2008 ACI 440.2R

# of Plies	Limiting strain in CFRP			Limiting strain in GFRP		
	440.2R-02	440.2R-08	2008/2002	440.2R-02	440.2R-08	2008/2002
1	0.00900	0.00610	0.68	0.01509	0.01044	0.69
2	0.00749	0.00431	0.58	0.01351	0.00738	0.55
3	0.00500	0.00352	0.70	0.01193	0.00603	0.51
4	0.00375	0.00305	0.81	0.01035	0.00522	0.50
5	0.00300	0.00273	0.91	0.00877	0.00467	0.53
6	0.00250	0.00249	1.00	0.00733	0.00426	0.58
7	0.00214	0.00230	1.08	0.00628	0.00395	0.63
8	0.00187	0.00216	1.15	0.00550	0.00369	0.67

Table 2: Properties of CFRP and GFRP used in Table 1 Calculations

Property	Carbon	Glass
E_f (psi)	13,900,000	3,790,000
ε_{fu} (in/in)	0.01	0.022
t_f (in)	0.04	0.05

Comments on Item #2:

- While we agree with the rational for the proposed change, we would like the language to distinguish the case of earthquake loading from all other loading conditions. Plastic deformations in steel under earthquake loads is not only acceptable, it is desired. If clear language is in place to prevent confusing the design professionals on this issue, we support the proposed 80% f_y limit on steel stress under service loads.

Thank you for the opportunity to comment.

Scott F. Arnold, P.E.
Vice-President
Fyfe Company

References

AC 125 (2007). Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer Composite Systems. ICC Evaluation Service.

ACI 440.2R-08 (2008). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. American Concrete Institute.

ACI 440.2R-02 (2002). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. American Concrete Institute.

CEB-FIP (2001). Externally Bonded FRP Reinforcement for RC Structures. Technical Report Bulletin 14.

Fyfe Company (2007). Design Manual for the Tyfo Fibrwrap Systems.

DESIGN ISSUES AND RECOMMENDED REVISIONS TO AC 125

"Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer (FRP) Composite Systems"

Prepared by

Silvia Rocca, PhD, Nestore Galati, PhD, and Tarek Alkhrdaji, PhD, PE

Structural Group, Inc.
Hanover, Maryland

June 27th, 2008

This document is provided to ICC-ES in response to its staff letter dated June 2, 2008, requesting public input for issues of concern with AC 125 (Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer Composite Systems). Below is list of these issues, our technical input, as well as proposed changes to resolve them. We have also included additional issues that we believe are of concern as well. Please note that the proposed changes to AC 125 were based on the latest ACI 440.2R-08 design guideline (in print).

It is our professional opinion that the two issues raised by ICC-ES staff as well as the additional issues provided by SG are of primary nature and can affect the performance and safety of FRP strengthened structures. In the past few years, we have experienced many situations in which the design engineer based the design of externally bonded FRP on ACI 125. Many professionals are using AC 125 as design guidelines, not understanding that the acceptance criteria is a process for material qualification, verification, and approval. We strongly recommend that the AC 125 be revised to clearly state its purpose and that the design should be based on the latest applicable guidelines by ACI 440.2R. The latter is the only consensus based design guide for FRP that was based on input from academics, professional engineers, and manufacturers from around the world. ACI 440.2R-08 is currently the only document that considers environmental, material, construction, safety, and strength reliability in the design of externally bonded FRP composites.

ICC-ES Staff Letter Issues of Concern:

1. Based on the information in ACI 440.2R-02 (Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures), Equation 1 in Section 7.3.2.1 of AC 125 (Flexural Strength Enhancement) may lead to underestimating the stress levels of the FRP reinforcement in cases where multiple layers of reinforcement are used for strengthening applications. Equation 1 in Section 7.3.2.1 of AC 125 limits the stress level of the FRP composite to less than or equal to 0.75 times the ultimate tensile stress of the composite ($f_{jt} = E_f \cdot \varepsilon_f \leq 0.75 f_{uj}$). ACI 440.2R-02 also requires that a limitation be applied to maximum strain level. In accordance with Sections 9.2.2 and 9.2.3 of this guideline, the stress level in the FRP reinforcement must be limited to $f_{fe} = E_f \cdot \varepsilon_{fe}$ (Equation 9-4), where $\varepsilon_{fe} = \varepsilon_{cu}[(h - c)/c] - \varepsilon bi \leq k_m \varepsilon_{fu}$ (Equation 9-3). The k_m factor, as defined in Equation 9-2 of ACI 440.2R-02, is a function of the number of layers used, modulus of elasticity of composite material and thickness of the composite; furthermore, the maximum value must not be greater than 0.90. Based on our observations, the k_m factor may get lower than 0.75 for multiple layer

FRP applications. ICC-ES staff would like to receive public input as to whether the limitation in Equation 1 of AC125 is conservative enough for multiple layer strengthening applications.

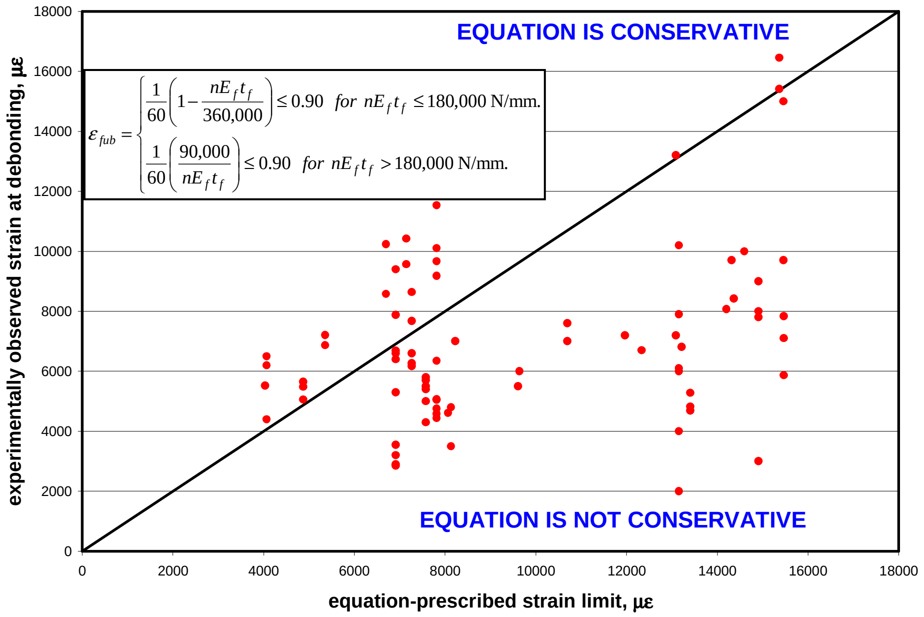
1.1. Technical Input:

Even though AC 125 criteria states that “The design stresses and strains shall be based on a characteristic value approach verified by test data,” the current criteria does not consider the effects of material stiffness and the use of multiple plies on the effective (achievable) design stress of FRP.

ACI 440 Subcommittee F, Task Group on FRP bond, recently evaluated experimental results from an extensive database of flexural beams with externally bonded FRP. The database includes 282 tests from 42 citable sources. Only 102 of these tests reported sufficient information than can be used to assess FRP debonding strains and behavior. From these 102 tests:

- 18 reported cover delamination failures;
- 72 reported FRP delamination failures;
- 12 reported rupture of the FRP;
- 64 beams had depths less than 12 in.;
- The majority of beams had span-to-depth ratios between 2 and 5, and only 30 beams had span-to-depth ratios greater than 6;
- Concrete compressive strength varied from 3500 psi to 8500 psi; and
- FRP stiffness as measured by the product of $nE_f t_f$ varied from 85 to 1750 kip/in.

Figure 1 shows a comparison between experimental and predicted FRP debonding strains based on the current ACI 440.2R-02 provisions for FRP strain limit (κ_m). In general, existing provisions do not account for the strength of the concrete substrate in any manner. The Task Group has concluded that the present 440.2R-02 provision to account for FRP delaminating/debonding is inadequate and may not be conservative.



$$\varepsilon_{fub} = \begin{cases} \frac{1}{60}\left(1-\frac{nE_f t_f}{360{,}000}\right) \le 0.90 & for\ nE_f t_f \le 180{,}000 \text{ N/mm.} \\ \frac{1}{60}\left(\frac{90{,}000}{nE_f t_f}\right) \le 0.90 & for\ nE_f t_f > 180{,}000 \text{ N/mm.} \end{cases}$$

Figure 1 - Comparison of Experimental to Theoretical Ultimate Strain of FRP According to ACI 440.2R-02

A more rational approach, supported by experimental evidence, was proposed by ACI committee 440, Subcommittee F. To prevent crack-induced debonding failure mode, the task group on FRP bond recommended that the effective design strain in FRP reinforcement ε_{fd} should be limited as follows:

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{nE_f t_f}} \leq 0.9\varepsilon_{fu}$$

Where,

ε_{fd} = debonding strain of externally bonded FRP reinforcement;
f'_c = specified compressive strength of concrete;
n = number of plies of FRP reinforcement;
E_f = tensile modulus of elasticity of FRP;
t_f = nominal thickness of one ply of FRP reinforcement;
ε_{fu} = design rupture strain of FRP reinforcement.

The equation above was based on a Subcommittee evaluation of a large number of full scale flexural beam tests exhibiting FRP debonding failure mode. Furthermore, it was calibrated using experimental values of FRP strains measured at FRP debonding. This equation was included in the latest draft for ACI 440.2R-2008 and has been reviewed and approved by TAC for publication.

Figure 2 shows the ratio of debonding strain to the design rupture strain of FRP ($\varepsilon_{fd}/\varepsilon_{fu}$) versus the stiffness of the FRP material, according to the provisions by ACI 4402R-08 and AC 125. The stiffness of the externally bonded FRP material is represented as the product of the number of plies (n), nominal thickness of one ply of FRP reinforcement (t_f), and the tensile modulus of elasticity of the FRP (E_f). In the figure below it can be observed that, as opposed to what AC 125 guideline prescribed, the use of multiple plies and/or stiffer material can reduce the effective design stress of FRP significantly. In addition, the task group has concluded that the concrete strength influences the bond strength and thus FRP stress for debonding failure modes as well.

Figure 3 shows the ratio of predicted nominal to experimental bending capacities in function of the stiffness of the FRP material (product $n \cdot t_f \cdot E_f$) according to ACI 440.2R-08 and AC 125, and incorporating the applicable strength reduction factors for each. Note that in several cases the predicted capacity per AC 125 can be 80% higher than that per ACI 440.2R-08. The current AC 125 equation is unsafe if used for design.

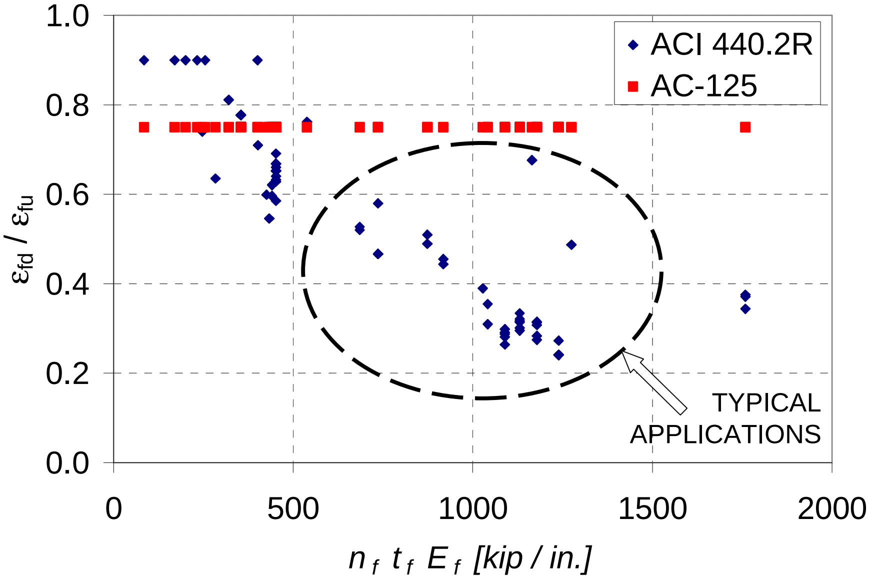


Figure 2 – Effect of Axial Stiffness (Multiple Plies) on FRP Strain at Failure

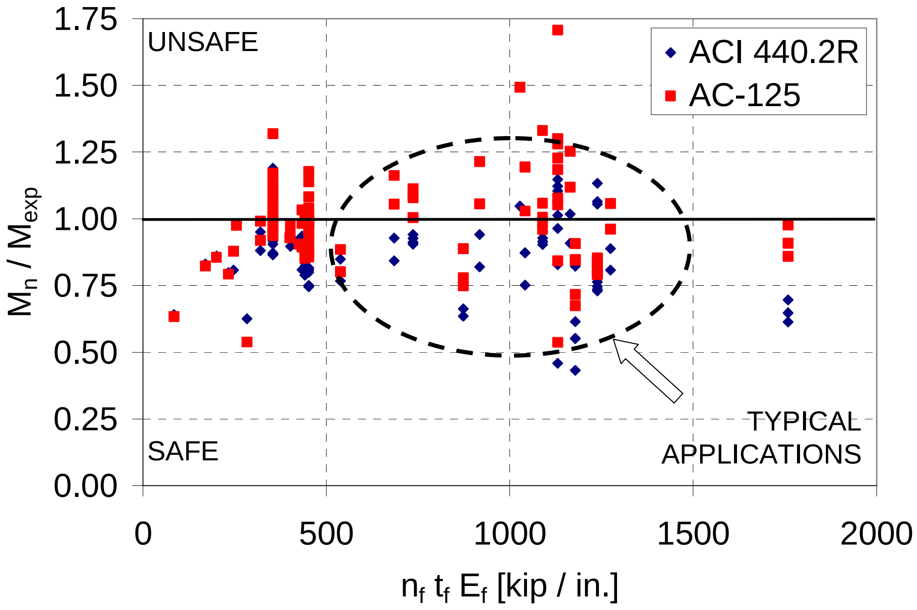


Figure 3 – Ratio of Predicted Flexural Nominal Capacities to Experimental Results (no strength reduction factor included)

1.2. Proposed Changes AC 125:

In light of the evidence presented in Section 1.1, we recommended that AC 125 modify the current approach for predicting FRP design strain limits and to make it consistent with ACI 440.2R-08, as follows (proposed changes underlined).

7.3.2.1 Flexural Strength Enhancement: Fiber-reinforced polymer (FRP) composite material bonded to surfaces of concrete or masonry may be used to enhance the design flexural strength of sections by acting as additional tension reinforcement. In such cases, section analysis shall be based on normal assumptions of a) plane sections remain plane after loading; b) the bond between the FRP and the substrate remains perfect; c) the maximum usable compressive strain in the concrete is 0.003; d) FRP has a linear elastic behavior to failure. ~~The enhancement of axial force provided by a fiber element is~~

~~$f_{jt} = E_f \varepsilon_f \leq 0.75 f_{uj}$ (1)~~

~~where ε_f is the strain in the concrete or masonry to which the fiber is bonded at the section strength in the direction of the member axis.~~

The flexural strength of a section depends on the controlling failure mode. The following flexural failure modes should be investigated for an FRP-strengthened section (ACI 440.2R-08):

- 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).

Failure of the externally bonded FRP is assumed to occur if the strain in the FRP reaches its design rupture strain ($\varepsilon_f = \varepsilon_{fu}$) before the concrete reaches its maximum usable strain.

The effective strain in FRP reinforcement should be limited to the strain level at which debonding may occur ε_{fd} as defined in Eq. (1):

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \qquad (1)$$

Checks must be done to ensure that the strain in the member is at least as high as what is assumed in design. Fibers must not have a misalignment of more than 5 degrees.

Dependable flexural strengths shall be determined by multiplying the nominal flexural strength, including the effects of fiber according to Equation (1), by the appropriate flexural strength reduction factor according to the IBC or UBC.

2. Section 9.4 of ACI 440.2R suggests that, to avoid inelastic deformations of the reinforced concrete members strengthened with external FRP reinforcement, the existing internal steel reinforcement should be prevented from yielding under service load levels; hence, the stress in the steel under service load should be limited to 80 percent of the specified yield strength ($f_{s,s} \leq 0.80 f_y$). This 80 percent strength reduction requirement does not exist in AC125. ICC-ES staff would like to get public

input as to whether AC125 needs to be revised to address the potential for inelastic deformations, and whether the 80 percent yield strength limitation should be adopted for AC125.

2.1. Technical Input:

Based on the experimental and analytical work conducted by El Tawi et al. (2001) on reinforced concrete beams strengthened with FRP, and subject to static and accelerated fatigue loading, the steel reinforcement in these members at the time of FRP strengthening may be significantly overstressed. This level of stress is the resultant of time-dependent redistribution of stresses due to creep, shrinkage, and cyclic fatigue. Since CFRP ruptures at a strain that is considerably higher than the steel yield strain, it is possible that steel reinforcement in a beam repaired/strengthened in flexure yield at service conditions.

Steel yield under service conditions must be avoided as it can cause reduce the effective stiffness of the member at service, and can result in excessive and permanent deformations, both of which may lead to severe serviceability problems. Therefore, based on experimental evidence, ACI 440.2R-08 limits the steel stress under service conditions to 80 percent of the yield strength. We strongly suggest that similar criterion is added to AC125 to ensure adequate service behavior.

2.2. Proposed Changes AC125:

Serviceability provisions given in ACI 440.2R-02 Section 9.4 remained unaltered in the new revised guideline (ACI 440.2R-08). Based on the information presented in Section 2.1, it is recommended that AC 125 adopt similar serviceability limitations to FRP strengthened reinforced concrete beams as follows.

New Sub-Section within 7.3.2: Serviceability

The serviceability of a member (deflections, crack widths) under service loads should satisfy applicable provisions of ACI 318 (or building code). The effect of the FRP external reinforcement on the serviceability can be assessed using the transformed-section analysis.

The stress in the steel reinforcement under service load should be limited to 80% of the yield strength. In addition, the compressive stress in concrete under service load should be limited to 45% of the concrete compressive strength.

$$f_{s,s} \leq 0.80 f_y \qquad (2a)$$

$$f_{c,s} \leq 0.45 f_c' \qquad (2b)$$

<u>Additional Issues of Concern</u>:

1. In general AC 125 criteria do not provide a clear definition for the design tensile properties which are typically different from the average tensile test results obtained using ASTM 3039. FRP materials are brittle in nature and test results can be influenced by many factors including, fiber alignment, material, preparation, and testing variables. A design based on average properties does not satisfy the statistical reliability approach used by ACI to establish the appropriate strength reduction factors, which are also indirectly adopted by building codes. Using average values in design will require adjustment of the strength reduction factors (ϕ factors) to account for reduced reliability.

    1.1. Technical Input:

    1.1.1. A more material reliability based approach is given in ACI 440.2R, which prescribes guaranteed design values for ultimate stress and strain defined as the mean value of 20 test specimens minus three times the standard deviation (Ave – 3SD, see ACI 440.2R-02, Section 3.3.1). These statistically based design tensile properties provide a 99.87 percent probability that the design values are exceeded. The same characteristic values are also used by some European design codes, such as *fib*.

    1.2. Proposed Changes AC 125:

    We recommend that AC 125 adopt the statistical approach for calculating the design properties of FRP reinforcement, as discussed above and presented inACI 440.2R.

2. Based on reliability analysis, ACI committee 440.2R, included an additional strength reduction factor ψ_f that is used to improve the reliability of strength prediction of FRP strengthened members, as follows:

    ψ_f = 0.85 for flexure;
    ψ_f = 0.85 for shear for three-sided FRP U-wrap or two-sided strengthening schemes;
    ψ_f = 0.95 for fully wrapped sections.

    2.1. Technical Input:

    Similar to Figure 3, Figure 4 shows the ratio of predicted nominal to experimental bending capacities according to ACI 440.2R-08 and AC 125 but includes the global strength reduction factors and the additional (FRP contribution only) reduction factors for each method. As shown in Figure 4, with o additional reduction factor applied to FRP contribution, AC 125 overestimates the experimental bending capacity up to 60 percent, which might lead to failure under service loads.

    2.2. Proposed Changes AC 125:

    We recommend that AC 125 adopt the reliability approach of ACI 440.2R. This would allow for maintaining the same reliability index as the one in the building codes without modifying the strength reduction factors. Alternatively, using the current approach of AC 125, new strength reduction factors should be defined in order to assure the same reliability levels as the current building codes.

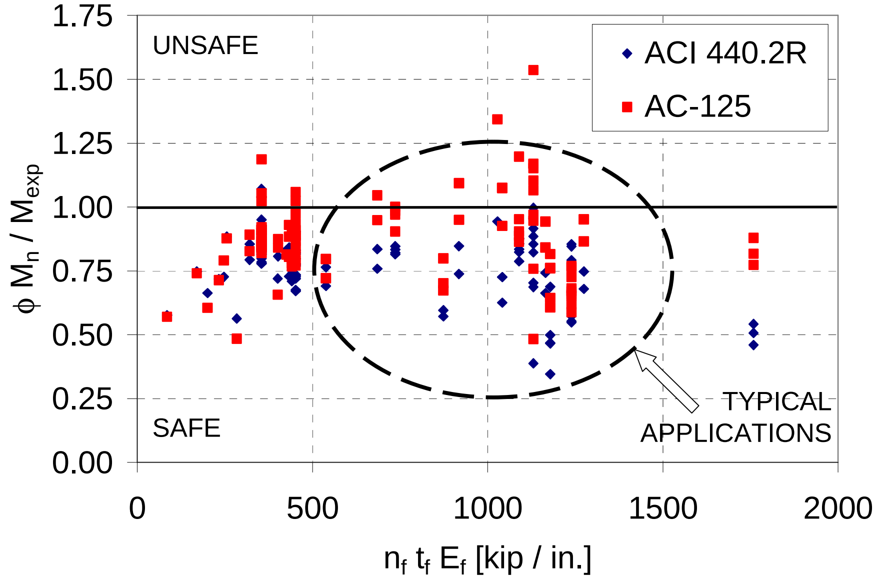


Figure 4 – Ratio of Predicted Flexural Design Capacities to Experimental Results (including applicable strength reduction factors)

3. The guideline does not account for time-dependent behavior: creep-rupture and fatigue. To avoid creep-rupture under sustained stresses or failure due to cyclic stresses and fatigue of the FRP reinforcement, the stress levels in the FRP under these conditions should be checked.

3.1 Technical Input:

Research has indicated that glass, aramid, and carbon fibers can sustain approximately 0.3, 0.5, and 0.9 times their ultimate strengths, respectively, before encountering a creep-rupture problem (Yamaguchi et al. 1997; Malvar 1998). To avoid failure of an FRP-reinforced member due to creep-rupture and fatigue of the FRP, stress limits for these conditions should be imposed on the FRP reinforcement.

3.2. Proposed Changes AC 125:

To avoid creep-rupture of the FRP reinforcement under sustained stresses or failure due to cyclic stresses and fatigue of the FRP reinforcement, the stress levels in the FRP reinforcement under these stress conditions should be limited to the following (per ACI 440.2R). Because these stress levels will be within the elastic response range of the member, the stresses can be computed by elastic analysis.

	Fiber Type		
Stress type	GFRP	AFRP	CFRP
Sustained plus cyclic stress limit	0.20f_{fu}	0.30 f_{fu}	0.55 f_{fu}

4. Section 7.3.2.6 Shear Strength Enhancement: there is very little provided on the background for the listed equation. No reasoning or reference is provided for the given equation. The use of the entire depth of the member (*H*) instead of the effective depth (*d*) in this equation is unclear and can only be true for flexural members with no slab attached to them (more likely experimental condition only). In

addition, the equations are independent from the concrete strength of the substrate, the number of plies and the development of FRP sheets and thus does not account for the use of multiple plies of stiff material. Overall, the document is very vague on providing complete and consistent provisions in this section.

4.1. Proposed Changes AC 125:

We recommend that AC 125 adopt the design approach of ACI 440.2R that account for the effect of concrete strength and the use of multiple plies of FRP.

5. Axial Load Capacity Enhancement (Section 7.3.2.3)
   4.1. Technical Input:
      4.1.1. The guideline does not provide a limiting value for the effective FRP hoop strain at failure. It is a generally acknowledged fact based on conducted research (Teng et al., 2002; Lam and Teng, 2003a) that the level of the FRP failure strain is less than the one observed in pure tensile test (mechanical characterization) because the FRP is subjected to combined tensile stress and laterally applied pressure resulting from concrete dilation.

      4.1.2. The expression of confining pressure f_l (Eq. 4: $f_l = 0.26\ \rho_{sj}\ f_{uj}\ sin^2\theta$) shows a reduction factor of 0.26. It is not clear what the basis is for this coefficient.

      4.1.3. The proposed expression (equation 6: $f'_{cc} = f'_c\ (1 + 1.5\rho_{sj}\ cos^2\theta)$) for the computation of confined concrete compressive strength f'_{cc} does not seem to be appropriate given existing documented evidence. The increment of concrete compressive strength (ratio f'_{cc}/f'_c) is not only function of the amount of FRP and geometry of the cross-section, but also on the stiffness of the FRP jacket, stiffness of the concrete, effective strain level of FRP at ultimate, chamfered corner radius, among other parameters. Furthermore, this equation implies a linear variation of the increment of concrete compressive strength ($f'_{cc}\ /f'_c$) to the FRP volumetric ratio (ρ_f). Additionally, in the derivation of the equation for f'_{cc}, values of f'_c = 5.3 ksi, E_f = 3200 ksi, and ε_{fu} = 0.002 were assumed constant limiting its applicability to such values of concrete and FRP material properties.

      4.1.4. There seems to be a misprint in the document regarding the angle θ since this parameter is defined as the angle between the FRP fiber orientation and the longitudinal axis of the member, θ should be greater or equal than 45 degrees, not the opposite.

      4.1.5. The current document does provide a limitation on the long to short side ratio ($H/B \leq 1.5$). Based on available full scale testing, this ratio has been extended to 2.0 in the latest ACI 440.2R-08.

Figure 5 below presents a comparison of the experimental versus the theoretical ratios f'_{cc}/f'_c for selected full-scale reinforced concrete specimens of non-circular cross-section, and whose gross concrete areas vary from 144 in^2 to 1296 in^2 (Kestner et al., 1997; Wang and Restrepo, 2001; Youssef, 2003; Carey and Harries, 2003; Rocca et al., 2007). The label of each data-point corresponds to the value of ρ_f. This figure also shows a “prediction” line based on regression analysis. Note the large scatter of the data. From the figure, it can be clearly seen that the proposed equation cannot be generalized since it is only valid for a particular case.

In conclusion, to allow for proper evaluation of FRP contribution to column confinement, it is suggested that the proposed equations and limitations should be based on more comprehensive and research findings available in the literature. New design guides have been published worldwide in recent years and they should not be ignored (*fib* bulletin 14, 2001; ACI 440, 2008; and Concrete Society TR 55, 2004).

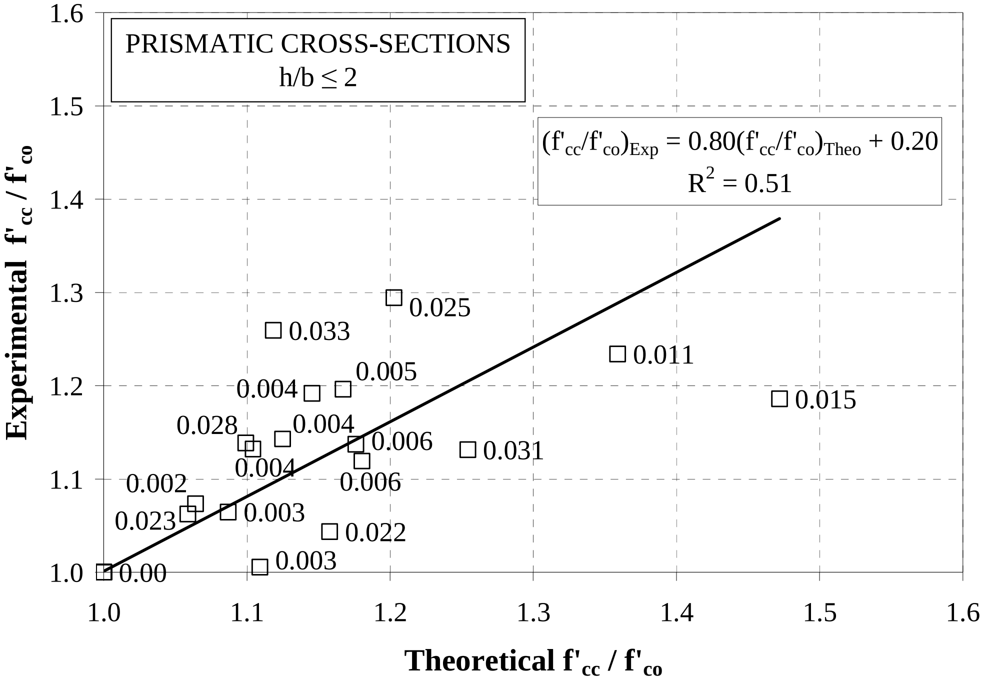


Figure 5 - Comparison of Experimental vs. Theoretical Increment of Concrete Compressive Strength

4.2. Proposed Changes to AC 125:

ACI 440 committee adopted the stress-strain model for FRP-confined concrete by Lam and Teng (2003a,b) based on reasonable predictions including for real size columns, and for allowing the definition of the stress-strain curve of the confined concrete and therefore the construction of interaction diagrams for FRP-confined columns (axial force and bending moment diagrams).

Considering the background presented in Section 4.1, it is suggested to AC 125 that Sections 7.3.2.3.1 and 7.3.2.3.2 of the guideline should be updated following provisions recently approved by ACI Committee 440 in this manner.

7.3.2.3 Axial Load Capacity Enhancement: FRP composite material may be bonded to external surfaces of concrete or masonry members to enhance axial load capacity. Circular sections, and rectangular sections where the ratio of longer to shorter section side dimension is not greater than 1.5, may have axial compression capacity enhanced by the confining effect of FRP composite material placed with fibers running essentially perpendicular to the members' axis~~ θ ≥ 75 (transverse fiber).~~

The stress-strain model by Lam and Teng (2003a,b) for FRP-confined concrete is illustrated in Figure 6 and computed using the following expressions:

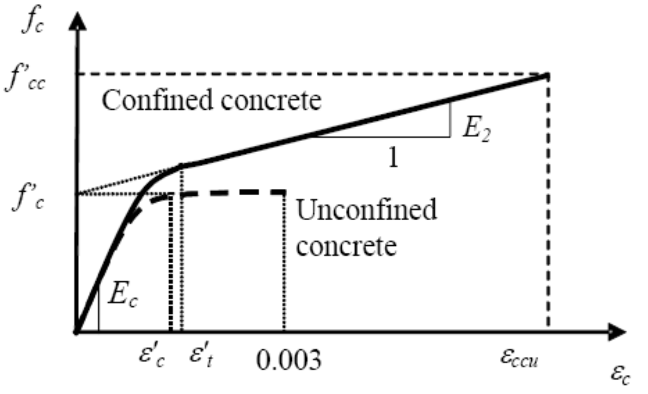


Figure 6 - Stress-Strain Diagram for FRP-Confined Concrete by Lam and Teng (ACI 440.2R-08)

$$f_c = \begin{cases} E_c\varepsilon_c - \dfrac{(E_c - E_2)^2}{4f'_c}(\varepsilon_c)^2 & 0 \le \varepsilon_c \le \varepsilon'_t \\ f'_c + E_2\varepsilon_c & \varepsilon'_t \le \varepsilon_c \le \varepsilon_{ccu} \end{cases} \qquad (1)$$

$$\varepsilon'_t = \frac{2f'_c}{E_c - E_2} \qquad (2)$$

$$E_2 = \frac{f'_{cc} - f'_c}{\varepsilon_{ccu}} \qquad (3)$$

The maximum confined concrete compressive strength, f'_{cc}, and the maximum confinement pressure, f_l, are calculated using Eq. (4) and Eq. (5), respectively with the inclusion of an additional reduction factor, $\psi_f = 0.95$.

$$f'_{cc} = f'_c + \psi_f 3.3\kappa_a f_l \qquad (4)$$

$$f_l = \begin{cases} \dfrac{2nt_f E_f \varepsilon_{fe}}{D} & \text{Circular Cross-section} \\ \dfrac{2nt_f E_f \varepsilon_{fe}}{\sqrt{b^2 + h^2}} & \text{Non-circular Cross-section} \end{cases} \qquad (5)$$

In Eq. (4), f'_c is the unconfined cylinder compressive strength of concrete, and the efficiency factor, κ_a, accounts for the geometry of the section, circular and noncircular, as defined in Sections 7.3.2.3.1 and 7.3.2.3.2. In Eq. (5), the effective strain level in the FRP at failure, ε_{fe}, is given by:

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} \qquad (6)$$

The FRP strain efficiency factor, κ_ε, accounts for the premature failure of the FRP system, (Pessiki et al. 2001) possibly due to the multi-axial state of stress to which it is subjected as opposed to the pure axial tension used for material tensile characterization. This behavior may also be related to stress concentration regions caused by cracking of the concrete as it dilates. A strain efficiency factor κ_ε of 0.55 and a minimum confinement ratio f_l/f'_c of 0.08 should be used (ACI 440.2R-08).

The maximum compressive strain in the FRP-confined concrete, ε_{ccu}, can be found using Eq. (7). This strain should be limited to 0.01 in/in to prevent excessive cracking and the resulting loss of concrete integrity. When this limit is applicable, the corresponding maximum value of f'_{cc} should be recalculated from the stress-strain curve (Concrete Society TR 55, 2004).

$$\varepsilon_{ccu} = \varepsilon'_c \left(1.5 + 12\kappa_b \frac{f_l}{f'_c} \left(\frac{\varepsilon_{fe}}{\varepsilon'_c} \right)^{0.45} \right) \leq 0.01 \qquad (7)$$

In Eq. (7), the efficiency factor, κ_b, accounts for the geometry of the section in the calculation of the ultimate axial strain, as defined in Sections 7.3.2.3.1 and 7.3.2.3.2. Strength enhancement for compression members with f'_c of 10,000 psi or higher has not been experimentally verified.

7.3.2.3.1 Circular Sections: FRP jackets are most effective at confining circular members with circular cross sections (ACI 440.2R-08). For this type of section, the shape factors κ_a and κ_b in Eqs. (4) and (7), respectively, can be taken as 1.0.

7.3.2.3.1 Rectangular Sections: Testing has shown that confining square and rectangular members with FRP jackets can provide marginal increases in the maximum axial compressive strength, f'_{cc}, of the member (ACI 440.2R-08). The provisions in this guide are not recommended for members featuring side aspect ratios, h/b, greater than 2.0, or face dimensions, b or h, exceeding 36 in., unless testing demonstrates their effectiveness.

For noncircular cross sections, f_l in Eq. (5) corresponds to the maximum confining pressure of an equivalent circular cross section with diameter, D, equal to the diagonal of the rectangular cross section.

The shape factors κ_a in Eq. (4) and κ_b in Eq. (7) depend mainly on two parameters: the cross-sectional area of effectively confined concrete, A_e, and the side-aspect-ratio, h/b, as shown in Eq. (8), (9), and Figure 7.

$$\kappa_a = \frac{A_e}{A_c} \left(\frac{b}{h} \right)^2 \qquad (8)$$

$$\kappa_b = \frac{A_e}{A_c} \left(\frac{h}{b} \right)^{0.5} \qquad (9)$$

Where,

$$\frac{A_e}{A_c} = \frac{1 - \left(\left(b/h\right)\left(h-2r\right)^2 + \left(h/b\right)\left(b-2r\right)^2\right) / \left(3A_g\right) - \rho_g}{1-\rho_g}$$

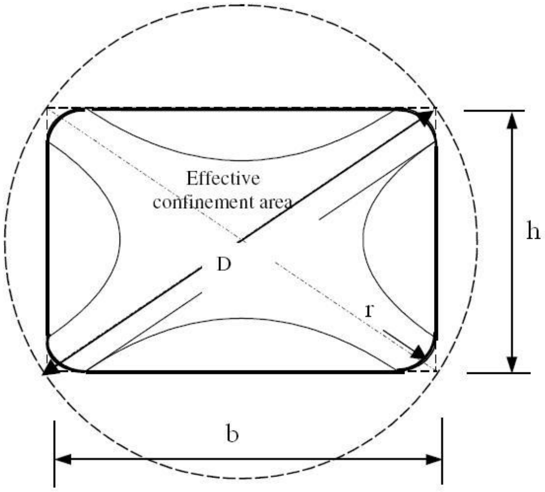


Figure 7 - Equivalent Circular Cross-Section (ACI 440.2R-08)

Additionally, the following notations should be added to the document:

f_c = axial stress of confined concrete, psi

$f_{s,s}$ = stress level in non-prestressed steel reinforcement at service loads, psi

f_y = specified yield strength f non-prestressed steel reinforcement, psi

E_c = elastic modulus of unconfined concrete, psi

E_2 = slope of the second linear portion, psi

f'_{cc} = maximum FRP-confined concrete compressive strength, psi

f_l = FRP confining pressure, psi

r = radius of the edges of non-circular cross section confined with FRP, in.

κ_a = geometry efficiency factor;

κ_b = geometry efficiency factor;

$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu}$ = effective strain of the FRP at ultimate, in/in

ε_{fd} = debonding strain of externally bonded FRP reinforcement, in/in

ε'_c = axial concrete compressive strain corresponding to f'_c = 0.002 in/in

ε_c = axial strain of confined concrete, in/in

ε_{cu} = ultimate axial strain of unconfined concrete, in/in

ε_{ccu} = ultimate axial strain of confined concrete, in/in

ε'_t = transition strain, in/in

κ_ε = strain efficiency factor = 0.55;

ρ_g = ratio of area of longitudinal steel reinforcement to the cross-sectional area of a compression member

ψ_f = Additional reduction factor 0.95

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