tance of an FRP-strengthened concrete element,it is important to recognize that the strength of a reinforced concrete element is reduced during fire exposure due to heating of both the reinforcing steel and the concrete.Performance in fire of the existing concrete member can be enhanced by installing an insulation system,which will provide thermal protection to existing concrete and internal reinforcing steel, thus improving the overall fire rating,although the FRP system contribution may be reduced (Bisby et al.2005a; Williams et al.2006;Palmieri et al.2011;Firmo et al.2012; ACI PRC-440.10).

An evaluation should be performed to ensure an FRP-strengthened structure will not collapse in a fire event.The required member resistance of an FRP-strengthened structural element can be determined using the load combination for a fire event specified in ACI CODE-562 as follows:

$$中eR\geq(0.9 \quad or \quad 1.2)SpL+0.5S+0.2Ss \qquad (9.2.1)$$

where R is the nominal resistance of the member at an elevated temperature without FRP;φer=1.0;and SpL,Su, and Ssz are the specified dead,live,and snow loads,respectively,calculated for the strengthened structure.The dead load factor of 0.9 is applied when the dead load effect mitigates the total load effect;otherwise,the dead load factor of 1.2 is used.For cases where the design live load has a high likelihood of being present for a sustained period of time, a live load factor of 1.0 should be used in place of 0.5 in Eq.(9.2.1).

If the FRP system is meant to allow greater load-carrying capacity,such as an increase in live load,the load effects should be computed using these greater loads.If the FRP system is meant to address a loss in strength,such as deterioration,the resistance should reflect this loss.

The nominal resistance of the member at an elevated temperature R may be determined using the procedure outlined in ACI216.1 or through testing.The nominal resistance R should be calculated based on the reduced material properties of the existing member(ACI PRC-440.10).The resistance should be computed for the time required by the member's fire-resistance rating—for example,a 2-hour fire rating—and should not account for the contribution of the FRP system unless the continued effectiveness of the adhesive resin during fire exposure can be proven through testing. More research is needed to accurately identify temperatures at which effectiveness is lost for different types of FRP.Until better information on the properties of FRP at high temperature is available,the critical temperature at which the FRP is considered ineffective during a fire event can be taken as the lowest Tg of the components of the system comprising the load path.

9.2.2 Overall structural strength—While FRPsystems are effective in strengthening members for flexure and shear and providing additional confinement,other modes of failure, such as punching shear and bearing capacity of footings, may be only marginally affected by FRP systems(Sharaf et al.2006).All members of a structure should be capable of withstanding the anticipated increase in loads associated with the strengthened members.

Additionally,analysis should be performed on the member strengthened by the FRP system to check that,under overload conditions,the strengthened member will fail in a flexural mode rather than in a shear mode.

9.2.3 Seismic applications—Requirements for seismic strengthening using FRP are addressed in **Chapter 13.**

9.3—Selection ofFRP systems

9.3.1 Environmental considerations—Environmental conditions uniquely affect resins and fibers of various FRP systems.The mechanical properties(for example,tensile strength,ultimate tensile strain,and elastic modulus)of some FRP systems degrade under exposure to certain environments such as alkalinity,salt water,chemicals,ultraviolet light,high temperatures,high humidity,and freezing-and-thawing cycles.The material properties used in design should account for this degradation in accordance with 9.4.

The licensed design professional should select an FRP system based on the known behavior of that system in the anticipated service conditions.Some important environmental considerations that relate to the nature of specific systems are given as follows.Specific information can be obtained from the FRP system manufacturer.

(a)Alkalinity/acidity—The performance ofan FRPsystem over time in an alkaline or acidic environment depends on the matrix material and the reinforcing fiber.Dry, unsaturated bare,or unprotected carbon fiber is resistant to both alkaline and acidic environments whereas bare glass fiber can degrade over time in these environments.A properly selected and applied resin matrix, however,should isolate and protect the fiber from an alkaline/acidic environment and resist deterioration.

(b)Thermal expansion—FRP systems may have thermal expansion properties that are different from those of concrete.In addition,the thermal expansion properties of the fiber and polymer constituents of an FRPsystem can vary.Carbon fibers have a coefficient of thermal expansion near zero whereas glass fibers have a coefficient of thermal expansion similar to concrete.The polymers used in FRP strengthening systems typically have coefficients of thermal expansion approximately five times that of concrete.Calculation of thermally induced strain differentials are complicated by variations in fiber orientation,fiber volume fraction,and thickness of adhesive layers.Experience indicates, however,that thermal expansion differences do not affect bond for small ranges of temperature change, such as±50°F(±28°C)(Motavalli et al.1997;Soudki and Green 1997;Green et al.1998).

(c)Electrical conductivity—Glass FRP(GFRP),basalt FRP(BFRP),and aramid FRP(AFRP)are effective electrical insulators,whereas carbon FRP(CFRP)is conductive.To avoid potential galvanic corrosion of steel elements,carbon-based FRP systems should not be placed in direct contact with steel.

9.3.2 Loading considerations—Loading conditions uniquely affect different fibers of FRP systems.The licensed design professional should select an FRP system based on the known behavior of that system in the anticipated service conditions.Some important loading considerations that relate to the nature of the specific systems are given in the following.Specific information should be obtained from material manufacturers.

(a)Impact tolerance—AFRP,GFRP,and BFRP systems demonstrate better tolerance to impact than CFRP systems.

(b)Creep rupture and fatigue—CFRP systems are highly resistive to creep rupture under sustained loading and fatigue failure under cyclic loading.GFRP and BFRP systems are more sensitive to both loading conditions.

9.3.3 Durability considerations—Durability of FRP systems is reported in available literature (Dolan et al. 2008;Karbhari 2007).The licensed design professional should select an FRP system that has undergone durability testing consistent with the application environment.Durability testing may include hot-wet cycling,alkaline immersion,freezing-and-thawing cycling,ultraviolet exposure, dry heat,and salt water(Cromwellet al.2011). ACI 440.9R provides guidance for assessing durability performance of external FRP reinforcement using accelerated conditioning protocols in combination with standard test methods for mechanical properties.

Any FRP system that completely encases or covers a concrete section should be investigated for the effects of a variety of environmental conditions including those of freezing and thawing,steel corrosion,alkali and silica aggregate reactions,water entrapment,vapor pressures,and moisture vapor transmission (Masoud and Soudki 2006; Soudki and Green 1997;Porter et al.1997;Christensen et al. 1996;Toutanji 1999).Many FRPsystems create a moisture-impermeable layer on the surface of the concrete.In areas where moisture vapor transmission is expected,adequate means should be provided to allow moisture to escape from the concrete structure.

9.3.4 Protective-coating selection—A coating or insulation system can be applied to the installed FRP system to protect it from exposure to certain environmental conditions (Bisby et al.2005a;Williams et al.2006).The thickness and type of coating should be selected based on the requirements of the composite repair;resistance to environmental effects such as moisture,salt water,temperature extremes,fire, impact,and ultraviolet exposure;resistance to site-specific effects;and resistance to vandalism.Coatings are relied upon to retard the degradation of the mechanical properties of the FRP systems.The coatings should be periodically inspected and maintained to ensure continued effectiveness.

External coatings or thickened coats of resin over fibers can protect them from damage due to impact or abrasion. In high-impact or traffic areas,additional levels of protection may be necessary.Portland cement plaster and polymer coatings are commonly used for protection where minor impact or abrasion is anticipated.

9.4—Design material properties

Unless otherwise stated,the material properties reported by manufacturers,such as the ultimate tensile strength,typically do not consider long-term exposure to environmental conditions and should be considered as initial properties. Because long-term exposure to various types of environments can reduce the tensile properties and can affect creep-rupture and fatigue endurance of FRPlaminates,the material properties used in design equations should be reduced based on the environmental exposure condition.

Equations(9.4a)through(9.4c)give the tensile properties that should be used in all design equations.The design ultimate tensile strength should be determined using the environmental reduction factor given in Table 9.4 for the appropriate fiber type and exposure condition:

$$fu=Cefn \tag{9.4a}$$

Similarly,the design rupture strain should be reduced for environmental exposure conditions:

$$Cju=CEGnu \tag{9.4b}$$

The expression for the modulus of elasticity,given in Eq.(9.4c),recognizes that the modulus is typically unaffected by environmental conditions.The modulus given in this equation will be the same as the initial value reported by the manufacturer and can be represented as:

$$Ey=fn/epu=fu^{\wedge}/epu \tag{9.4c}$$

The constituent materials,fibers,and resins of an FRP system affect its durability and resistance to environmental exposure.The environmental reduction factors given in Table 9.4 are conservative estimates based on the relative durability of each fiber type.

As Table 9.4 illustrates,if the FRP system is located in a relatively benign environment,such as indoors,the reduction factor is closer to unity.If the FRP system is located

Table 9.4—Environmental reduction factor for various FRP systems and exposure conditions

Exposure conditions	Fiber type	Environmental reduction factor CE
Interior exposure	Carbon	0.95
	Glass	0.75
	Basalt	0.75
	Aramid	0.85
Exterior exposure(bridges,piers,and unenclosed parking garages)	Carbon	0.85
	Glass	0.65
	Basalt	0.65
	Aramid	0.75
Aggressive environment(chemical plants and wastewater treatment plants)	Carbon	0.85
	Glass	0.50
	Basalt	0.50
	Aramid	0.70

in an aggressive environment where prolonged exposure to high humidity,freezing-and-thawing cycles,salt water,or alkalinity is expected,a lower reduction factor should be used.The reduction factor can be modified to reflect the use ofa protective coating if the coating has been shown through testing to lessen the effects of environmental exposure and the coating is maintained for the life of the FRP system.

CHAPTER 10—FLEXURAL STRENGTHENING

Bonding fiber-reinforced polymer(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.Increases in overall flexural strength from 10 to 160%have been documented(Meier and Kaiser 1991;Ritchie et al.1991;Sharif et al.1994) . When taking into account the strengthening limits of 9.2 and ductility and serviceability limits,however,strength increases of up to 40%are more reasonable.

This chapter does not apply to FRP systems used to enhance the flexural strength of members in the expected plastic hinge regions of ductile moment frames resisting seismic loads;these are addressed in Chapter 13.

10.1—Nominal strength

The strength design approach requires that the design flexural strength of a member exceed its required factored moment,as indicated by Eq.(10.1).The design flexural strength φM,refers to the nominal strength of the member multiplied by a strength reduction factor,and the factored moment M,refers to the moment calculated from factored loads(for example,apLML+αMu+..)

$$\varphi M \geq M \qquad (10.1)$$

This guide recommends that the factored moment Mu of a section be calculated by use of load factors as required by ACI 318.An additional strength reduction factor for FRP, ψ,should be applied to the flexural contribution of the FRP reinforcement,Mą,as described in 10.2.10.The additional strength reduction factor,ψ;is used to improve the reliability of strength prediction and accounts for the different failure modes observed for FRP-strengthened members(delamination ofFRP reinforcement).

The nominal flexural strength of FRP-strengthened concrete members with mild steel reinforcement and with bonded prestressing steel can be determined based on strain compatibility,internal force equilibrium,and the controlling mode of failure.For members with unbonded prestressed steel,strain compatibility does not apply and the stress in the unbonded tendons at failure depends on the overall deformation of the member and is assumed to be approximately the same at all sections.

10.1.1 Failure modes—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(GangaRao and Vijay 1998) :

(a)Crushing of the concrete in compression before yielding of the reinforcing steel

(b)Yielding of the steel in tension followed by rupture of the FRP laminate

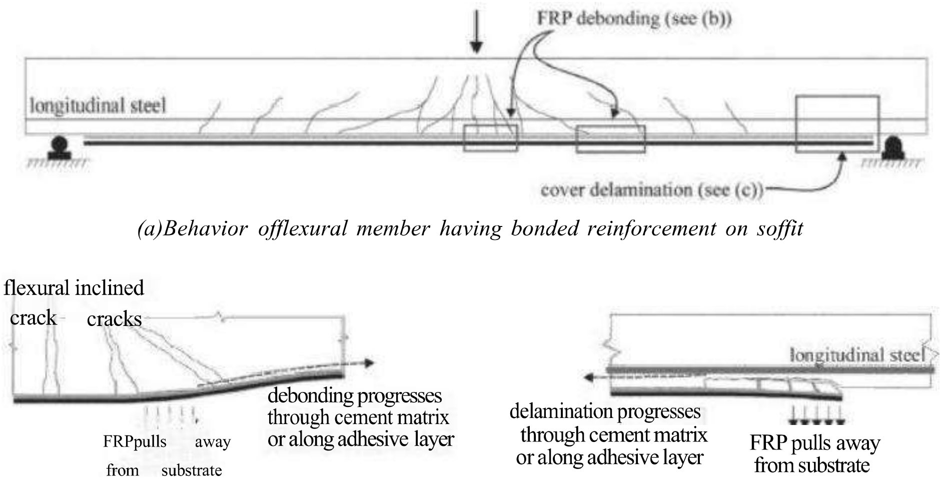
(c)Yielding of the steel in tension followed by concrete crushing

(d)Shear/tension delamination of the concrete cover (cover delamination)

(e)Debonding of the FRP from the concrete substrate (FRP debonding)

Concrete crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain(ce=8cu=0.003).Rupture of the externally bonded FRP is assumed to occur if the strain in the FRP reaches its design rupture strain(=8元) before the concrete reaches its maximum usable strain.

Cover delamination or FRP debonding can occur if the force in the FRP cannot be sustained by the substrate (Fig.10.1.1).Such behavior is generally referred to as debonding,regardless of where the failure plane propagates



(a)Behavior offlexural member having bonded reinforcement on soffit

(b)Debonding initiated by flexural and/or shear cracks

(c)Cover delamination initiatedat curtailment of bonded FRP reinforcement

Fig.10.1.1—Debonding and delamination of externally bonded FRP systems.

within theFRP-adhesive-substrate region.Guidance to avoid the cover delamination failure mode is given in Chapter 14.

Away from the section where the externally bonded FRP terminates,a failure controlled by FRP debonding may govern (Fig.10.1.1(b).To prevent such an intermediate crack-induced debonding failure mode,the effective strain in FRP reinforcement should be limited to the strain at which debonding may occur,ca,as defined in Eq.(10.1.1).

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu} \quad \text{(in.-lb)}$$
$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu} \quad \text{(SI)} \qquad (10.1.1)$$

Equation(10.1.1)takes a modified form of the debonding strain equation proposed by Teng et al.(2003,2004)that was based on committee evaluation of a significant database for flexural beam tests exhibiting FRP debonding failure.The proposed equation was calibrated using average measured values ofFRP strains at debonding for flexural tests experiencing intermediate crack-induced debonding to determine the best-fit coefficient of0.083(0.41 in SI).Reliability of the FRP contribution to flexural strength is addressed by incorporating an additional strength reduction factor for FRP,ψ; in addition to the strength reduction factor φ per ACI 318 for structural concrete.Anchorage systems such as U-wraps and fiber anchors(examples are shown schematically in Fig.14.1.1)have been proven to control debonding failure of the longitudinal FRP.Providing such anchorage can improve the flexural strength and deformability of the externally bonded FRP-strengthened member(Kalfat et al.2018; Grelle and Sneed 2013;Lee and Lopez 2020;Smith et al. 2013).Providing anchorage does not necessarily prevent local FRP debonding nor the associated brittle failure mode. However,FRP anchorage may mitigate sudden complete debonding of the FRP,allowing the FRP to continue to resist force.Experimental studies have shown that these anchorage systems can increase the effective strain in the flexural FRP to values up to tensile rupture(Lee et al.2010; Orton et al.2008).

For near-surface-mounted(NSM)FRP applications,the value of ca may vary from 0.68;u to 0.9c,depending on many factors such as member dimensions,steel and FRP reinforcement ratios,and surface roughness of the FRP bar. Based on analysis of a database of existing studies(Bianco et al.2014),the committee recommends the use of ea= 0.7ep.for NSM bars and plates.To achieve the debonding design strain of NSM FRP bars and plates,ejad,the bonded length should be greater than the development length given in Chapter 14.

10.2—Reinforced concrete members

This section presents guidance on the calculation of the flexural strengthening effect of adding longitudinal FRP reinforcement to the tension face of a reinforced concrete member.A specific illustration of the concepts in this section applied to strengthening of existing rectangular sections reinforced in the tension zone with nonprestressed steel is given.The general concepts outlined herein can,however,be extendedto nonrectangular shapes(T-sections andI-sections) and to members with steel compression reinforcement.

10.2.1 Assumptions—The following assumptions are made in calculating the flexural resistance of a section strengthened with an externally applied FRP system:

(a)Design calculations are based on the dimensions, internal reinforcing steel arrangement,and material properties of the existing member being strengthened.

(b)The strains in the steel reinforcement and concrete are directly proportional to their distance from the neutral axis.That is,a plane section before loading remains plane after loading.

(c)There is no relative slip between external FRP reinforcement and the concrete.

(d)The shear deformation within the adhesive layer is neglected because the adhesive layer is very thin with only slight variations in its thickness.

(e)The maximum usable compressive strain in the concrete is 0.003.

(1)The tensile strength of concrete is neglected.

(g)The FRPreinforcement has a linear elastic stress-strain relationship to failure.

While some of these assumptions are necessary for the sake of computational ease,the assumptions do not accurately reflect the true fundamental behavior ofFRP flexural reinforcement.For example,there will be shear deformation in the adhesive layer,causing relative slip between the FRP and the substrate.The inaccuracy of the assumptions will not,however,significantly affect the computed flexural strength of an FRP-strengthened member.An additional strength reduction factor (presented in 10.2.10)will conservatively compensate for these discrepancies.

10.2.2 Shear strength—When FRP reinforcement is being usedtoincrease the flexural strengthofamember,themember should be capable of resisting the shear forces associated with the increased flexural strength.The potential for shear failure of the section should be considered by comparing the design shear strength of the section to the required shear strength.If additional shear strength is required,FRP laminates oriented transverse to the beam longitudinal axis can be used to resist shear forces,as described in Chapter 11.

10.2.3 Existing substrate strain—Unless all loads on a member,including self-weight and any prestressing forces, are removed before installation of FRP reinforcement,the substrate to which the FRP is applied will be strained.These strains should be considered to be initial strains and should be excluded from the strain in the FRP(Arduini and Nanni 1997;Nanni and Gold 1998).The initial strain on the bonded substrate,Ebi,can be determined from an elastic analysis of the existing member,considering all loads that will be on the member during the installation of the FRP system.The elastic analysis of the existing member should be based on cracked section properties.

10.2.4 Flexural strengthening of concave soffits—The presence of curvature in the soffit of a concrete member may lead to the development of tensile stresses normal to the adhesive and surface to which the FRP is bonded.Such tensile stresses result when the FRP tends to straighten under load and can promote the initiation ofFRP debonding or interlaminar failures that reduce the effectiveness of the FRP flexural strengthening(Aiello et al.2001;Eshwar et al. 2003).If the extent of the curved portion of the soffit exceeds a length of 40 in.(1.0m)with a rise of 0.2 in.(5mm),the surface should be made flat before strengthening.Alternately,anchorage systems such as U-wraps,mechanical fasteners,fiber anchors,or NSM anchors should be installed to control delamination(Eshwar et al.2005).

10.2.5 Strain in FRP reinforcement—It is important to determine the strain in the FRP reinforcement at the ultimate limit state.Because FRP systems are linear elastic until failure,the strain in the FRPwill dictate the stress developed in the FRP.The maximum strain that can be achieved in the FRP reinforcement will be governed by either the strain developed in the FRP at the point at which concrete crushes, the point at which the FRP ruptures,or the point at which the FRP debonds from the substrate.The effective strain in the FRP reinforcement at the ultimate limit state can be found from Eq.(10.2.5)

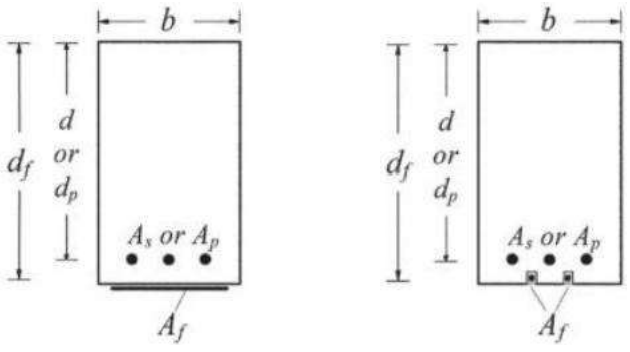
$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \le \varepsilon_{fd} \qquad (10.2.5)$$

where cb is the initial substrate strain as described in 10.2.3, and dy is the effective depth ofFRP reinforcement,as indicated in Fig.10.2.5.

10.2.6 Stress in the FRP reinforcement—The effective stress in the FRP reinforcement is the maximum level of stress that can be developed in the FRP reinforcement before fexural failure of the section.This effective stress can be found from the strain in the FRP,assuming elastic behavior

Jfe=EEe (10.2.6)

10.2.7 Strength reductionfactor—The use of externally bonded FRP reinforcement for flexural strengthening will reduce the ductility of the original member.In some cases, the loss of ductility is negligible.Sections that experience a significant loss in ductility,however,should be addressed. To maintain a sufficient degree of ductility,the strain in the steel at the ultimate limit state should be checked.For



(a)FRPLaminates *(b)NSM bars*

Fig.10.2.5—Effective depth ofFRP systems.

reinforced concrete members with nonprestressed steel reinforcement,adequate ductility is achieved if the strain in the steel at the point of concrete crushing or failure of the FRP,including delamination or debonding,is at least 0.005, according to the definition of a tension-controlled section,as given in ACI 318.

The approach taken by this guide follows the philosophy ofACI 318.A strength reduction factor given by Eq.(10.2.7) should be used,where ε,is the net tensile strain in the extreme tension steel at nominal strength,as defined in ACI 318

$$\phi = \begin{cases} 0.90 \text{ for } \varepsilon_t \ge 0.005 \\ 0.65 + \dfrac{0.25(\varepsilon_t - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} \text{ for } \varepsilon_{sy} < \varepsilon_t < 0.005 \\ 0.65 \text{ for } \varepsilon_t \le \varepsilon_{sy} \end{cases} \qquad (10.2.7)$$

This equation sets the reduction factor at 0.90 for ductile sections and 0.65 for brittle sections where the steel does not yield and provides a linear transition for the reduction factor between these conditions.The condition may arise in which the governing failure mode is FRP debonding;this occurs if the right term of the inequality in Eq.(10.2.5)controls.In these cases,although e,may be greater than 0.005,cemay be less than 0.003 at section capacity.This leads to section curvature less than that which defines tension-controlled behavior;that is,&,+Ecu<0.008.In these cases,anchorage of the FRP can be used to increase Eyu,control a debonding failure mode,or both.The use of Eq.(10.2.7)is limited to steel having a yield strength f,less than 80 ksi(550 MPa) (ACI 318).

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

To avoid inelastic deformations of reinforced concrete members with nonprestressed steel reinforcement strengthened with external FRP reinforcement,the existing internal steel reinforcement should be prevented from yielding under service load levels,especially for members subjected to cyclic loads (El-Tawil et al.2001).The stress in the steel reinforcement under service load should be limited to 80% of the yield strength,as shown in Eq.(10.2.8a).In addition, the compressive stress in concrete under service load should be limited to 60%of the compressive strength,as shown in Eq.(10.2.8b)

Jfs≤0.80f, (10.2.8a)

fcs≤0.60fe (10.2.8b)

10.2.9 Creep rupture andfatigue stress limits—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 in the FRP reinforcement under these stress conditions should be checked.Because this stress will be within the elastic response range of the member,the stresses can be computed by elastic analysis using cracked section properties as appropriate.

In Section 4.4,the creep rupture phenomenon and fatigue characteristics ofFRP systems are described and the resistance to these effects by various types of fibers is examined. As stated in 4.4.1,research has indicated that glass,basalt, aramid,and carbon fibers can sustain approximately 0.3,

$f_s \leq$ sustained plus cyclic stress limit (10.2.9)

10.2.10 Ultimate strength ofsingly reinforced rectangular section—To illustrate the concepts presented in this chapter, this section describes the application of these concepts to a nonprestressed singly-reinforced rectangular section. Figure 10.2.10 illustrates the internal strain and stress distribution for a rectangular section under flexure at the ultimate limit state.

The calculation procedure used to arrive at the ultimate strength should satisfy strain compatibility and force equilibrium,and should consider the governing mode of failure. Several calculation procedures can be derived to satisfy

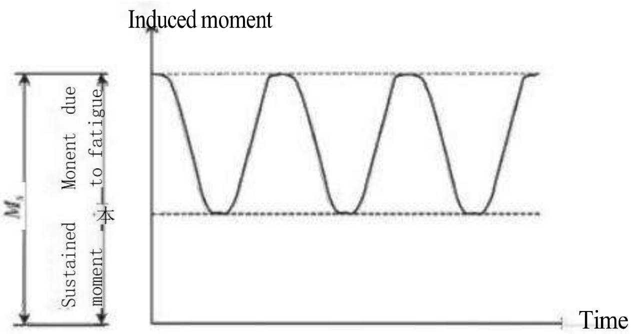


Fig.10.2.9—Illustration of the level of applied moment to be used to check the stress limits in the FRP reinforcement.

Table 10.2.9—Sustained plus cyclic service load stress limits in FRP reinforcement

Stress type	Fiber type			
	GFRP	BFRP	AFRP	CFRP
Sustainedplus cyclic stress limit	0.20/m	0.20/m	0.30f	0.55.

these conditions.The calculation procedure described herein illustrates an iterative method that involves selecting an assumed depth to the neutral axis,c,calculating the strain in each material using strain compatibility;calculating the associated stress in each material;and checking internal force equilibrium.If the internal force resultants do not equilibrate,the depth to the neutral axis is revised and the procedure repeated.

For any assumed depth to the neutral axis,c,the strain in the FRP reinforcement can be computed from Eq.(10.2.5). This equation considers the governing mode of failure for the assumed neutral axis depth.If the left term of the inequality controls,concrete crushing controls flexural failure of the section.If the right term of the inequality controls,FRP failure (rupture or debonding)controls flexural failure of the section.

The effective stress in the FRP reinforcement can be found from the strain in the FRP,assuming elastic behavior using Eq.(10.2.6).Based on the strain in the FRP reinforcement, the strain in the nonprestressed steel reinforcement can be found from Eq.(10.2.10a)using strain compatibility:

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d-c}{d_f-c}\right) \qquad (10.2.10a)$$

The stress in the steel is determined from the strain in the steel using its assumed elastic-perfectly plastic stress-strain curve:

$$f_s = E_s \varepsilon_s \leq f_y \qquad (10.2.10b)$$

With the stress in the FRP and steel reinforcement determined for the assumed neutral axis depth,internal force equilibrium may be checked using Eq.(10.2.10c):

$$\alpha_1 f_c' \beta_1 bc = A_s f_s + A_f f_{fe} \qquad (10.2.10c)$$

The terms $\alpha_1$and $\beta_1$in Eq.(10.2.10c)are parameters defining a rectangular stress block in the concrete equivalent to the nonlinear distribution of stress.If concrete crushing is the controlling mode of failure(before or after steel yielding),$\alpha_1$and $\beta_1$can be taken as the values associated with the Whitney stress block(ACI 318);that is,α_1=0.85 and β_1=0.85 for f_c'between 2500 and 4000 psi(17 and 27 MPa),and $\beta_1$is reduced linearly at a rate of 0.05 for each 1000 psi(7 MPa)of concrete strength exceeding 4000 psi (27 MPa).Note that $\beta_1$need not be taken less than 0.65.If FRP rupture or FRP debonding occur,the Whitney stress block will give reasonably accurate results.A nonlinear stress distribution in the concrete or a more accurate stress block appropriate for the strain level reached in the concrete at the ultimate-limit state may also be used.

The depth to the neutral axis,c,is found by simultaneously satisfying Eq.(10.2.5),(10.2.6),(10.2.10a),(10.2.10b),and (10.2.10c),thus establishing internal force equilibrium and strain compatibility.To solve for the depth of the neutral axis,c,an iterative solution procedure can be used.An initial value for c is assumed and the strains and stresses

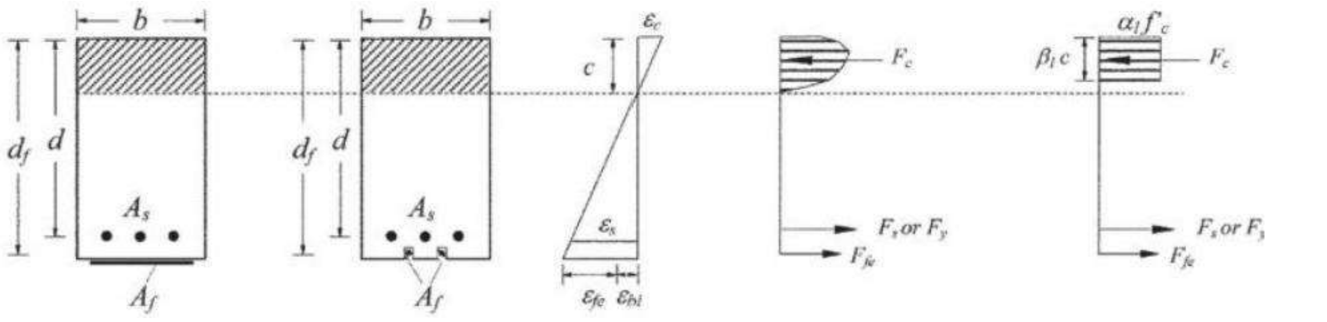


Fig.10.2.10—Internal strain and stress distributionfor a rectangular section underflexure at ultimate limit state.

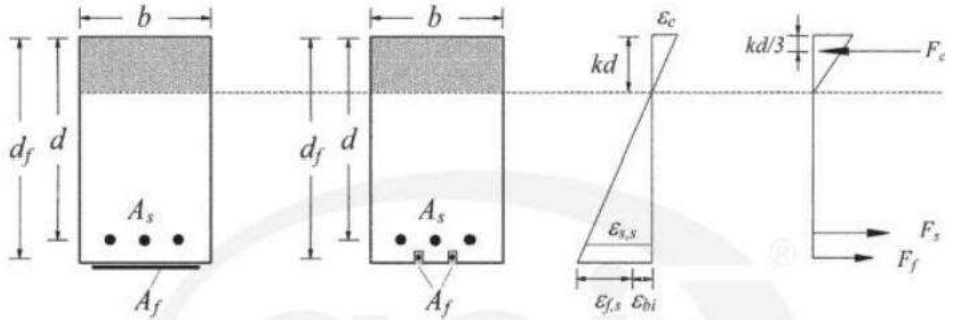


Fig.10.2.10.1—Elastic strain and stress distribution.

are calculated using Eq.(10.2.5),(10.2.6),(10.2.10a),and (10.2.10b).A revised value for the depth of neutral axis, c,is then calculated from Eq.(10.2.10c).The calculated and assumed values of c are then compared.If they agree within an acceptable tolerance,then the correct value of cis reached.If the calculated and assumed values do not agree, another value for c is selected,and the process is repeated until convergence is attained.

The nominal flexural strength of the section with FRP external reinforcement is computed from Eq.(10.2.10d). An additional reduction factor for FRP,ψ,is applied to the flexural-strength contribution of the FRP reinforcement.The recommended value of y is 0.85.This reduction factor for the strength contribution ofFRP reinforcement is based on the reliability analysis discussed in 9.1,which was based on the experimentally calibrated statistical properties of the flexural strength(Okeil et al.2007).

$$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right) \quad (10.2.10d)$$

10.2.10.1 Stress in steel under service loads—The stress in the steel reinforcement can be calculated based on a cracked-section analysis of the FRP-strengthened reinforced concrete section,as indicated by Eq.(10.2.10.1):

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f\left(d_f - \frac{kd}{3}\right)\right](d-kd)E_s}{\left[\begin{array}{l} A_s E_s\left(d - \frac{kd}{3}\right)(d-kd) \\ +A_f E_f\left(d_f - \frac{kd}{3}\right)(d_f - kd)\end{array}\right]} \quad (10.2.10.1)$$

The distribution of strain and stress in the reinforced concrete section is shown in Fig.10.2.10.1.Similar to conventional reinforced concrete,the depth to the neutral axis at service,kd,can be computed by taking the first moment of the areas of the transformed section.The transformed area of the FRP may be obtained by multiplying the area of FRP by the modular ratio of FRP to concrete. Although this method neglects the difference in the initial strain of the FRP,the initial strain does not greatly influence the depth to the neutral axis in the elastic response range of the member.

The stress in the steel under service loads computed from Eq.(10.2.10.1)should be compared against the limits described in 10.2.8.The value of M,from Eq.(10.2.10.1)is equal to the moment due to all sustained loads (dead loads and the sustained portion of the live load)plus the maximum moment induced in a fatigue loading cycle,as shown in Fig.10.2.9.

10.2.10.2 Stress in FRPunder service loads—The stress in the FRPreinforcement can be computed using Eq.(10.2.10.2) with fss from Eq.(10.2.10.1).Equation(10.2.10.2)gives the stress in the FRP reinforcement under an applied moment within the elastic response range of the member:

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi}E_f \quad (10.2.10.2)$$

The stress in the FRP under service loads computed from Eq.(10.2.10.2)should be compared against the limits described in 10.2.9.

10.3—Prestressed concrete members

This section presents guidance on the effect of adding longitudinal FRP reinforcement to the tension face of a rectangular prestressed concrete member.The general concepts outlined herein can be extended to nonrectangular shapes (T-sections and I-sections)and to members with nonprestressed steel reinforcement.

10.3.1 Members with bondedprestressing steel

10.3.1.1 Assumptions—In addition to the basic assumptions for concrete and FRPbehavior for a reinforced concrete section listed in 10.2.1,the following assumptions are made in calculating the flexural resistance of a prestressed section strengthened with an externally applied FRP system:

(a)Strain compatibility can be used to determine strain in the externally bonded FRP,strain in the nonprestressed steel reinforcement,and the strain or strain change in the prestressing steel.

(b)Aflexural failure mode controlled by prestressing steel rupture should be investigated.

(c)For cases where the prestressing steel is draped or harped,several sections along the span of the member should be evaluated to verify strength requirements.

(d)The initial strain of the concrete substrate,Ebi,should be calculated and excluded from the effective strain in the FRP.The initial strain can be determined from an elastic analysis of the existing member,considering all loads that will be applied to the member at the time of FRP installation.Analysis should be based on the actual condition of the member(cracked or uncracked section)to determine the substrate initial strain.

10.3.1.2 Strain in FRP reinforcement—The maximum strain that can be achieved in the FRP reinforcement will be governed by strain limitations due to either concrete crushing,FRP rupture,FRP debonding,or prestressing steel rupture.The effective design strain for FRP reinforcement at the ultimate-limit state for failure controlled by concrete crushing can be calculated using Eq.(10.2.5)

For failure controlled by prestressing steel rupture, Eq.(10.3.1.2a)can be used.For Grade 270 and 250 ksi (1860 and 1725 MPa)strand,the value of epu to be used in Eq.(10.3.1.2a)is 0.035:

$$\varepsilon_{fe} = (\varepsilon_{pu} - \varepsilon_{pi})\left(\frac{d_f - c}{d_p - c}\right) - \varepsilon_{bi} \le \varepsilon_{fd} \quad (10.3.1.2a)$$

in which

$$\varepsilon_{pi} = \frac{P_e}{A_p E_{ps}} + \frac{P_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) \quad (10.3.1.2b)$$

*10.3.1.3 Strength reductionfactor—To maintain a suffi*cient degree of ductility,the strain in the prestressing steel at the nominal strength should be checked.Adequate ductility is achieved if the strain in the prestressing steel at the nominal strength is at least 0.013.Where this strain cannot be achieved,the strength reduction factor is decreased to account for a less ductile failure.The strength reduction factor for a member prestressed with standard 270 and 250 ksi(1860 and 1725 MPa)prestressing steel is given by Eq.(10.3.1.3),where seps is the prestressing steel strain at the nominal strength:

$$\phi = \begin{cases} 0.90 & \text{for } \varepsilon_{ps} \ge 0.013 \\ 0.65 + \dfrac{0.25(\varepsilon_{ps} - 0.010)}{0.013 - 0.010} & \text{for } 0.010 < \varepsilon_{ps} < 0.013 \\ 0.65 & \text{for } \varepsilon_{ps} \le 0.010 \end{cases} \quad (10.3.1.3)$$

10.3.1.4 Serviceability—To avoid inelastic deformations of the strengthened member,the prestressing steel should be prevented from yielding under service load levels.The stress in the steel under service load should be limited per Eq.(10.3.1.4a)and(10.3.1.4b).In addition,the compressive stress in the concrete under service load should be limited to 60%of the compressive strength:

fpss≤0.82fpy (10.3.1.4a)

Jpss≤0.74fpu (10.3.1.4b)

When fatigue is a concern,the stress in the prestressing steel due to transient live loads should be limited to 18 ksi (125 MPa)when the radii of prestressing steel curvature exceed 29 ft (9m),or to 10ksi(70 MPa)when the radii of prestressing-steel curvature does not exceed 12ft(3.6m).A linear interpolation should be used for radii between 12 and 29 ft(3.6 and 9m)(AASHTO LRFD).These limits have been verified experimentally for prestressed members with harped and straight strands strengthened with externally bonded FRP(Rosenboom and Rizkalla 2006).

10.3.1.5 Creep rupture andfatigue stress limits—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 in the FRP reinforcement under these stress conditions should not exceed the limits provided in 10.2.9.

10.3.1.6 Nominal strength—The calculation procedure to compute nominal strength should satisfy strain compatibility and force equilibrium,and should consider the governing mode of failure.The calculation procedure described herein uses an iterative method similar to that discussed in 10.2.

For any assumed depth to the neutral axis,c,the effective strain and stress in the FRP reinforcement can be computed from Eq.(10.2.5)and(10.2.6),respectively.This equation considers the governing mode of failure for the assumed neutral axis depth.If the left term of the inequality in Eq.(10.2.5)controls,concrete crushing controls flexural failure of the section.If the right term of the inequality controls,FRP failure (rupture or debonding)controls flexural failure of the section.

The strain in the prestressed steel can be found from Eq.(10.3.1.6a)based on strain compatibility:

$$\varepsilon_{ps} = \varepsilon_{pe} + \frac{P_e}{A_c E_c}\left(1+\frac{e^2}{r^2}\right) + \varepsilon_{pnet} \le 0.035 \qquad (10.3.1.6a)$$

in which εpe is the effective strain in the prestressing steel after losses,and epmer is the net tensile strain in the prestressing steel beyond decompression,at the nominal strength.The value of pme will depend on the mode of failure and can be calculated using Eq.(10.3.1.6b)and(10.3.1.6c):

$$\varepsilon_{pnet} = 0.003\left(\frac{d_p - c}{c}\right) \text{ for concrete crushing failure} \qquad (10.3.1.6b)$$

$$\varepsilon_{pnet} = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d_p - c}{d_f - c}\right) \qquad (10.3.1.6c)$$

for FRP rupture or debonding failure modes

The stress in the prestressing steel is calculated using the material properties of the steel.For a typical seven-wire low-relaxation prestressing strand,the stress-strain curve may be approximated by the following equations(Prestressed/ Precast Concrete Institute 2004).

For Grade 250 ksi(1725 MPa)steel:

$$f_{ps} = \begin{cases} 28,500\varepsilon_{ps} & \text{for } \varepsilon_{ps} \le 0.0076 \\ 250 - \dfrac{0.04}{\varepsilon_{ps} - 0.0064} & \text{for } \varepsilon_{ps} > 0.0076 \end{cases} \quad \text{(in.-lb)}$$

$$f_{ps} = \begin{cases} 196,500\varepsilon_{ps} & \text{for } \varepsilon_{ps} \le 0.0076 \\ 1720 - \dfrac{0.276}{\varepsilon_{ps} - 0.0064} & \text{for } \varepsilon_{ps} > 0.0076 \end{cases} \quad \text{(SI)} \qquad (10.3.1.6d)$$

For Grade 270 ksi(1860 MPa)steel:

$$f_{ps} = \begin{cases} 28,500\varepsilon_{ps} & \text{for } \varepsilon_{ps} \le 0.0086 \\ 270 - \dfrac{0.04}{\varepsilon_{ps} - 0.007} & \text{for } \varepsilon_{ps} > 0.0086 \end{cases} \quad \text{(in.-lb)}$$

$$f_{ps} = \begin{cases} 196,500\varepsilon_{ps} & \text{for } \varepsilon_{ps} \le 0.0086 \\ 1860 - \dfrac{0.276}{\varepsilon_{ps} - 0.007} & \text{for } \varepsilon_{ps} > 0.0086 \end{cases} \quad \text{((SI)}$$

(10.3.1.6e)

With the strain and stress in the FRPand prestressing steel determined for the assumed neutral axis depth,internal force equilibrium may be checked using Eq.(10.3.1.6f):

α₁fe'β₁bc=Apfp+Afe (10.3.1.6f)

For the concrete crushing mode of failure,the equivalent compressive stress block factor a₁can be taken as 0.85,and βi can be estimated as described in 10.2.10.If FRP rupture or FRP debonding failure occurs,the use of equivalent rectangular concrete stress block factors is appropriate.Methods considering a nonlinear stress distribution in the concrete can also be used.

The depth to the neutral axis,c,is found by simultaneously satisfying Eq.(10.2.5),(10.2.6),and(10.3.1.6a)to (10.3.1.6f),thus establishing internal force equilibrium and strain compatibility.To solve for the depth of the neutral axis,c,an iterative solution procedure can be used.An initial value for c is assumed,and the strains and stresses are calculated using Eq.(10.2.5),(10.2.6),and(10.3.1.6a) to(10.3.1.6e).A revised value for the depth of neutral axis, c,is then calculated from Eq.(10.3.1.6f).The calculated and assumed values for c are then compared.If they agree, then the correct value of c is reached.If the calculated and assumed values do not agree,another value for c is selected, and the process is repeated until convergence is attained.

The nominal flexural strength of the section with FRP external reinforcement can be computed using Eq.(10.3.1.6g).The additional reduction factor ψ=0.85 is applied to the flexural-strength contribution of the FRP reinforcement:

$$M_n = A_p f_{ps}\left(d_p - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right) \qquad (10.3.1.6g)$$

10.3.1.7 Stress in prestressing steel under service loads— The stress in the prestressing steel can be calculated based on the actual condition(cracked or uncracked section)of the strengthened reinforced concrete section.The strain in prestressing steel at service,Eps.s,can be calculated as:

$$\varepsilon_{ps,s} = \varepsilon_{pe} + \frac{P_e}{A_c E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet,s} \quad (10.3.1.7a)$$

in which epe is the effective prestressing strain,and Eper.s is the net tensile strain in the prestressing steel beyond decompression at service.The value of spne,s depends on the effective section properties at service,and can be calculated using Eq.(10.3.1.7b)and(10.3.1.7c):

$$\varepsilon_{pnet,s} = \frac{M_{snet}e}{E_c I_g} \text{ for uncracked section at service} \quad (10.3.1.7b)$$

$$\varepsilon_{pnet,s} = \frac{M_{snet}e}{E_c I_{cr}} \text{ for cracked section at service} \quad (10.3.1.7c)$$

where Msner is the net service moment beyond decompression.The stress in the prestressing steel under service loads can then be computed from Eq.(10.3.1.6d)and(10.3.1.6e) and compared against the limits described in 10.3.1.4.

10.3.1.8 Stress in FRP under service loads—Equation(10.3.1.8)gives the stress in the FRP reinforcement under an applied moment within the elastic response range of the member.The calculation procedure for the initial strain Eb at the time of FRP installation will depend on the state of the concrete section at the time of FRP installation and at service condition.Prestressed sections can be uncracked at both installation and service,uncracked at installation and cracked at service,or cracked at both installation and service.The initial strain on the bonded substrate,Ebi,can be determined from an elastic analysis of the existing member, considering all loads that will be on the member during the installation of the FRP system.The elastic analysis of the existing member should be based on cracked or uncracked section properties,depending on existing conditions.In most cases,the initial strain before cracking is relatively small, and may conservatively be ignored:

$$f_{f,s} = \left(\frac{E_f}{E_c}\right)\frac{M_s y_b}{I} - \varepsilon_{bi} E_f \quad (10.3.1.8)$$

Depending on the actual condition at service(cracked or uncracked),the moment of inertia,I,can be taken as the moment of inertia of the uncracked section transformed to concrete,I,or the moment of inertia of the cracked section transformed to concrete,I.The variable ys is the distance from the centroidal axis of the gross section,neglecting reinforcement,to the extreme bottom fiber.The computed stress in the FRP under service loads should not exceed the limits provided in 10.2.9.

10.3.2 Members with unbonded prestressing steel—The design procedure ofFRP strengthening for members having unbonded prestressing (post-tensioning)reinforcement is similar to that for those having bonded prestressing reinforcement.In sections having unbonded prestressed reinforcement,the unbonded steel slips relative to the surrounding concrete resulting in the calculation of steel strain or stress becoming a function of overall member deformation rather than just section curvature.The same equilibrium approach applied to a flexural section with bonded prestressing may be used to determine the nominal strength of the FRP-strengthened member provided an appropriate method is used to calculate the strains or stresses in the unbonded tendons at the ultimate flexural strength (EI Meski and Harajli 2014). Although many expressions have been presented in the technical literature for evaluating the stress fps in prestressing steel at nominal flexural strength,a simplified expression may be adopted for FRP-strengthened simply supported and continuous members as is described herein.

10.3.2.1 Assumptions—In addition to the assumptions listed in Sections 10.2.1 and 10.3.1,the effect of concrete precompression strain on the tendon stress at nominal flexural capacity is assumed to be minor and may be neglected in unbonded members when calculating the flexural resistance of an unbonded prestressed section strengthened with an externally applied FRP system(Harajli 2012).

10.3.2.2 Nominal strength—The calculation procedure to compute nominal strength at a critical section follows the same trial-and-error calculation procedure described in Section 10.3.1.

At a critical section under consideration,and for any assumed depth to the neutral axis,c,the effective strain and stress in the FRP reinforcement can be computed from Eq.(10.2.5)and (10.2.6),respectively.This equation considers the governing mode of failure for the assumed neutral axis depth.If the left term of the inequality in Eq.(10.2.5)controls,concrete crushing controls flexural failure of the section and the value of e in Eq.(10.3.2.2b)is equal to Ecu.If the right term of the inequality controls,FRP failure (rupture or debonding)controls flexural failure of the section and the value of εcin Eq.(10.3.2.2b)is found from Eq(10.3.2.2a):

$$\varepsilon_c = \left(\varepsilon_{fd} + \varepsilon_{bi}\right)\left(\frac{c}{d_f - c}\right) \quad (10.3.2.2a)$$

The strain level in the unbonded prestressed steel at the corresponding critical section can be found from Eq.(10.3.2.2b)(Harajli 2012;El Meski and Harajli 2014):

$$\varepsilon_{ps} = \varepsilon_{pe} + \eta\varepsilon_c\left(\frac{d_p - c}{L_a}\right) \quad (10.3.2.2b)$$

in which cpe is the effective strain in the prestressing steel after losses,and La is the total length of tendon between anchorages.H is a parameter that combines the effects of member continuity and applied load pattern for producing maximum factored moment at the critical section under consideration and may be taken as follows(Harajli 2012; EI Meski and Harajli 2014):

Positive moment:

simply supported spans η=14.0

exterior spans η=19.0

interior spans η=24.5

Negative moment at first interior support:

two-span members η=38.5

three or more spans η=43.5

Negative moment at all other interior supports:η=49.0

Negative moment at support of cantilever spans:η=5.3

In unbonded prestressed members,the stress in the prestressing steel seldom exceeds yield;limiting the corresponding stress to 0.95f,allows the use of a linear stress-strain relationship for the prestressing steel:

$$f_{ps}=E_p\varepsilon_s\leq 0.95f \qquad (10.3.2.2c)$$

With the strain and stress level in the FRPand prestressing steel determined for the assumed neutral axis depth,internal force equilibrium may be checked using Eq.(10.3.1.6f) and the same iterative approach to finding c as described in Section 10.3.1 used until convergence is attained.

The nominal flexural strength of the section with FRP external reinforcement can be computed using Eq.(10.3.1.6g).

10.3.2.3 Strength reduction factor—To maintain a sufficient degree of ductility,the concrete tension strain at the level of the tensile force resultant at the nominal strength should be checked.Adequate ductility is achieved if this strain exceeds 0.005.Where this strain cannot be achieved, the strength reduction factor is decreased to account for the less ductile failure.The strength reduction factor for a post-tensioned member is given by Eq.(10.3.2.3a)where ecis the strain in the concrete at the level of the tensile force resultant that may be calculated from Eq.(10.3.2.3b).

$$\phi=\begin{cases} 0.90 \text{ for } \varepsilon_{ct}\geq 0.005 \\ 0.65+\dfrac{0.25(\varepsilon_{ct}-0.002)}{0.005-0.002} \text{ for } 0.002<\varepsilon_{ct}<0.005 \\ 0.65 \text{ for } \varepsilon_{ct}\leq 0.002 \end{cases}$$

(10.3.2.3a)

$$\varepsilon_{ct}=\varepsilon_c\left(\frac{d_p-c}{c}\right) \qquad (10.3.2.3b)$$

10.4—Moment redistribution

Moment redistribution for continuous reinforced concrete beams strengthened using externally bonded FRP can be used to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment for any assumed loading arrangement by not more than 1000ε,percent,to a maximum of 20%.Moment redistribution is only permitted when the strain in the tension steel reinforcement,e,exceeds 0.0075 at the section at which moment is reduced.Moment redistribution is not permitted where approximate values of bending moments are used.

The reduced moment should be used for calculating redistributed moments at all other sections within the spans. Static equilibrium should be maintained after redistribution of moments for each loading arrangement.El-Refaie et al. (2003)demonstrated that continuous reinforced concrete beams strengthened with carbon FRP sheets can redistribute moment in the order of 6 to 31%.They also demonstrated that lower moment redistribution was achieved for beam sections retrofitted with greater amounts of carbon FRP reinforcement.Silva and Ibell (2008)demonstrated that sections that can develop a curvature ductility capacity greater than 2.0 can develop moment redistribution ofat least 7.5%of the design moment.

CHAPTER 11—SHEAR STRENGTHENING

Fiber-reinforced polymer(FRP)systems have been shown to increase the shear strength of existing concrete beams and columns by wrapping or partially wrapping the members (Malvar et al.1995;Chajes et al.1995;Norris et al.1997; Kachlakev and McCurry 2000).Orienting FRP fibers transverse to the axis of the member or perpendicular to potential shear cracks is effective in providing additional shear strength (Sato et al.1996).An increase in shear strength may be required when flexural strengthening is implemented to ensure that flexural capacity remains critical.

11.1—General considerations

This chapter presents guidance on the calculation of added shear strength resulting from the addition ofFRP shear reinforcement to a reinforced concrete beam or column.The additional shear strength that can be provided by the FRP system is based on many factors,including geometry of the beam or column,wrapping scheme,and existing concrete strength,but should be limited in accordance with the recommendations of Chapter 9.

Shear strengthening using external FRP may be provided at locations of expected plastic hinges or stress reversal and for enhancing post-yield flexural behavior of members in moment frames resisting seismic loads,as described in Chapter 13.

11.2—Wrapping schemes

The three types ofFRP wrapping schemes used to increase the shear strength of prismatic,rectangular beams,or columns are illustrated in Fig.11.2.Completely wrapping the FRP system around the section on all four sides is most commonly used in column applications where access to all four sides of the column is available.In beam applications where an integral slab makes it impractical to completely wrap the member,the shear strength can be improved by wrapping the FRP system around three sides of the member(U-wrap)or bonding to the two opposing sides of the member.

Although all three techniques have been shown to improve the shear strength of a rectangular member,completely wrapping the section is the most efficient,followed by the three-sided U-wrap.Bonding to two sides of a beam is the least efficient scheme.

For shear strengthening of circular members,only complete circumferential wrapping of the section in which the FRP is oriented perpendicular to the longitudinal axis of the member(that is,α=90 degrees)is recommended.

In allwrapping schemes,the FRP system can be installed continuously along the span of a member or placed as discrete strips.As discussed in 9.3.3,the potential effects of entrapping moisture in the substrate when using continuous reinforcement should be carefully considered.Specific means of allowing moisture vapor transmission out of the substrate should be employed where appropriate.

11.3—Nominal shear strength

The design shear strength of a concrete member strengthened with an FRP system should exceed the required shear strength(Eq.(11.3a)).The required shear strength of an FRP-strengthened concrete member should be computed with the load factors required by ACI 318.The design shear strength should be calculated by multiplying the nominal shear strength by the strength reduction factor φ,as specified by ACI 318:

$$\varphi Vn \geq Vu \tag{11.3a}$$

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 from the reinforcing steel(stirrups,ties,or spirals)and the concrete(Eq.(11.3b).An additional reduction factor y is applied to the contribution of the FRP system:

$$\varphi Vn=\varphi(Ve+V_s+yV) \tag{11.3b}$$

where V.and V,are the concrete and internal reinforcing steel contributions to shear capacity calculated using the provisions of ACI 318,respectively.For prestressed members,Ve is the minimum of V.and Vewdefined by ACI 318.

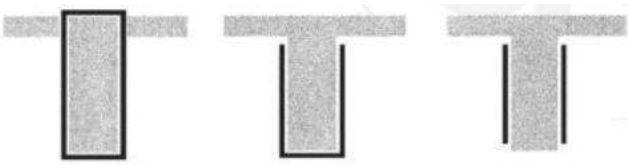


Fig.11.2—Typical wrapping schemes for shear strengthening using FRPlaminates.

Table 11.3—Recommended additional reduction factors for FRP shear reinforcement

4=0.95	Completely wrapped members
ψ=0.85	Three-side and two-opposite-sides schemes

Based on a reliability analysis using data from Bousselham and Chaallal(2006),Deniaud and Cheng(2001, 2003),Funakawa et al.(1997),Matthys and Triantafillou (2001), and Pellegrino and Modena(2002),the reduction factor y of 0.85 is recommended for the three-sided FRP U-wrap or two-opposite-sides strengthening schemes. Insufficient experimental data exist to perform a reliability analysis for fully-wrapped sections;however,there should be less variability with this strengthening scheme,as it is less bond-dependent and,therefore,the reduction factor y of 0.95 is recommended.The y,factor was calibrated based on design material properties.These recommendations are given in Table 11.3.

11.4—FRP contribution to shear strength

Figure 11.4 illustrates the dimensional variables used in shear-strengthening calculations for FRP laminates.The contribution of the FRP system to shear strength of amember is based on the fiber orientation and an assumed crack pattern(Khalifa et al.1998).The shear strength provided by the FRP reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across the assumed crack.The shear contribution of the FRP shear reinforcement is then given by Eq.(11.4a):

$$V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_{fv}}{s_f} \tag{11.4a}$$

For rectangular sections:

$$Ap=2Nt,w; \tag{11.4b}$$

For circular sections,d,is taken as 0.8 times the diameter of the section and:

$$A=(\pi/2)Ntyw \tag{11.4c}$$

The tensile stress in the FRP shear reinforcement at nominal strength is directly proportional to the strain that can be developed in the FRP shear reinforcement at nominal strength:

$$fe=E_I ce \tag{11.4d}$$

11.4.1 Effective strain in FRP laminates—The effective strain is the maximum strain that can be achieved in the FRP system at the nominal strength and is governed by the failure

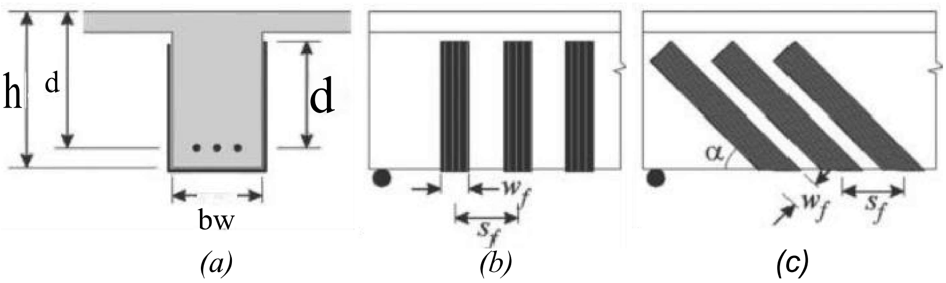


Fig.11.4Illustration of the dimensional variables used in shear-strengthening calculations for repair;retrofit,or strengtheningusing FRP laminates.

mode of the FRP system and of the strengthened reinforced concrete member. The licensed design professional should consider all possible failure modes and use an effective strain representative of the critical failure mode. The following subsections provide guidance on determining this effective strain for different configurations of FRP laminates used for shear strengthening of reinforced concrete members.

11.4.1.1 Completely wrapped members—For reinforced concrete column and beam members completely wrapped by FRP, loss of aggregate interlock of the concrete has been observed to occur at fiber strains less than the ultimate fiber strain. To preclude this mode of failure, the maximum strain used for design should be limited to 0.4% for members that are completely wrapped with FRP (Eq.(11.4.1.1)):

$$e=0.004\leq 0.758/u \quad (11.4.1.1)$$

This strain limitation is based on testing (Priestley et al. 1996) and experience. Higher strains should not be used for FRP shear-strengthening applications.

11.4.1.2 Bonded U-wraps or bonded face plies—FRP systems that do not completely wrap around 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 have been analyzed to determine the efficiency of these systems and the effective strain that can be achieved (Triantafillou 1998). The effective strain is calculated using a bond-reduction coefficient K, applicable to shear:

$$e=K,Epu\leq 0.004 \quad (11.4.1.2a)$$

The bond-reduction coefficient is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the laminate. The bond-reduction coefficient can be computed from Eq.(11.4.1.2b) through (11.4.1.2e) (Khalifa et al. 1998):

$$\kappa_v=\frac{k_1k_2L_e}{468\varepsilon_{fu}}\leq 0.75 \quad \text{(in.-1b)}$$
$$\kappa_v=\frac{k_1k_2L_e}{11{,}900\varepsilon_{fu}}\leq 0.755\text{(SI)} \quad (11.4.1.2b)$$

The active bond length Le is the length over which the majority of the bond stress is maintained. This length is given by Eq.(11.4.1.2c):

$$L_e=\frac{2500}{(Nt_fE_f)^{0.58}} \quad \text{(in.-lb)}$$
$$L_e=\frac{23{,}300}{(Nt_fE_f)^{0.58}} \quad \text{(SI)} \quad (11.4.1.2c)$$

The bond-reduction coefficient also relies on two modification factors, k_1 and k_2, that account for the concrete strength and the type of wrapping scheme used, respectively. Expressions for these modification factors are given in Eq.(11.4.1.2d) and (11.4.1.2e):

$$k_1=\left(\frac{f_c'}{4000}\right)^{2/3} \quad \text{(in.-lb)}$$
$$k_1=\left(\frac{f_c'}{27}\right)^{2/3} \quad \text{(SI)} \quad (11.4.1.2d)$$

$$k_2=\begin{cases}\dfrac{d_{fv}-L_e}{d_{fv}} & \text{for U-wraps}\\[2ex] \dfrac{d_{fv}-2L_e}{d_{fv}} & \text{for two sides bonded}\end{cases} \quad (11.4.1.2e)$$

The methodology for determining k, has been validated for members in regions of high shear and low moment, such as monotonically loaded simply supported beams. Although the methodology has not been confirmed for shear strengthening in areas subjected to combined high flexural and shear stresses or in regions where the web is primarily in compression (negative moment regions), K, is considered sufficiently conservative for such cases. The design procedures outlined herein have been developed by a combination of analytical and empirical results (Khalifa et al. 1998).

Anchorage details have been used to develop higher strains in bonded U-wraps used in shear strengthening applications. Anchorage systems include mechanical fasteners, FRP strips, fiber anchors, and near-surface-mounted (NSM) anchors; examples are shown schematically in Fig. 14.1.1 (Khalifa et al. 1999; Kalfat et al. 2018; Grelle and Sneed 2013). Properly anchored U-wraps can be designed to fail by FRP rupture (Belarbi et al. 2011). U-wraps anchored with fiber anchors designed to develop the fracture strength of the U-wraps have been shown to increase the shear strength of full-scale concrete bridge girders by 20 to 60% and to demonstrate favorable fatigue and long-term creep performance (Kim et al. 2012; Jirsa et al. 2017; Ghannoum et al. 2018). U-wraps may be anchored using fiber anchors detailed in accordance with 14.1.4, in which case the maximum U-wrap strain used for design should be that given by Eq.(11.4.1.1) and y=0.85.

11.4.2 Clear spacing—Clear spacing between discrete FRP strips used for shear strengthening should not exceed the lesser of d/4 or 12 in. (300 mm).

11.4.3 Reinforcement limits—The total shear strength provided by reinforcement should be taken as the sum of the contribution of the FRP shear reinforcement and the steel shear reinforcement. The sum of the shear strengths provided by the shear reinforcement should be limited based on the criteria given for steel alone in ACI 318:

$$V,+V,\leq 8\sqrt{fb},d \quad \text{(in.-1b)}$$
$$V,+V,\leq 0.66\sqrt{fb},d\text{(SD} \quad (11.4.3)$$

For circular sections,bd in Eq.(11.4.3)is taken as $0.8D^2$, where Dis the member diameter

CHAPTER 12—STRENGTHENING OF MEMBERS SUBJECTED TOAXIAL FORCE OR COMBINED AXIAL AND BENDNG FORCES

Confinement of reinforced concrete columns by means of fiber-reinforced polymer(FRP)jackets can be used to enhance their strength and ductility.An increase in capacity is calculated in terms of improved peak load resistance. Ductility enhancement,on the other hand,requires more complex calculations to determine the ability of a member to sustain rotation and drift without a substantial loss in strength.This chapter applies only to members confined with FRP systems.

12.1—Pure axial compression

FRPsystems can be used to increase the axial compression strength ofa concrete member by providing confinement with an FRP jacket(Nanni and Bradford 1995;Toutanji 1999). Confining a concrete member is accomplished by orienting the fibers transverse to the longitudinal axis of the member. In this orientation,the transverse or hoop fibers are similar to conventional spiral or tie reinforcing steel.Any contribution of longitudinally aligned fibers to the axial compression strength of a concrete member should be neglected.

FRP jackets provide passive confinement to the compression member,remaining unstressed until dilation and cracking of the wrapped compression member occur.For this reason,intimate contact between the FRPjacket and the concrete member is critical.

Depending on the level of confinement,the uniaxial stress-strain curve of a reinforced concrete column can be depicted by one of the curves in Fig.12.1a,where f'and fec represent the peak concrete strengths for unconfined and confined cases,respectively.These strengths are calculated as the peak load minus the contribution of the steel reinforcement,all divided by the cross-sectional area of the concrete.The ultimate strain of the unconfined member corresponding to 0.85f.(Curve (a))is Ecu.The strain Eccu corresponds to:a)0.85f'in the case of the lightly confined member(Curve(b));and b)the failure strain in both

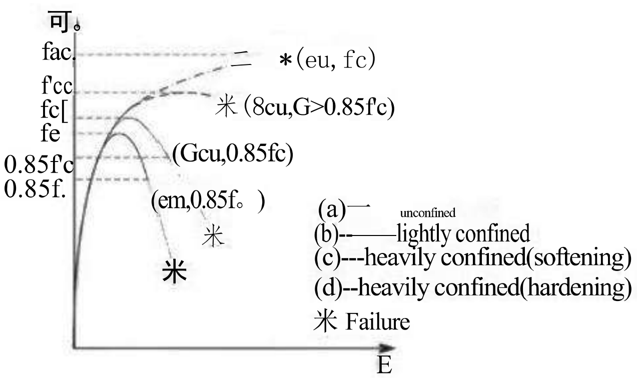


Fig.12.1a—Schematic stress-strain behavior ofunconfined and confined reinforced concrete columns(Rocca et al. 2006).

the heavily confined-softening case(the failure stress is larger than 0.85fc'[Curve(c)])or in the heavily confined-hardening case(Curve(d)).

The definition of Eccu at 0.85fcor less is arbitrary, although consistent with modeling of conventional concrete (Hognestad 1951).Furthermore,defining ccu in this manner results in the descending branch of the stress-strain curve at levels of stress greater than 0.85fe'not being as sensitive to the test procedure in terms of rate of loading or the stiffness of the equipment used.

The axial compressive strength of a nonslender normal-weight concrete member confined with an FRPjacket may be calculated using the confined concrete strength (Eq.(12.1a) and(12.1b)).The axial force acting on an FRP-strengthened concrete member should be computed using the load factors required byACI318,and the values of the φfactors as established in ACI 318 for both types of transverse reinforcing steel(spirals or ties)apply.

For nonprestressed members with existing steel spiral reinforcement:

$$\varphi P_n=0.85\varphi[0.85f'(Ag-As)+fA] \qquad (12.1a)$$

For nonprestressed members with existing steel-tie reinforcement:

$$\varphi P_n=0.8\varphi[0.85fe'(Ag-As)+fyAs] \qquad (12.1b)$$

Several models that simulate the stress-strain behavior ofFRP-confined compression sections are available in the literature(Teng et al.2002;De Lorenzis and Tepfers 2003; Lam and Teng 2003a).The stress-strain model by Lam and Teng (2003a,b)for FRP-confined concrete is illustrated in Fig.12.1b and computed using the following expressions:

$$f_c=\begin{cases} E_c\varepsilon_c-\dfrac{(E_c-E_2)^2}{4f_c'}\varepsilon_c^2 & 0\le\varepsilon_c\le\varepsilon_t' \\ f_c'+E_2\varepsilon_c & \varepsilon_t'\le\varepsilon_c\le\varepsilon_{ccu}\end{cases} \qquad (12.1c)$$

$$Eccu\le 0.01 \qquad (12.1d)$$

$$E_2=\frac{f_{cc}'-f'}{\varepsilon_{cc}} \qquad (12.1e)$$

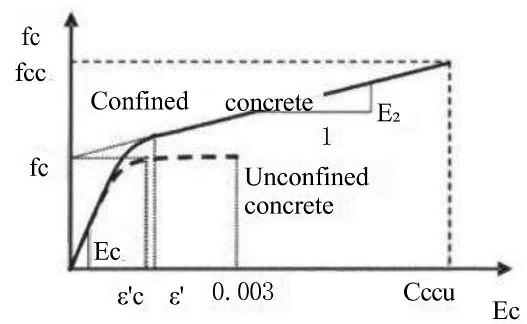


Fig.12.1b—Stress-strain modelfor FRP-confined concrete (Lam and Teng 2003a).

$$\varepsilon_t' = \frac{2f_c'}{E_c - E_2} \tag{12.1f}$$

The maximum confined concrete compressive strength, f',and the maximum confinement pressuref are calculated using Eq.(12.1g)and(12.1h),respectively(Lam and Teng 2003a,b),with the inclusion of an additional reduction factor V=0.95:

$$fc'=fe'+\psi;3.3k_4fi \tag{12.1g}$$

$$f_l = \frac{2E_f N t_f \varepsilon_{fe}}{D} \tag{12.1h}$$

In Eq.(12.1g),f'is the unconfined cylinder compressive strength of concrete,and the efficiency factor Kaaccounts for the geometry of the section,circular and noncircular,as defined in 12.1.1 and 12.1.2.In Eq.(12.1h),the effective strain in the FRP at failure,Efe,is given by:

$$e=K_6Epu \tag{12.1i}$$

The FRP strain efficiency factor K,accounts for the premature failure of the FRP system(Pessiki et al.2001),related primarily to stress concentration regions caused by cracking of the concrete as it dilates.Based on experimental calibration using mainly carbon FRP(CFRP)-confined concrete specimens,an average value of 0.59 was computed for Kg by Lam and Teng(2003a).Similarly,a database of 251 test results(Harries and Carey 2003)computed a value of Kg= 0.58,whereas experimental tests on medium-and large-scale columns resulted in values of Kg=0.57 and 0.61,respectively(Carey and Harries 2005).

Based on tests by Lam and Teng (2003a,b),the ratio fl fc'should not be less than 0.08.This is the minimum level of confinement required to assure a nondescending second branch in the stress-strain performance,as shown by Curve(d) in Fig.12.1a.This limitation was confirmed for circular cross sections by Spoelstra and Monti(1999)using their analytical model.A strain efficiency factor K of 0.55 and a minimum confinement ratiofif'of 0.08 should be used.

The maximum compressive strain in the FRP-confined concrete,&ccu,can be found using Eq.(12.1j)(Concrete Society 2004).The maximum concrete strain,Eccu,used in Eq.(12.1c)should be limited to 0.01 to prevent excessive cracking and the resulting loss of concrete integrity.

$$\varepsilon_{ccu} = \varepsilon_c'\left(1.50 + 12\kappa_b \frac{f_l}{f_c'}\left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right) \tag{12.1j}$$

In Eq.(12.1j),the efficiency factor Kg accounts for the geometry of the section in the calculation of the ultimate axial strain,as defined in 12.1.1 and 12.1.2.

Strength enhancement for compression members with fe' of 10,000 psi(70 MPa)or higher has not been experimentally verified.Enhancement of concrete having strength fc in excess of 10,000 psi(70 MPa)should be based on experimental testing.

12.1.1 Circular cross sections—FRP jackets are most effective at confining members with circular cross sections (Demers and Neale 1999;Pessiki et al.2001;Harries and Carey 2003;Youssef 2003;Matthys et al.2005;Rocca et al. 2006).The FRPsystem provides a circumferentially uniform confining pressure to the radial expansion of the compression member when the fibers are aligned transverse to the longitudinal axis of the member.For circular cross sections, the shape factors kaand k,in Eq.(12.1g)and(12.1j),respectively,can be taken as 1.0.

12.1.2 Noncircular cross sections—Testing has shown that confining square and rectangular members with FRP jackets can provide marginal increases in the maximum axial compressive strength fe'of the member(Pessiki et al. 2001;Wang and Restrepo 2001;Harries and Carey 2003; Youssef 2003;Rocca et al.2008).The provisions in this chapter are not recommended for members featuring side aspect ratios h/b greater than 2.0,or face dimensions b or h exceeding 36 in.(900 mm),unless testing demonstrates their effectiveness.

For noncircular cross sections,fe in Eq.(12.1h)corresponds to the maximum confining pressure of an equivalent circular cross section with diameter D equal to the diagonal of the rectangular cross section.

$$D=\sqrt{b^2+h^2} \tag{12.1.2a}$$

The shape factors rain Eq.(12.1g)and k,in Eq.(12.1j) depend on two parameters:the cross-sectional area of effectively confined concrete Ag,and the side-aspect ratio h/b,as shown in Eq.(12.1.2b)and(12.1.2c),respectively:

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 \tag{12.1.2b}$$

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5} \tag{12.1.2c}$$

The generally accepted theoretical approach for the definition of Ae consists of four parabolas within which the concrete is fully confined,outside of which negligible confinement occurs(Fig.12.1.2).The shape of the parabolas and the resulting effective confinement area is a function of the dimensions of the column(b and h),the radius of the corners re,and the longitudinal steel reinforcement ratio pg, and can be expressed as:

$$\frac{A_e}{A_c} = \frac{1 - \dfrac{\left[\left(\dfrac{b}{h}\right)(h-2r_c)^2 + \left(\dfrac{h}{b}\right)(b-2r_c)^2\right]}{3A_g} - \rho_g}{1-\rho_g} \tag{12.1.2d}$$

12.1.3 Serviceability considerations—As loads approach factored load levels,damage to the concrete in the form of significant cracking in the radial direction might occur.The FRPjacket contains the damage and maintains the structural integrity of the column.At service load levels,however, this type of damage should be avoided.In this way,the FRP jacket will only act during overloading conditions that are temporary in nature.

To ensure that radial cracking will not occur under service loads,the transverse strain in the concrete should remain below its cracking strain at service load levels.This corresponds to limiting the compressive stress in the concrete to 0.65f.In addition,the service stress in the longitudinal steel should remain below 0.60f,to avoid plastic deformation under sustained or cyclic loads.By maintaining the specified stress in the concrete at service,the stress in the FRPjacket willbe relatively low.The jacket is only stressed to significant levels when the concrete is transversely strained above the cracking strain and the transverse expansion becomes large.Service load stresses in the FRP jacket should never exceed the creep rupture stress limit.In addition,axial deformations under service loads should be investigated to evaluate their effect on the performance ofthe structure.

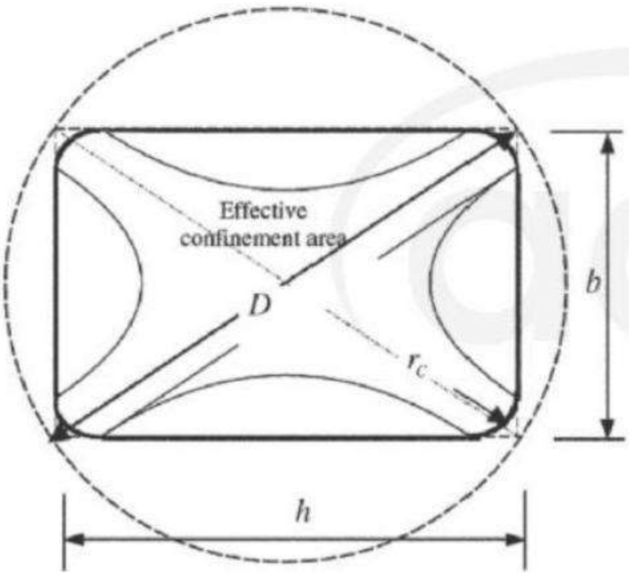


Fig.12.1.2—Equivalent circular cross section (Lam and Teng 2003b).

12.2—Combined axial compression and bending

Wrapping with an FRP jacket can also provide strength enhancement for a member subjected to combined axial compression and flexure(Nosho 1996;Saadatmanesh et al. 1996;Chaallal and Shahawy 2000;Sheikh and Yau 2002; lacobucci et al.2003;Bousias et al.2004;Elnabelsy and Saatcioglu 2004;Harajli and Rteil 2004;Sause et al.2004; Memon and Sheikh 2005).

For predicting the effect ofFRP confinement on strength enhancement,Eq.(12.1a)and(12.1b)are applicable when the eccentricity present in the member is less than or equal to 0.1h,where h is the dimension ofthe member section perpendicular to the axis ofbending.When the eccentricity is greater than 0.1h,the methodology and equations presented in 12.1 can be used to determine the concrete material properties ofthe member cross section under compressive stress. Based on that,the axial load-moment(P-M)interaction diagram for the FRP-confined member can be constructed using well-established procedures(Bank 2006).

The following limitations apply for members subjected to combined axial compression and bending:

(a)The effective strain in the FRPjacket should be limited to the value given in Eq.(12.2)to ensure the shear integrity ofthe confined concrete:

$$e=0.004\leq KEfu \qquad (12.2)$$

(b)The strengthenhancement canonly be considered when the applied ultimate axial force and bending moment, Pand M,respectively,fall above the line connecting the origin and the balanced point in the P-M diagram for the unconfined member(Fig.12.2).This limitation stems from the fact that strength enhancement is only significant for members in which compression failure is the controlling mode(Bank 2006).

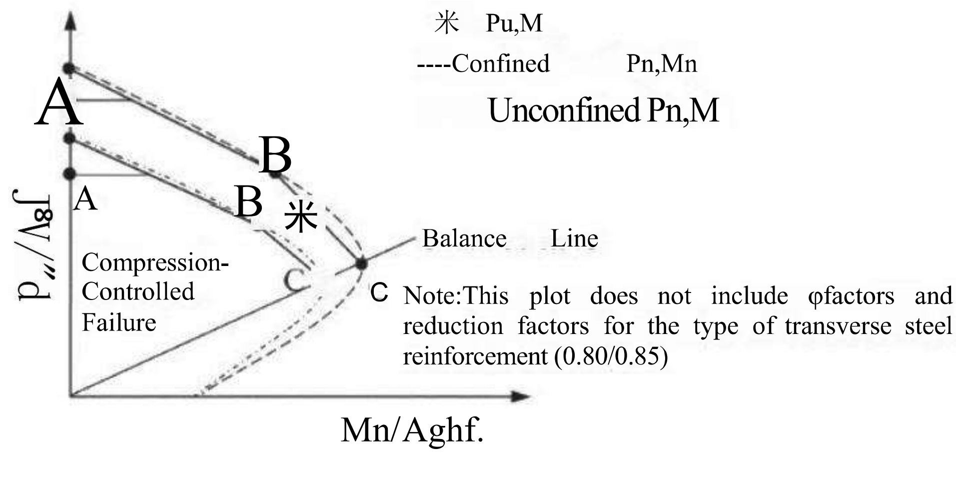


Fig.12.2—Representative interaction diagram.

P-M diagrams may be developed by satisfying strain compatibility and force equilibrium using the model for the stress-strain behavior for FRP-confined concrete presented in Eq.(12.1c)through(12.1f).For simplicity,the portion of the unconfined and confined P-M diagrams corresponding to compression-controlled failure can be reduced to two bilinear curves passing through three points (Fig.12.2).For values of eccentricity greater than 0.1h and up to the point corresponding to the balanced condition,the methodology provided in Appendix C may be used for the computation of a simplified P-M interaction diagram.The values of the φ factors as established inACI 318 for both types oftransverse reinforcing steel(spirals or ties)apply.

12.3—Ductility enhancement

Increased ductility of a section results from the ability to develop greater compressive strains in the concrete before compressive failure(Seible et al.1997).The FRPjacket can also serve to delay buckling of longitudinal steel reinforcement in compression and to clamp lap splices of longitudinal steel reinforcement.

For seismic applications,FRPjackets should be designed to provide a confining stress sufficient to develop concrete compression strains associated with the displacement demands as described in Chapter 13 .Shear forces should also be evaluated in accordance with Chapter 11 to prevent brittle shear failure in accordance with ACI 318.

12.3.1 Circular cross sections—The maximum compressive strain for FRP-confined members with circular cross sections can be found from Eq.(12.1j)with fc from Eq.(12.1g)and using Kb=1.0.

12.3.2 Noncircular cross sections—The maximum compressive strain for FRP-confined members with square or rectangular sections can be found from Eq.(12.1j),with fæ'from Eq.(12.1g),and using Kpas given in Eq.(12.1.2c). The confining effect ofFRP jackets should be assumed to be negligible for rectangular sections with aspect ratio h/b exceeding 2.0,or face dimensions b or h exceeding 36 in. (900 mm),unless testing demonstrates their effectiveness.

12.4—Pure axial tension

FRP systems can be used to provide additional tensile strength to a concrete member.Due to the linear-elastic nature of FRP systems,the tensile contribution of the FRP system is directly related to its strain and is calculated using Hooke's Law.

The tension capacity provided by the FRPis limited by the design tensile strength of the FRP and the ability to transfer stresses into the substrate through bond(Nanni et al.1997). The effective strain in the FRP can be determined based on the criteria given for shear strengthening in Eq.(11.4.1.1) through(11.4.1.2d).The value ofk2in Eq.(11.4.1.2b)can be taken as 1.0.A minimum bonded length of laj,as calculated in 14.1.3 ,should be provided to develop this level of strain.

CHAPTER 13—SEISMIC STRENGTHENING

Many strengthening techniques have been developed and used for repair and rehabilitation of earthquake damaged and seismically deficient structures(Federal Emergency Management Agency 2006).Identification of an effective rehabilitation method is directly related to the outcome of a seismic evaluation of the structure and is based on consideration of many factors,including type of structure,rehabilitation objective,strengthening scheme effectiveness, constructability,and cost.

A classification of seismic rehabilitation methods for buildings in ASCE/SEI 41 and ACI 369R provides information about the following strategies:local modification of components,removal or lessening of existing irregularities and discontinuities,global structural stiffening,global structural strengthening,mass reduction,seismic isolation, and supplemental energy dissipation.Strengthening using FRP systems allows for local modification of components and can be implemented in improving the overall seismic performance of the structure.The main advantages of FRP strengthening can be summarized as follows:

(a)At the component level,FRP strengthening can be used to efficiently mitigate brittle mechanisms of failure.These may include shear failure of unconfined beam-column joints;shear failure of beams,columns, or both;and lap splice failure.FRP strengthening can also be used to increase the flexural capacity of reinforced concrete members,to resist the buckling of flexural steel bars,and to increase the inelastic rotational capacity of reinforced concrete members.

(b)Implementing FRP strengthening schemes translates into an increase in the global displacement and energy dissipation capacities of the structure,thus improving the overall behavior of reinforced concrete structures subjected to seismic actions.

(c)FRP strengthening and confinement has a small effect on the stiffness or mass of the structure.In such cases, a reevaluation of the seismic demand after strengthening is typically not required.When the structural stiffness needs to be increased,FRP strengthening of local components can be coupled with other traditional local or global upgrade techniques.

Many research programs have evaluated the adequacy of externally bonded FRP composites for seismic rehabilitation of concrete structures (Haroun et al.2005;Pantelides et al.2000;Ghobarah and Said 2002;Gergely et al.2000; Antonopoulos and Triantafillou 2002;Hamed and Rabinovitch 2005;Pampanin et al.2007;DiLudovico et al.2008a). Other research programs have confirmed the potential of FRP techniques for upgrading the seismic performance of local elements such as reinforced concrete columns(Bousias et al.2004)and connections(Antonopoulos and Triantafillou 2002; Prota et al.2004).Research results for FRP applied at the local element or partial structural frame level were subsequently validated on full-scale structures(Pantelides et al. 2000,2004;Balsamo et al.2005;Engindeniz et al.2008a,b). In addition,several structures that include FRP-strengthened members have experienced seismic events.Failure of these members has not been reported.

This chapter presents design guidelines for the seismic strengthening of reinforced concrete elements using externally bonded FRP.The design guidelines described herein are intended to be used in conjunction with the fundamental concepts,analysis procedures,design philosophy,seismic rehabilitation objectives,and acceptance criteria set forth in documents such as ASCE/SEI 41 and ACI 369R.Strengthening of reinforced concrete(RC)building components or structures with FRP should follow capacity protection principles.In capacity design(Hollings 1968;Park and Paulay 1976),a desirable mechanism of inelastic response under seismic action is ensured by providing a strength hierarchy(for example,strong column-weak beam;shear strength>flexural strength).In certain conditions,particularly when the reinforced concrete member is not completely wrapped around its perimeter,the governing failure mode for the FRP-strengthened member may be a FRP debonding failure.In such conditions,the ductility requirements of the FRP-strengthened member should be assessed.This may be relevant when the member is required to possess ductility beyond the seismic demand levels for which the FRP is designed.The brittle debonding failure mode can be controlled by appropriately anchoring the FRP.Application of these design guidelines for the seismic rehabilitation of nonbuilding structures such as bridges,wharves,silos,and nuclear facilities warrant additional consideration.

These guidelines do not provide information required to complete a seismic evaluation of an existing structure,determine if retrofit is required,or identify the seismic deficiencies that need to be corrected to achieve the desired performance objective.These guidelines are also not intended to address post-seismic conditions or residual strength of either the structure or the FRP retrofit system.After a seismic event,a structure that has been retrofitted with FRP composites could develop large displacements and excessive cracking,resulting in residual stresses or damage to the FRP system.An investigation of the stability,ductility, and residual strength of the structure should be performed to assess the adequacy of the existing FRP retrofit system and to determine if remedial measures are needed.

Because seismic events and fire are both considered to be extreme events,fire protection is typically not required for FRP reinforcement designed solely for seismic strengthening and the fire design limit in Section 9.2.1 need not be considered(ACI PRC-440.10) .

13.1—Background

One of the most comprehensive documents developed to assess the need for seismic rehabilitation of reinforced concrete buildings is ASCE/SEI 41.FEMA P695(Federal Emergency Management Agency 2009) provides further guidance for the selection of appropriate design criteria to achieve the seismic performance objectives.

FEMA(Federal Emergency Management Agency 2006) provides an extensive list of references on technical design standards and analysis techniques that are available to design professionals.Other resources dealing with seismic upgrade of existing reinforced concrete structures can be obtained from Japan Building Disaster Prevention Association(2005),Eurocode 8(EN 1998-3),International Federation for Structural Concrete(2003,2006),Italian National Research Council(2004), and Sabnis et al.(1996).

Experience gained from examining the performance of reinforced concrete structures after a seismic event indicates that many structural deficiencies result from inadequate confinement of plastic hinge regions,insufficient transverse and continuity reinforcement in connections and structural members,buckling of flexural reinforcement,lap splice failures,and anchorage failures(Pessiki et al.1990;Priestley et al.1996;Haroun et al.2003;Sezen et al.2003;Pantelides et al.1999,2004).These deficiencies have typically led to brittle failures,soft-story failure,and large residual displacements(Moehle et al.2002;Di Ludovico et al.2008b;Prota et al.2004).Experimental work has also demonstrated that externally bonded FRPsystems can be effective in addressing many of the aforementioned structural deficiencies(Engindeniz et al.2005;Pantelides et al.2008;Silva et al.2007).

13.2—FRP properties for seismic design

For seismic upgrades,the material environmental factors given in Table 9.4 should be used in the design of the FRP strengthening solution.The creep rupture limits in Table 10.2.9 need not be considered for seismic strengthening applications unless initial strains are imposed on the FRP as part of the retrofit scheme.Typically,when used for seismic retrofit,the FRP system will not be exposed to significant sustained service loads and creep rupture failure will not govern the design.Creep rupture limits should be considered,however,in cases where the application may impose initial or service strains that can produce sustained stresses on the FRP.Some examples include applications with expansive grouts,pretensioned FRP,or other methods that generate sustained stress in the FRP system.When this chapter is used in conjunction with ASCE/SEI 41,FRP material properties should be considered lower-bound material properties.

13.3—Confinement with FRP

Jacketing concrete structural members with FRP having the primary fibers oriented around the perimeter of the member provides confinement to plastic hinges,mitigates the splitting failure mode of poorly detailed lap splices,and prevents buckling of the main reinforcing bars.

13.3.1 General considerations—In seismic applications, jacketing concrete structural members with FRP is not recommended for rectangular sections with aspect ratios h/b greater than 1.5,or face dimensions b or h exceeding 36 in. (900 mm)(Seible et al.1997),unless testing demonstrates the effectiveness ofFRP for confinement of these members. For rectangular sections with an aspect ratio greater than 1.5, the section can be modified to be circular or oval to enhance the effectiveness of the FRPjacket (Seible et al.1997).Fiber anchors have been shown to increase the effectiveness of the FRP jacket in rectangular sections with aspect ratios as great as 2.0(Kim et al.2011).Such anchors should extend through the width or depth of the section and be splayed on opposite faces.The anchors should be spaced longitudinally and transversely in accordance with requirements for hoops and cross ties for columns in special moment frames given in ACI 318.

13.3.2 Plastic hinge region confinement—FRP-jacketed reinforced concrete members can achieve higher inelastic rotational capacity of the plastic hinge(Seible et al.1997). FRPjacketing can be used to increase the concrete compressive strength as described in 12.2 and 12.3.Increase in flexural strength of confined sections due to greater concrete compressive strain capacity should be considered to verify that hinges can form prior to reaching the shear strength of members.

The design curvature φD for a confined reinforced concrete section at the plastic hinge can be calculated using Eq.(13.3.2a).

$$\phi_D = \frac{\theta_p}{L_p} + \phi_{y,frp} \leq \phi_{u,frp} \qquad (13.3.2a)$$

whereθ,is the plastic rotation demand,which can be determined following the analytical procedures outlined inASCE/SEI 41.In Eq.(13.3.2a),the curvatures ofthe FRP-confined section at steel yielding,中yp,and at ultimate capacity,中ufip, are determined by Eq.(13.3.2b)and(13.3.2c),and Lp is the plastic hinge length computed using Eq.(13.3.2d).

$$\phi_{y,frp} = \frac{\varepsilon_{sy}}{d - c_{y,frp}} \qquad (13.3.2b)$$

where ε,and cyip are the steel strain and depth of the neutral axis as presented in ASCE/SEI41,and dis the distance from the extreme compression fibers to the extreme tension steel.

$$\phi_{u,frp} = \frac{\varepsilon_{ccu}}{c_{u,frp}} \qquad (13.3.2c)$$

where Ecu and Cufp are the ultimate extreme compression fiber strain and corresponding depth of the neutral axis,respectively.

For beams,the plastic hinge length should be determined using detailed analysis.In FRP-jacketed columns,the plastic hinge length Lp can be computed using Eq.(13.3.2d) (Priestley et al.1996).

Lp=g+0.0003f,dbe(in.-1b)

(13.3.2d)

L=g+0.044f,ds(SI)

where dse and f,are the diameter and yield stress of the flexural steel,respectively,and g is the clear gap between the FRP jacket and adjacent members,as shown in Fig.13.3.2. The gap g should not be greater than 2 in.(50.8 mm).

In plastic hinge regions for beams,the FRP confinement should be provided over a length not less than twice the beam height(2h).In plastic hinge regions for columns,the FRP confinement should be provided over a length not less than the greater of the plastic hinge length andl₀,wherel。is the length,measured along the member axis from the face of

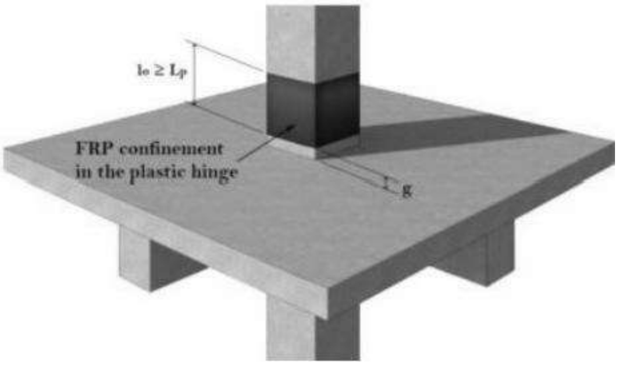


Fig.13.3.2—Column plastic hinge confinement.

the joint,over which special transverse reinforcement must be provided as defined in Chapter 18 of ACI 318-19(22). Plastic hinges may occur at locations other than the ends of the member.Complete wrapping around the perimeter of the member should be used for confinement.Continuous (full)coverage with an FRP jacket is recommended.When a continuous jacket is not possible,discrete transverse FRP strips around the perimeter of the section can be used.

Once the design curvature φo has been established,the ultimate extreme compression fiber strain in the concrete, Ccu,can be calculated using Eq.(13.3.2e).

Cecu=φDCufim≤0.01 (13.3.2e)

where Cun is the neutral axis depth at the ultimate design limit state.For members subjected to combined axial and flexural forces,&ccu should be limited to 0.01.

Once &ccu is determined,the required thickness of the FRP jacket can be determined in accordance with 12.1 and 12.2. To ensure the shear integrity of the confined concrete section, the effective design strain in the FRP jacket,ge,should be limited to the value given by Eq.(12.2).

13.3.3 Lap splice clamping—The capacity of lap splices having inadequate lap length,especially those located in plastic hinge regions,can be improved by continuously confining the section over at least the length of the splice with externally bonded FRP(Seible et al.1997;Haroun and Elsanadedy 2005).The required thickness of the FRPjacket can be calculated as follows:

circular sections:Nt=145(D/E)

(13.3.3a)

rectangular sections:Nty=218(D/E)

where D is in inches and E is in ksi.

circular sections:Nty=1000(D/E)

rectangular sections:Nty=1500(D/E)

where D is in mm and E is in MPa.

where D is the diameter of a circular member or the diameter of an equivalent circular column for prismatic members (per Eq.(12.1.2a));and E,is the tensile modulus of the FRPjacket.

While confining the section with FRPcan mitigate the splitting mode of failure,the pullout failure mode may control the capacity of the confined lap splice.Therefore,regardless of FRP retrofit,the stress in the flexural reinforcing bar,fs, should not exceed the limit given in Eq.(13.3.3b)(Harries et al.2006).

$$f_s \le \frac{33\ell_{prov}\sqrt{f_c'}}{d_{bf}\Psi_t\Psi_e\Psi_s} \quad \text{(in.-1b)}$$

$$f_s \le \frac{2.75\ell_{prov}\sqrt{f_c'}}{d_{bf}\Psi_t\Psi_e\Psi_s} \quad \text{(SD} \qquad (13.3.3b)$$

where lpov is the length of splice provided;dbe is the diameter of the flexural reinforcement;and they factors are those given in Section 25.4 of ACI 318-19(22).

13.3.4 Preventing buckling of flexural steel bars—Continuous or discrete FRP strips having the primary fibers oriented around the perimeter of the member can be used to prevent buckling of the longitudinal steel bars(Priestley et al.1996).For circular sections,the volumetric reinforcement ratio provided by transverse FRP,p,is:

$$\rho_f = \frac{4Nt_f}{D}\frac{w_f}{s_f} \qquad (13.3.4a)$$

where w is the FRP strip width;and sy is the center-to-center spacing of the FRP strips.For continuous confinement,w/S=1.In rectangular sections,the volumetric reinforcement ratio provided by the FRP,p,is(Priestley et al.1996):

$$\rho_f = 2Nt_f\left(\frac{b+h}{bh}\right)\frac{w_f}{s_f} \qquad (13.3.4b)$$

The amount of volumetric transverse reinforcement ratio should be at least:

$$\rho_f \ge \frac{0.0052\rho_f D}{d_{bf}}\frac{f_y}{f_{fe}} \qquad (13.3.4c)$$

where pris the flexural reinforcement ratio;D is the diameter of a circular section or the diameter of an equivalent circular column for prismatic members(per Eq.(12.1.2a));dbe andf are the diameter and the yield strength of the flexural reinforcement,respectively;pr is the volumetric transverse reinforcement ratio computed by Eq.(13.3.4a)or(13.3.4b);and fe is the effective design stress in the FRP jacket computed by Eq.(13.3.4d):

Jfje=8/E (13.3.4d)

where sge is the effective design strain in the FRP jacket given by Eq.(12.2).

When discrete FRP strips rather than a continuous jacket are used,the clear spacing between FRP strips should not exceed the limits in Eq.(13.3.4e):

$$s_f \le [3+6]\left(\frac{f_u}{f_y}-1\right)d_{bf} \le 6d_{bf} \qquad (13.3.4e)$$

where f,f,and dbe are the ultimate and yield strengths and the smallest diameter of the internal flexural reinforcement,respectively.The clear spacing should not exceed 6 in.(150 mm).These requirements ensure that if the cover concrete spalls in the region between strips,the FRP can provide sufficient resistance against bar buckling. This approach neglects any contribution from the existing internal transverse reinforcement because the internal ties may not coincide with the gaps between the FRP strips,and the interaction of the internal ties and external FRP strips has not been studied.

13.4—Flexural strengthening

The flexural capacity of reinforced concrete beams and columns in expected plastic hinge regions can be enhanced using FRP only in cases where strengthening will eliminate inelastic deformations in the strengthened region and transfer inelastic deformations to other locations in the member or the structure that are able to handle the ensuing ductility demands.The required flexural strength should be calculated in accordance with the design standard being used for rehabilitation,such as ASCE/SEI41 andAC1369R. When this chapter is used in conjunction with ASCE/SEI 41,the strengthened reinforced concrete members with FRP should be considered force-controlled unless a deformation-controlled classification can be justified based on experimental data.

The flexural capacity of reinforced concrete beams and columns can be enhanced using the design methodology presented in Chapter 10.The flexural strength φMshould satisfy the requirement of Eq.(13.4):

$$\varphi M \ge M \qquad (13.4)$$

where M is the ultimate moment demand resulting from combined gravity and seismic demands.The flexural capacity of reinforced concrete members should be evaluated based on concrete and reinforcing steel strain limits set forth in the design standard.ASCE/SEI 41 provides a comprehensive list of concrete and reinforcing steel strain limits.In addition,the stress in the reinforcing steel should be limited to the stress that can be achieved based on the existing development lengths and lap-splice details.The strength reduction factor φ is per the design standard being used for the rehabilitation.The additional strength reduction factor for FRP,ψ,is applied to the flexural contribution of the FRP reinforcement as described in 10.2.10.

13.4.2 Development and anchorage offlexural FRP reinforcement—This section provides conceptual methods for anchorage of flexural FRP reinforcement under seismic

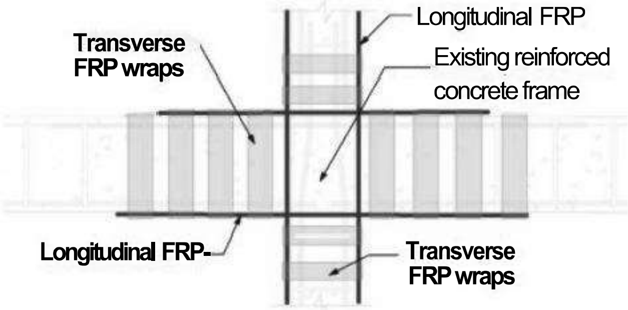


Fig.13.4.2—Conceptual FRP strengthening detail(cross section elevation)

loads.Any anchorage method must be properly evaluated before it is selected for field implementation.

In seismic applications and within plastic hinge regions, the flexural FRP reinforcement should be confined using FRP strips that completely wrap around the perimeter of the section.Alternatively,the flexural FRP reinforcement could be confined over its entire length to provide higher resistance against debonding.Because no anchorage design guidelines for seismic applications are currently available, the performance of any anchorage system should be substantiated through representative physical testing.

Such detailing provides higher resistance against debonding of the flexural FRP reinforcement.In applications involving floor systems,complete wrapping of the beam may require localized cutting of the slab to continue the FRP around the section.

Away from the plastic hinge region,transverse FRP U-wrap strips should be used to provide anchorage to the FRP flexural reinforcement.Other anchorage systems may also be used alone or in conjunction with FRPU-wrap strips. Anchorage systems should be verified experimentally to demonstrate their effectiveness in preventing the debonding of the flexural FRP reinforcement.Several details for FRP anchorage at ends of retrofitted members are discussed in Orton et al.(2009).

The area ofthe transverse FRPwrap reinforcement,Ajanchors should be determined in accordance with Eq.(14.1.2).In addition,the length over which the FRP anchorage wraps are provided,l.E,should not be less than the value given by Eq.(13.4.2a).

$$laE\geq_0+84 \qquad (13.4.2a)$$

where l。is defined in Fig.13.3.2,and lar is the required development length of the FRP system computed using Eq.(14.1.3).

For adequately anchored flexural FRP reinforcement,the effective design strain for FRP should be limited to:

$$8a\leq 0.908fi \qquad (13.4.2b)$$

Figure 13.4.2 depicts a conceptual detail for flexural strengthening of beams and columns at ajoint and is intended to convey the critical elements of such a flexural strengthening.The design professional should detail the flexural FRP reinforcement to achieve continuity of the FRP across the joint.Appropriate development of the flexural FRP at both ends as well as adequate transverse reinforcement for confinement of the flexural FRP should be provided.

13.5—Shear strengthening

FRP shear strengthening can prevent brittle failures and promote the development of plastic hinges,resulting in an enhanced seismic behavior of concrete members.The design shear strength φV,of a concrete member strengthened with FRP should satisfy Eq.(13.5):

$$\varphi V\geq V \qquad (13.5)$$

where φ is per the design standard being used for the rehabilitation,and V.is the design shear force.When this chapter is used in conjunction with ASCE/SEI 41,the shear in the strengthened member should be considered force-controlled unless a deformation-controlled classification can be justified based on experimental data.

13.5.1 Design shear force V.—The design shear force should be calculated in accordance with the design standard being used for the rehabilitation,such as ASCE/SEI 41 and ACI 369R. The shear capacity should be equal to or greater than the shear corresponding to the flexural capacity of the section.For example,when the rehabilitation is based on ASCE/SEI41,the design shear force is based on the seismic category and targeted seismic performance of the structure. When required by ASCE/SEI 41 for the determination of the design shear force,the calculation of the probable flexural strength should be based on FRP stress taken as the lesser of 1.2fa and fiu,o should be taken as unity,and the FRP strength reduction factor ψ,should be 1.0.Other limits for FRP strain and strength specified in this document should also be considered.

13.5.2 Nominal shear strength V—The shear strength of the existing member V,*should be determined following the procedures described in the design standard being used for rehabilitation,such as ASCE/SEI 41 and ACI 369R.The shear strength of an FRP-strengthened concrete member is calculated using Eq.(13.5.2).

$$Vn=V,*+yV, \qquad (13.5.2)$$

where y is the reduction factor applied to the contribution of the FRP system in accordance with Chapter 11.The contributions of FRPto shear strength,V,should be determined in accordance with Chapter 11.To account for effects of stress reversal,FRP shear strengthening should be fully wrapped around the perimeter of the section (refer to 11.2).

13.6—Beam-column joints

Experimental tests(Bracci et al.1992a;Prota et al.2004; Pampanin et al.2007)and observations of post-seismic damage(Moehle et al.2002)in structures designed to withstand only gravity loads show that unconfined beam-column joints frequently lead to brittle failures and prevent structures from achieving higher global displacements before failure.Experimental evidence(Pantelides et al.2008;Silva et al.2007;Pampanin et al.2007;Bracci et al.1992b,c)has shown that FRP systems can be effective for increasing the shear and energy dissipation capacity ofunconfined joints. FRPlayout and detailing will depend on the geometry of the existing joint and the number of members framing into it. FRP reinforcement in both directions is typically required at the joint to resist the cyclic loading effects of a seismic event(Engindeniz et al.2008a).The FRPused to confine the joints should be anchored to be effective.Pantelides et al.(2008),Silva et al.(2007),and Engindeniz et al.(2008b) provide guidance on determining if FRP is a viable option for enhancing the performance ofunconfined joints.Additionally,FRP reinforcement can be used to provide continuity acrossjoints with discontinuous internal reinforcement (Orton et al.2009).

13.7—Strengthening reinforced concrete shear walls and wall piers

13.7.1 General considerations—This section presents design guidelines for the seismic strengthening of reinforced concrete walls at risk of sustaining brittle failure during an earthquake.Applying horizontal FRP strips along the height of the walls can increase the in-plane shear strength of reinforced concrete shear walls.For walls with low height-to-length ratios(as defined by ACI 318),vertical FRP strips may also be used.Likewise,the in-plane flexural strength of reinforced concrete shear walls is increased by placing vertical FRP strips at the ends or boundaries of walls (Lombard et al.2000;Hiotakis et al.2004).

Ensuring that the shear strength of walls,strengthened in shear using FRP,is greater than the shear corresponding to the development of the expected flexural strength of the wall,including any amplification required by ACI 318,is good practice.When this section is used in conjunction with ASCE/SEI 41,shear should be considered a force-controlled action.If the shear strength of the strengthened wall is greater than the shear corresponding to the expected flexural strength of the wall,flexure may be considered a deformation-controlled action.

*13.7.2 Flexural strengthening—In-plane flexural strength*ening is achieved by applying longitudinal (that is,vertical) FRP at the ends of the shear wall,on one or both sides of the wall,to resist tensile stresses.The flexural strength of shear walls in regions where plastic hinging is expected can be enhanced with FRP only in cases where the strengthening will eliminate inelastic deformations in the strengthened region and transfer them to other locations in the wall or structure that are able to handle the ensuing ductility demands.Figure 13.7.2 shows a wall retrofitted with FRP reinforcement for flexural strengthening.This figure also provides a description of the main variables required for design.The strength reduction factor$\varphi$is per the design standard being used for the rehabilitation and the strengthened wall should satisfy Eq.(13.7.2).

$$\varphi Mn \geq M \qquad (13.7.2)$$

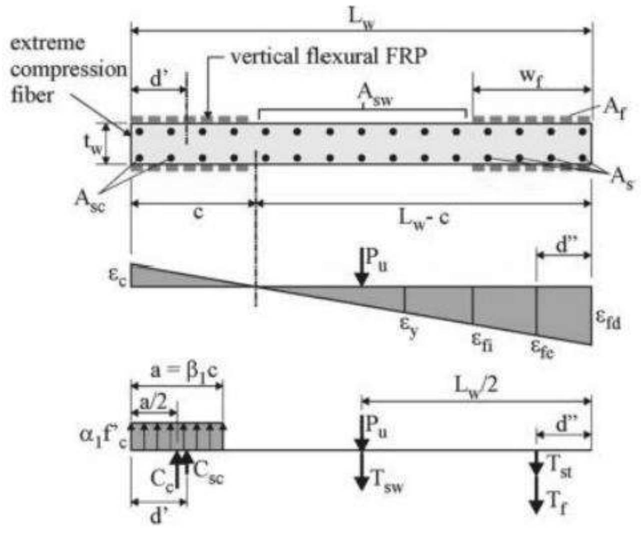


Fig.13.7.2—FRP reinforcementforflexural strengthening.

*13.7.2.1 Concrete strain limits—For flexural strength*ening,the concrete compressive strains εe should be limited by Eq.(13.7.2.1):

$$\varepsilon_c = \varepsilon_{fd}\left(\frac{1}{L_w/c-1}\right) \leq 0.003 \qquad (13.7.2.1)$$

where εa corresponds to the strain at which debonding of the FRP may occur,per Eq.(10.1.1).Where FRP debonding controls failure,anchorage of the longitudinal FRP can be used to enhance flexural performance as discussed in Section 10.1.1.

*13.7.2.2 Anchorage,confinement,and continuity oflongi*tudinal FRP—In walls strengthened for flexure,the longitudinal FRP should be continuous through existing slabs and be anchored to the foundations to ensure load path continuity.Any anchorage method should be evaluated experimentally prior to field implementation.Where feasible, the longitudinal FRP reinforcement should be confined by transverse FRP strips or U-wraps that extend around the perimeter of the section.Other anchorage methods,such as fiber anchors,may be provided to control debonding of the longitudinal FRP.The strain used to calculate the capacity of the longitudinal FRP should not exceed the maximum strain achievable with the anchoring system and be justified with experimental data.

13.7.3 Confinement ofwall boundary elements inplastic hinge regions—In plastic hinge regions,where the concrete ultimate compression strain,Ecu,is expected to exceed 0.003, damage can be expected at the wall boundary elements as defined in Chapter 18 of ACI 318-19(22).To impart greater deformation capacity to the wall,the boundary element of the wall may need to be adequately confined.FRP can be used per 12.1 and 13.3 to achieve Eeu greater than 0.003 but not greater than 0.01.IfFRP is used for confinement of plastic hinge regions,such transverse FRP should be provided with complete continuity around the boundary element or be anchored outside the boundary element to develop the strength of the confining FRP at a strain of 0.004.The effectiveness of the anchorage method for the confinement FRP should be demonstrated by experimental testing.

13.7.4 Shear strengthening of reinforced concrete shear walls—Experimental investigations have demonstrated the effectiveness of FRP for enhancing the shear strength of reinforced concrete walls subjected to seismic or cyclic loading(Cruz-Noguez et al.2015;Woods et al.2016;Layssi et al.2012).The design shear strength φV,of a reinforced concrete shear wall strengthened with FRP should satisfy Eq.(13.7.2.4a).

$$\varphi Vn \geq V \tag{13.7.2.4a}$$

The strength reduction factor φ is per the design standard being used for the rehabilitation.

For shear walls with externally bonded FRP,the nominal shear strength V,can be computed using Eq.(13.7.2.4b):

$$Vn=V^{*}+\psi V \tag{13.7.2.4b}$$

where V is the nominal shear strength of the existing shear wall;y is the reduction factor applied to the contribution of the FRP in accordance with Chapter 11;and V,is the shear strength provided by the FRP.The shear strength enhancement for a wall section of length Lin the direction of the applied shear force,with total area of shear FRP,on one or both sides of the wall,A.spaced at s,can be calculated using Eq.(13.7.2.4c)(Haroun et al.2005):

$$\text{for a two-sided retrofit:} V=AnE_6E,ds/s;$$
$$\text{for a one-sided retrofit:} V=0.75AnE_6E,dj/s; \tag{13.7.2.4c}$$

where dp is the effective depth of the shear wall,as defined in 13.7.4.1 and eeis per 11.4.1.The maximum nominal shear strength of a wall segment should not exceed the value given in Eq.(13.7.2.4d):

$$V_0 \leq 10\sqrt{J!}A \tag{13.7.2.4d}$$

where Acw is area of concrete section of an individual pier or horizontal wall segment.

13.7.4.1 Detailing of FRP shear reinforcement—In regions where plastic hinging is not expected,the effective depth of the shear wall can be taken as dp=0.8Lw and anchorage of shear FRP is considered good practice. Anchoring the FRP will help to control the FRP debonding failure mode,thereby maintaining the integrity of the shear mechanism to larger wall deformations.Anchoring the FRP shear reinforcement can be achieved by wrapping the FRP layers around the ends of the wall,by using mechanical anchorage devices such as steel anchors and steel plates (Paterson and Mitchell 2003),or by using fiber anchors (Binici and Ozcebe 2006).

For shear strengthening in plastic hinge regions with confinement per 13.7.3,the effective depth can be taken as du=L.The transverse shear FRP should be extended to the wall ends with the confinement FRP installed over the transverse FRP.

If shear strengthening is provided in plastic hinge regions without boundary region confinement per 13.7.3,dyu should not exceed the smaller of 0.8L,the distance between the anchored ends of the shear FRP,and the wall length less the length of the boundary elements.The transverse shear FRP should be anchored outside the boundary elements.

When vertical FRP is used for shear strengthening,dp should not exceed the height of the wall,hw

The maximum clear spacing between the FRP shear strips should be limited to the minimum of one-fifth of the overall length of the wall,three times the thickness of the wall,and 18 in.(460 mm).

CHAPTER 14—FIBER-REINFORCED POLYMER REINFORCEMENT DETAILS

This chapter provides guidance for detailing externally bonded fiber-reinforced polymer(FRP)reinforcement. Detailing will typically depend on the geometry of the structure,the soundness and quality of the substrate,and the levels of load that are to be sustained by the FRP sheets or laminates.Many bond-related failures can be avoided by following these general guidelines for detailing FRP sheets or laminates:

(a)Do not turn inside corners such as at the intersection of beams and joists or the underside of slabs
(b)Provide a minimum 1/2 in.(13 mm)radius when the sheet is wrapped around outside corners
(c)Provide adequate development length
(d)Provide sufficient overlap when splicing FRP plies

14.1—Bond,delamination,and anchorage

The actual distribution of bond stress in an FRP laminate is complicated by cracking of the substrate concrete.The general elastic distribution of interfacial shear stress and normal stress along an FRP laminate bonded to uncracked concrete is shown in Fig.14.1.

The weak link in the concrete/FRP interface is the concrete.The soundness and tensile strength of the concrete substrate will limit the overall effectiveness of the bonded FRP system.Design requirements to control FRP debonding failure modes are discussed in 10.1.1.

14.1.1 FRP debonding—In reinforced concrete members having relatively long shear spans or where the end peeling (refer to 14.1.2)has been effectively mitigated,debonding may initiate at flexural cracks,flexural/shear cracks,or both, near the region of maximum moment.Under loading,these cracks open and induce high local interfacial shear stress that initiates FRP debonding that propagates across the shear span in the direction of decreasing moment.Typically, this failure does not engage the aggregate in the concrete, progressing through the thin mortar-rich layer comprising the surface of the concrete substrate.This failure mode is exacerbated in regions having a high shear-moment ratio.

Anchorage systems,such as U-wraps,fiber anchors, and near-surface-mounted(NSM)anchors(examples are shown schematically in Fig.14.1.1),have been proven to control debonding failure of externally bonded FRP (Eshwar et al.2003;Kalfat et al.2018;Grelle and Sneed 2013;Lee and Lopez 2020;Smith et al.2013).Numerical and experimental studies have shown that these systems can increase the effective strain in the flexural FRP to values up to tensile rupture(Lee et al.2010;Orton et al. 2008).A couple studies have proposed analytical models to predict the behavior of specific anchor systems(Kim and Smith 2010;del Rey Castillo et al.2019a);however, limited published anchorage design guidelines are currently available.Therefore,the performance of any anchorage system should be substantiated through representative physical testing.

14.1.2 Concrete cover delamination—Concrete cover delamination(also referred to as FRPend peeling)can result from the normal stresses developed at the ends of externally bonded FRP reinforcement.With this type of delamination, the existing internal reinforcing steel provides a weak horizontal plane along which the concrete cover pulls away from the rest ofthe beam,as shown in Fig.14.1.2a.

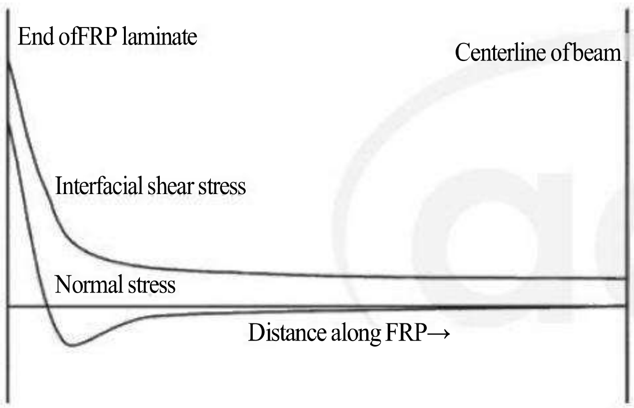


Fig.14.1—Conceptual interfacial shear and normal stress distributions along the length of a bonded FRP laminate (Roberts and Haji-Kazemi 1989;Malek et al.1998).

The tensile concrete cover splitting failure mode is controlled,in part,by stress at the termination point of the FRP.In general,the FRP end peeling failure mode can be mitigated by using anchorage(U-wraps,fiber anchors,or NSM anchors),by minimizing the stress at the FRP curtailment or termination by locating these as close to the region of zero moment as possible,or both.When the factored shear force at the termination point is greater than two-thirds of the concrete shear strength(V>0.67V.),the FRP laminates should be anchored with transverse reinforcement to prevent the concrete cover layer from splitting.The area of the transverse clamping FRPU-wrap reinforcement,Afanchor, can be determined in accordance with Eq.(14.1.2)(Reed et al.2005)

$$A_{fanchor} = \frac{(A_f f_{fe})_{longitudinal}}{(E_f \kappa_v \varepsilon_{fu})_{anchor}} \qquad (14.1.2)$$

in which k,is calculated using Eq.(11.4.1.2b).

Instead of detailed analysis,the following general guidelines for the location of cutoff points for the FRP laminate can be used to avoid end peeling failure mode:

(a)For simply supported beams,all FRPlaminates should be terminated at least a distance equal to lar past the point along the span at which the resisted moment falls below the cracking moment M.For multiple-ply laminates,successive plies should be terminated not less than 6 in.(150 mm)beyond the previous ply (Fig.14.1.2b).

(b)For continuous beams,all FRP laminates should be terminated at least a distance equal to lr or d/2 beyond the inflection point (point of zero moment resulting from factored loads).For multiple-ply laminates,successive plies should be terminated no less than 6 in.(150 mm)from the end of the previous ply (Fig.14.1.2b).These guidelines apply for positive and negative moment regions.

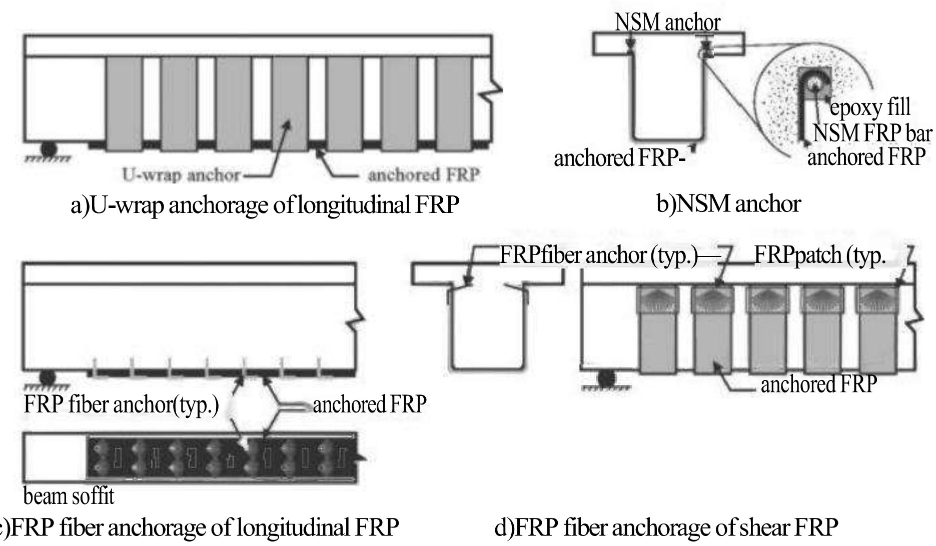


Fig.14.1.1—FRPanchorage systems

14.1.3 Development length—The bond capacity ofFRP is developed over a critical length lar.To develop the effective FRP stress at a section,the available anchorage length of FRP should exceed the value given by Eq.(14.1.3)(Teng et al.2003).

$$\ell_{df} = 0.057\sqrt{\frac{NE_f t_f}{\sqrt{f_c'}}} \quad \text{(in.-lb)}$$

$$\ell_{df} = \sqrt{\frac{NE_f t_f}{\sqrt{f_c'}}} \quad \text{(SI)} \tag{14.1.3}$$

14.1.4 Fiber anchors for anchoring U-wraps—As described in 11.4.1.2,fiber anchors located near both terminations of three-sided U-wraps may be used to anchor field-saturated FRP U-wraps(Kobayashi et al.2001;del Rey Castillo et al.2019a;Shekarchi et al.2020).Fiber anchors are typically installed into predrilled holes in the concrete substrate,with the top of the anchor splayed over the U-wrap. Fiber anchors should not be used to anchor preformed/ precured FRP plates.

The effectiveness of fiber anchors diminishes with increasing anchor cross-sectional area and increasing width of U-wrap anchored by an individual anchor (Sun et al. 2016;Pudleiner et al.2019;del Rey Castillo et al.2019b; Flores et al.2019).Fiber anchors may be used,in accordance with Table 14.1.4,to anchor one or more layers ofFRP having a total unit design stiffness,NEA,less than or equal to 1150 kip/in.(200 kN/mm)in the primary fiber direction of the U-wrap.The tensile material properties of the fiber anchor FRP material,fn,E,and eju,should be equal to or greater than those of the FRP material being anchored. The gross laminate area of the fiber anchor securing each leg of a U-wrap should not be less than the value given by Eq.(14.1.4).

$$A_{ame} \geq R_4(Ntsance) \tag{14.1.4}$$

where RA is the ratio of the anchor fiber area to the U-wrap fiber area and t is the thickness per ply of the laminate. Minimum values of R are given in Table 14.1.4.The anchor spacing,Sanc,as defined in Fig.14.1.4.For U-wraps that exceed NEty=1150 kip/in.(200kN/mm),the design of fiber anchors may be based on testing that demonstrates the effectiveness of the anchors.

Fiber anchors should be spaced uniformly along continuous or wide U-wraps(wy>10 in.[250 mm])at a spacing, Sane,not exceeding 10 in.(250 mm).For discrete U-wraps having a width w≤10in.(250mm),a single anchor centered on the U-wrap width can be used;in this case Sane=w for use with Eq.(14.1.4)and Table 14.1.4.

The fiber anchors can be installed immediately adjacent to the ends of the U-wrap or within the U-wrap no further than 2 in.(50 mm)from the ends of the U-wrap(refer to Fig.14.1.4).If the fiber anchor is installed through the ends of the U-wrap,d,,should be measured from the center of the anchor hole to center of the tensile reinforcement as shown in Fig.14.1.4.Fiber anchors should have an enclosed splay angle,αane,not exceeding 60 degrees.The fibers comprising the splay should be uniformly distributed in the splay and the splay should extend to the edges of discrete U-wraps or to adjacent splays as shown in Fig.14.1.4.

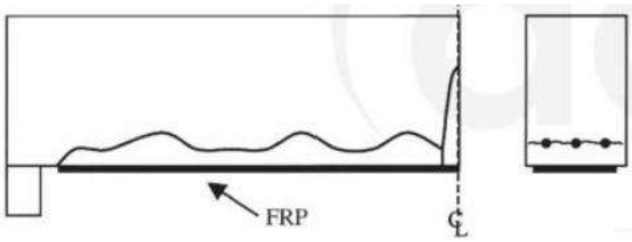


Fig.14.1.2a—Delamination caused by tensionfailure ofthe concrete cover:

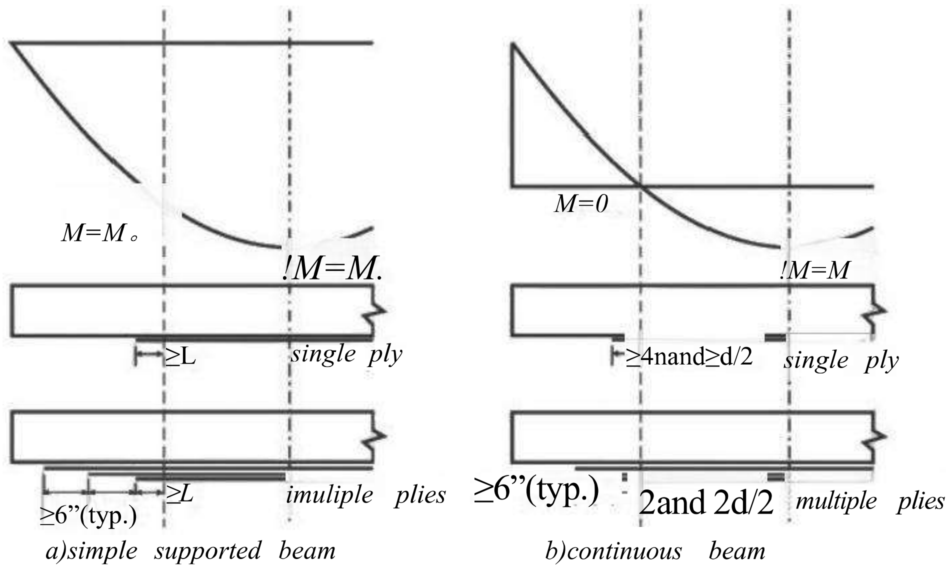


Fig.14.1.2b—Graphical representation ofthe guidelinesfor allowable terminationpoints ofa three-ply FRPlaminate.

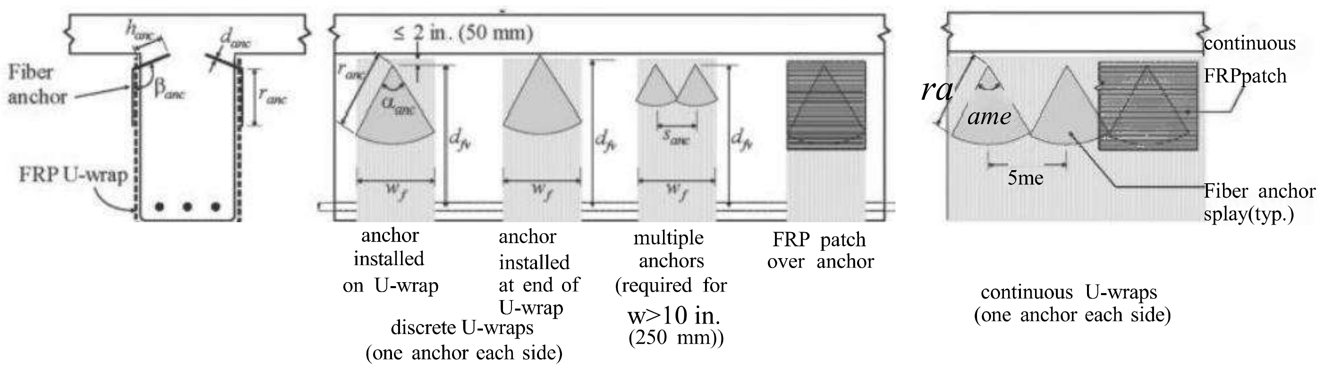


Fig.14.1.4—Details offiber anchorsfor U-wraps.

Table 14.1.4(in.-lb)—Recommendations for fiber anchors for anchoring shear U-wraps

Sane' in.	U-wrap NEt,kip/in.	RA		Fame, in.	hame, in.		
		90 degrees≤ β ane ≤125 degrees	125 degrees< β ane ≤180 degrees		90 degrees≤Pame ≤110 degrees	110 degrees<Pame ≤125 degrees	125 degrees< β ame ≤180 degrees
≤4	NEt≤288 288<NEIy≤575 575<NEty≤863 863<NEd≤1150	1.25 1.25 1.25 1.25	1.00 1.00 100 1.00	Sane San 8.0 10.0	Larger of 4.0 and 74mm25	Larger of 6.0 and 74am25	6.0 6.0 6.0 8.0
4<8San≤6	NEty≤288 288<NEty≤575 575<NE;A;≤863 863<NEA≤1150	1.25 1.25 1.50 1.50	1.00 1.00 1.25 125	Sane Sae 8.0 10.0			6.0 6.0 8.0 10.0
6<8am≤8	NE,ty≤288 288<NEt≤575 575<NEIy≤863 863<NEA≤1150	1.25 1.50 1.50 1.75	1.00 1.25 1.25 1.50	Sune Sune San 10.0			6.0 8.0 10.0 12.0
8<8m≤10	NE,ty≤288 288<NE;ty≤575 575<NEA≤863 863<NEt≤1150	125 150 1.75 2.00	100 1.25 1.50 2.00	Sane Sa San Same			6.0 8.0 10.0 12.0

For discrete U-wraps having width,w≤10 in.anchored with a single anchor,use sSam=F

Increase ra 50%ifno patch is provided.

Embedment should extend at least 2 in.beyond concrete cover.

The length of the splay,rane,should be equal to or exceed that given in Table 14.1.4.The anchor splay should be placed over the U-wrap.A single-layer FRP patch of the same material as that being anchored should be placed over the entire anchor splay with fibers oriented perpendicular to the anchored FRP.In place of providing a perpendicular FRP patch,rane in Table 14.1.4 may be increased by 50%. Although not necessary to achieve the objectives of this section,an additional FRP patch with fibers parallel to the anchored FRP has been shown to further enhance the effectiveness of the anchors(Pudleiner et al.2019).

The angle at which the anchor is embedded,βane,can be between 90 and 180 degrees (Fig.14.1.4).The minimum embedment depth of the anchor,hane,for different installation angles,βanc,is given in Table 14.1.4.Anchors should not be installed such that βane<90 degrees.

Fiber anchors are fabricated to be nominally round to be inserted into drilled holes.The diameter of the drilled hole should be between 0.125 in.(+3 mm)and 0.375 in. (+10 mm)larger than the resulting anchor diameter,dane The depth of the drilled hole should be at least 0.25 in. (6 mm)deeper than hanc.Holes should be located to avoid damaging existing internal reinforcement.To mitigate stress concentrations in the anchor fibers,the edge of the drilled hole should be rounded or chamfered to a distance of one half the hole diameter,but not less than 0.5 in. (13 mm)and should extend over the splay angle,αane(refer to Fig.14.1.4).Anchor installation,including drilling and cleaning of holes,should be in accordance with manufacturer's recommendations.

14.2—Detailing of laps and splices

Splices of FRP laminates should be provided only as permitted in drawings,specifications,or as authorized by the licensed design professional or as recommended by the system manufacturer.

The fibers of FRP systems should be continuous and oriented in the direction of the greatest tensile forces.