

Seismic Retrofitting using Externally Bonded Fibre Reinforced Polymers (FRP)

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1 Introduction

Externally bonded (eb) fibre reinforced polymers (FRPs) in the form of continuous carbon (C), glass (G) or aramid (A) fibres bonded together in a matrix made of epoxy, vinylester or polyester, are being employed extensively throughout the world in retrofitting reinforced concrete structures. Early applications have been mainly for strengthening against non-seismic actions. Nonetheless, their high strength-to-weight ratio, immunity to corrosion and easy handling and installation are making FRP jackets the material of choice in an increasingly large number of seismic retrofitting projects, despite the relatively high material costs.

The literature on FRP-strengthened RC elements is vast: several journal or conference papers cover a variety of aspects on seismic retrofitting, the most important ones being shear strengthening and increase of confinement. The basic concepts involved in using FRPs as strengthening materials of concrete structures are covered in a review article of Triantafillou (1998a). Progress in various strengthening methods, questions associated with the long-term durability of FRP, as well as the development of design guidelines and codes for non-seismic applications were addressed in a review paper by Neale (2000). Presently the most comprehensive and up-to-date overview – albeit without emphasis on seismic retrofitting – is the one in (*fib*, 2001). A recent survey of the literature on seismic retrofitting with FRPs may be found in a review article by Triantafillou (2001).

Given that continuity and anchorage of FRPs in a joint beyond a member end is difficult to achieve, the main uses of FRPs in seismic retrofitting of existing RC elements are the following:

- The shear capacity of sub-standard elements (columns, shear walls, etc.) can be enhanced, by providing externally bonded FRPs with the fibres in the hoop direction.
- A ductile behaviour of flexural plastic hinges at beam or column ends can be achieved through added confinement in the form of FRP jackets, with the fibres placed along the beam or column perimeter.
- The flexural strength of RC columns can only be developed when debonding of the reinforcement in lap splices is prevented. Such debonding occurs once vertical cracks develop in the cover concrete and progresses with increased dilation and cover spalling. The associated rapid flexural strength degradation can be prevented or limited with increased lap confinement, again with fibres along the column perimeter.

This section focuses on the seismic retrofitting of RC members (mainly columns, but also beams, shear walls and beam-column joints) using FRPs. Following a brief review of key properties of FRP materials, some basic retrofitting issues (shear strengthening, increase of confinement) are summarised and a brief summary of application techniques is given. Next, more information on the behaviour and design of shear-strengthened or confined (with FRP) members is provided. Finally, a brief description of some recent developments related to seismic strengthening of beam-column joints is presented.

A presentation of the entire body of research and design issues in this field is not feasible within the scope of this section. The intent rather is to provide a representative overview, to underline the basis of design methods and to provide an extensive, but not exhaustive, bibliography on the subject.

2 Materials and their application

The combination of unidirectional fibres, which constitute the primary load-carrying elements,

with a polymeric matrix results in a material that behaves as linear elastic to failure, without a yield plateau. Hence, a fairly complete characterization of this material is obtained by defining the elastic modulus and the tensile strength in the direction of the fibres (Fig. 1). Both parameters can be estimated by the so-called “rule of mixtures”, which, for typical FRP materials, can be simplified as:

$$P_f \approx P_{fib} V_{fib} \tag{1}$$

where P_f = property of FRP (elastic modulus E_f or tensile strength f_{ft}), P_{fib} = corresponding property of fibres and V_{fib} = volume fraction of fibres in the FRP (in the order of 40-65%). A summary of basic mechanical properties for various types of fibres is given in Table 1 and, in a more schematic form, in Fig. 2 (fib 2001).

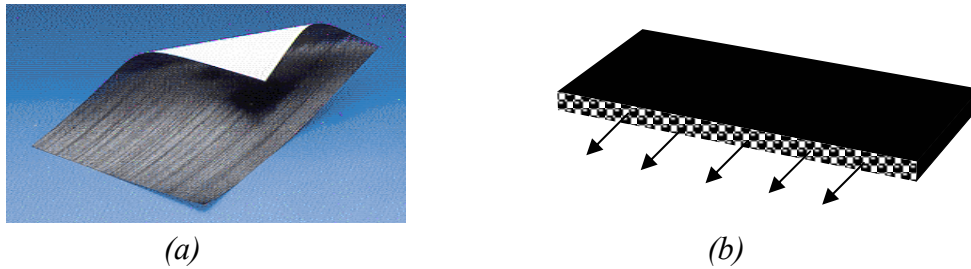


Fig. 1: (a) FRP sheet made of unidirectional fibres. (b) Stresses carried by the fibres.

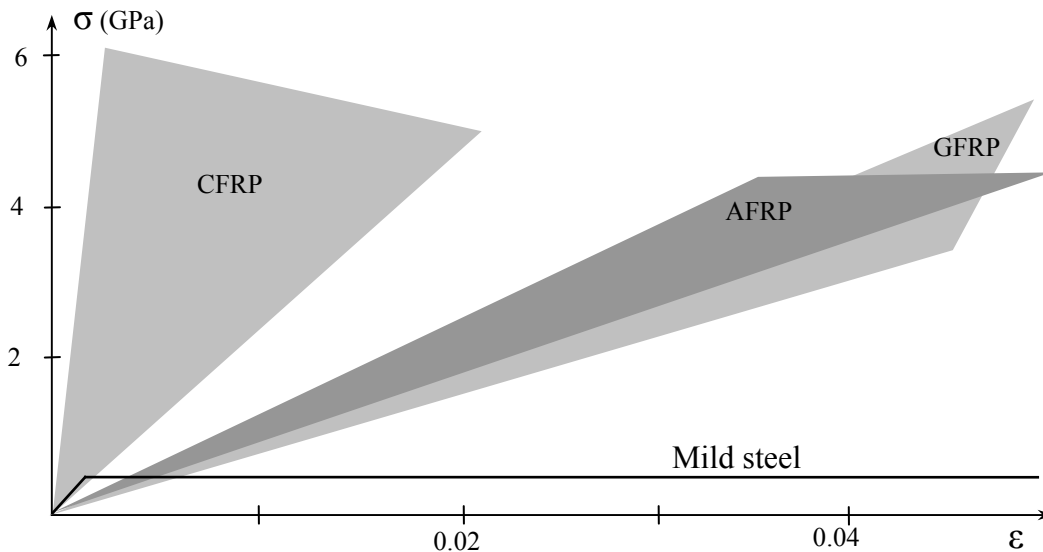


Fig. 2: Uniaxial tension stress-strain diagrams for different unidirectional FRPs and steel. CFRP = carbon FRP, AFRP = aramid FRP, GFRP = glass FRP (fib 2001).

Material	Elastic modulus (GPa)	Tensile strength (MPa)	Ultimate tensile strain (%)
Carbon			
High strength	215-235	3500-4800	1.4-2.0
Ultra high strength	215-235	3500-6000	1.5-2.3
High modulus	350-500	2500-3100	0.5-0.9
Ultra high modulus	500-700	2100-2400	0.2-0.4
Glass			
E	70	1900-3000	3.0-4.5
S	85-90	3500-4800	4.5-5.5
Aramid			
Low modulus	70-80	3500-4100	4.3-5.0
High modulus	115-130	3500-4000	2.5-3.5

Table 1: Typical properties of fibres (fib 2001).

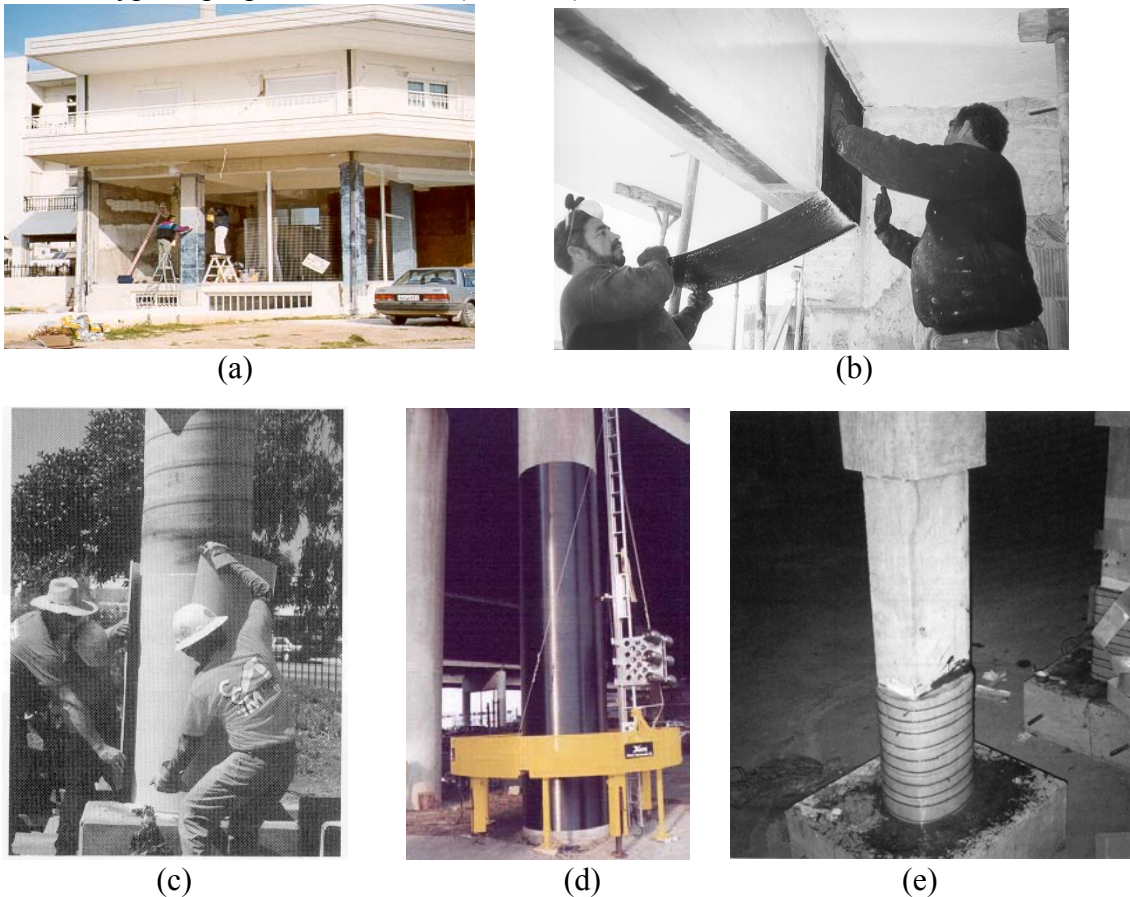


Fig. 3: Retrofitting with: (a) and (b) FRP sheets; (c) prefabricated shells; (d) automated wrapping; (e) prestressed FRP jackets through the use of expansive mortar.

FRP jackets may be applied either as *wet lay-up* systems (Fig. 3a,b) or as *prefabricated* systems. Wet lay-up systems include: a) dry sheets or fabrics impregnated with resin in-situ, b) pre-impregnated sheets or fabrics or tows (untwisted bundles of continuous fibres) installed with or without additional resin and c) dry tows impregnated with resin during winding. Prefabricated systems are typically factory-made curved or shaped elements, or split shells that can be fitted around columns (Fig. 3c).

Techniques for seismic retrofitting with FRP comprise manual application of the aforementioned systems or automated wrapping of fibre tows (Fig. 3d), which may be either unstressed or prestressed during application. Another option to achieve prestressing is by filling prefabricated FRP jackets placed around RC members with expansive mortar or concrete (e.g. Saadatmanesh et al. 1997, Shimbo et al. 1997), see Fig. 3e.

3 Shear strengthening of RC components

(1) Beams, columns and walls

Shear capacity, a strength issue, can be added by FRP jackets to rectangular or circular RC columns or shear walls, in a way similar to adding shear strength through internal stirrup reinforcement.

A number of experimental studies have been carried out to investigate the shear strengthening of flexural members with FRP jackets. In these studies, various researchers have shown that the shear strength of RC members can be increased as the thickness of the jacket increases (e.g. Priestley and Seible 1995, Fujisaki et al. 1997, Masukawa et al. 1997, Sirbu et al. 1998, Matsuzaki et al. 2000). Typical lateral force-displacement curves for an as-built shear-critical and a companion retrofitted

column are compared in Fig. 4 (Priestley and Seible, 1995), which demonstrates that brittle shear failure is suppressed and stable hysteretic behaviour may lead to high ductility levels.

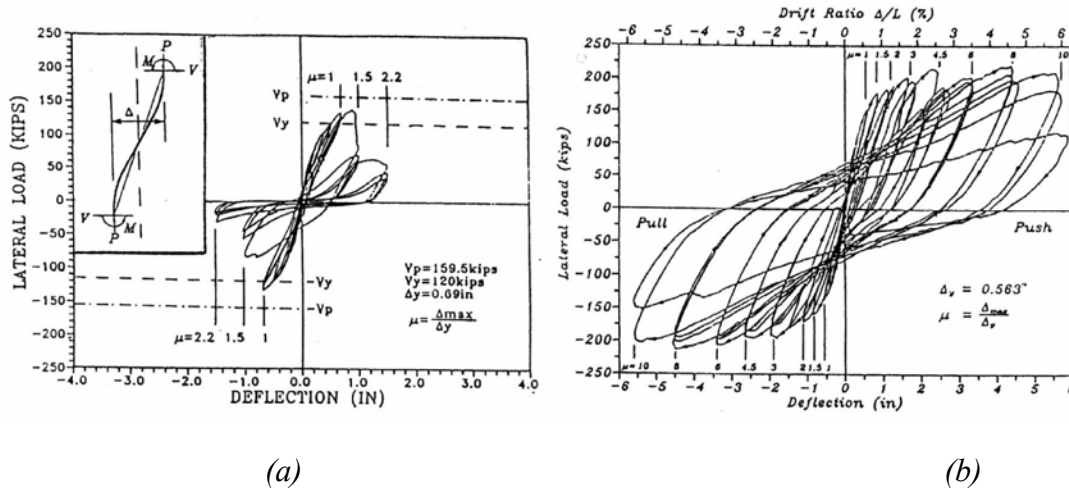


Fig. 4: Lateral force-displacement response of shear-critical rectangular columns: (a) as-built; (b) retrofitted with GFRP jacket (Priestley and Seible 1995).

Detailed investigations on shear strengthening of RC members have been relatively limited and, to a certain degree, controversial. Quite a few researchers have idealised the FRP materials in analogy with internal steel stirrups, assuming that the contribution of FRP to shear capacity (V_{fd} , in analogy to V_{wd}) emanates from the capacity of fibres to carry tensile stresses at a more or less constant strain, which is equal either to the FRP ultimate tensile strain, ϵ_{fu} , or to a reduced value. Priestley and Seible (1995) suggested that the FRP contribution to the shear capacity of columns be calculated by assuming an FRP strain equal to 0.004; Japanese researchers (e.g. JCI 1998) proposed a fixed fraction of ϵ_{fu} , which varied from 20% to 60% of ϵ_{fu} for AFRP (aramid FRP) and from 2/3 to 100% for CFRP (carbon FRP).

In more recent studies, Triantafillou (1998b) and Triantafillou and Antonopoulos (2000) introduced the concept of the effective strain, $\epsilon_{f,e}$, defined as the strain in the FRP when the strengthened member reaches its shear capacity, and demonstrated that this strain depends on a number of factors, including: (a) the FRP failure mode, which may be characterized by tensile fracture or by debonding, depending on the type of jacket (closed jackets and low elongation capacity fibres are more likely to fracture than to debond, compared with partially wrapped open jackets and high strain capacity fibres); (b) the axial rigidity of the jacket along the perimetre, $E_f t_f$, where E_f = elastic modulus of FRP and t_f = thickness of jacket ($\epsilon_{f,e}$ decreases as $E_f t_f$ increases); and (c) the strength of concrete ($\epsilon_{f,e}$ increases with the tensile strength of concrete).

In design format, the following equations may be adopted:

$$V_{fd} = 0.9 \epsilon_{fd,e} E_f \rho_f b_w d (\cot \theta + \cot \alpha) \sin \alpha \quad (2)$$

where: $\epsilon_{fd,e}$ = design value of effective FRP strain; b_w = minimum width of cross section over the effective depth; d = effective depth of cross section; ρ_f = FRP reinforcement ratio equal to $2t_f \sin \alpha / b_w$ for continuously bonded shear reinforcement of thickness t_f (b_w = minimum width of the concrete cross section over the effective depth), or to $(2t_f / b_w)(b_f / s_f)$ for FRP reinforcement in the form of strips or sheets of width b_f at a spacing s_f (Fig. 5); E_f = elastic modulus of FRP in the principal fibre orientation; θ = angle of diagonal crack with respect to the member axis; and α = angle between principal fibre orientation and longitudinal axis of member (Fig. 5).

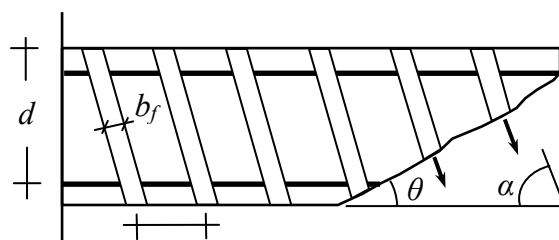


Fig. 5: Orientation of FRP strips and definition of dimensions.

The design value of the effective FRP strain, $\varepsilon_{fd,e}$, may be obtained in terms of the characteristic value, $\varepsilon_{fk,e}$, which is related to the mean value $\varepsilon_{f,e}$ (see *fib*, 2001, for details). A synthesis and evaluation of the published experimental results on shear strengthening of RC members with FRP sheets or fabrics reported in the literature up to early 1999 resulted in the best fit power-type expressions given next (Triantafillou and Antonopoulos 2000). Adoption of these equations in cases other than the ones given below (e.g. for GFRP jackets) should be done with caution.

- Fully wrapped (i.e. closed) or properly anchored (in the compression zone) CFRP jackets (fracture of the jacket is dominant):

$$\varepsilon_{f,e} = 0.17 \left(\frac{f_{cm}^{2/3}}{E_f \rho_f} \right)^{0.30} \varepsilon_{fu} \quad (3a)$$

- Side or U-shaped (i.e. open) CFRP jackets (fracture of the jacket may be preceded by debonding):

$$\varepsilon_{f,e} = \min \left[0.65 \times 10^{-3} \times \left(\frac{f_{cm}^{2/3}}{E_f \rho_f} \right)^{0.56}, 0.17 \left(\frac{f_{cm}^{2/3}}{E_f \rho_f} \right)^{0.30} \varepsilon_{fu} \right] \quad (3b)$$

- Fully wrapped (i.e. closed) or properly anchored AFRP jackets (fracture of the jacket is dominant):

$$\varepsilon_{f,e} = 0.048 \left(\frac{f_{cm}^{2/3}}{E_f \rho_f} \right)^{0.47} \varepsilon_{fu} \quad (3c)$$

In the above equations f_{cm} is in MPa and E_f is in GPa. Furthermore, $\varepsilon_{f,e}$ should be limited to a maximum value (ε_{max}) in the order of 0.006, to ensure that the shear integrity of concrete is maintained sufficiently, so that other mechanisms, such as aggregate interlock, may be activated too.

The above concept of non-constant effective FRP strain (but decreasing with increasing FRP stiffness or decreasing concrete strength) was adopted recently in design guidelines by both ACI (ACI-440 2001) and JSCE (2001), with slightly different formulation.

According to a less empirical approach, the design value of the effective FRP strain, $\varepsilon_{fd,e}$, can be related to the maximum FRP strain (design value) at the ultimate limit state in shear, $\varepsilon_{fd,max}$. Assuming that the FRP jacket is properly anchored (e.g. closed jackets) and a linearly increasing opening of the shear crack, this relationship is $\varepsilon_{fd,e} \approx 0.5\varepsilon_{fd,max}$, with $\varepsilon_{fd,max} = f_{jd}/E_f$, where f_{jd} = design tensile strength of FRP jacket and E_f = elastic modulus of jacket. Note that f_{jd} is, in general, less than the uniaxial tensile strength f_{fd} . This reduction is attributed to several reasons, including: (a) the multiaxial state of stress in the FRP (e.g. at corners of low radius); and (b) the quality of execution (potential local ineffectiveness of some fibres due to misalignment and overstressing of others; damaged fibres at sharp corners or local protrusions etc). In the case of open jackets (e.g. U-shaped) the tensile capacity of the FRP is typically not exhausted, due to debonding; this can be described by an appropriate bond stress-slip model. For instance, an appropriate modification of the model by Neubauer and Rostásy (1997) results in $\varepsilon_{fd,max}$ being approximately equal to $0.40(f_{cm}/E_f t_f)^{1/2}$, where f_{cm} = mean tensile strength of concrete in MPa, E_f in MPa and t_f in mm. The value of 0.006 may still be assumed as a limiting one for $\varepsilon_{fd,max}$, to ensure that the shear integrity of concrete (e.g. aggregate interlock) is maintained sufficiently.

The material above refers mainly to RC members of rectangular (or nearly rectangular) cross sections, which are mainly of interest for buildings. If the cross section is circular (as is usually the case in bridge piers), the contribution of FRP to shear capacity is controlled by the tensile strength of the FRP jacket, but is limited to a maximum value corresponding to excessive dilation of the concrete due to aggregate interlock at inclined cracks. By limiting concrete dilation, that is the radial strain (which is equal to the FRP hoop strain) to a maximum value, say, ε_{max} , one may show that for inclined cracks forming an angle θ with the column axis, the FRP contribution to shear capacity is as given below (assuming fibres perpendicular to the member axis):

$$V_{fd} = 0.5\varepsilon_{fd,e}E_f\rho_fA_c \cot\theta$$

(4)

where A_c = column cross-sectional area and ρ_f = volumetric ratio of FRP. Eq. (5-33) assumes that at shear failure all the FRP material crossing an inclined crack is strained uniformly at $\varepsilon_{fd,e}$. Experimental evidence suggests that $\varepsilon_{fd,e}$ is in the order of 0.004 (Priestley and Seible, 1995).

Testing of shear walls strengthened with FRP jackets has been very limited. A recent study (Lombard et al. 2000) focused on flexural strengthening of RC walls and hence shear failure is not relevant, whereas some researchers have tested RC columns with wing walls strengthened in shear (Masuo 1999, Iso et al. 2000). From the limited data available in the literature it may be deduced that FRP jackets perform more or less the same as in beams or columns: they act as horizontal reinforcement with a strain that decreases as the thickness and/or the Elastic Modulus (along the perimeter) of the jacket increases.

(2) Beam-column joints

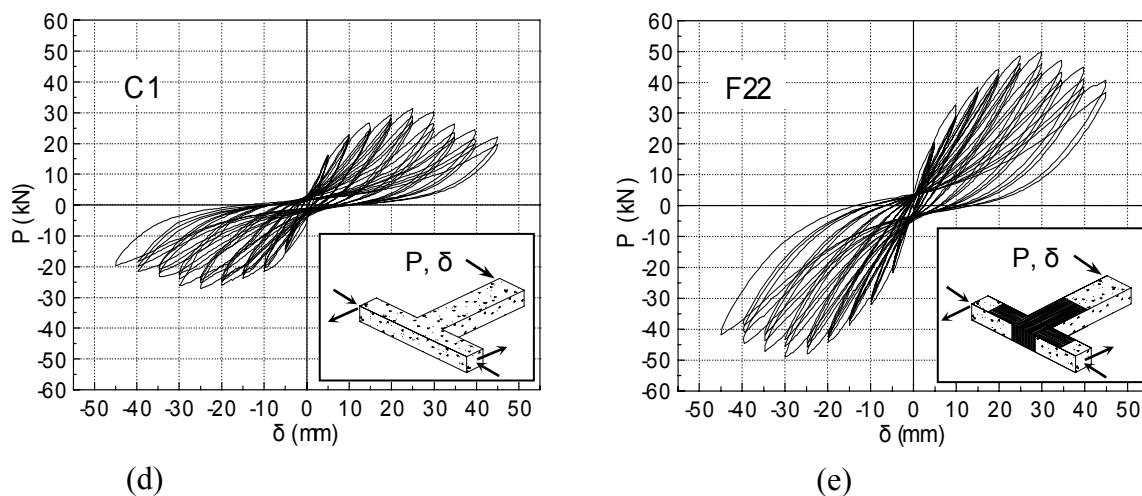
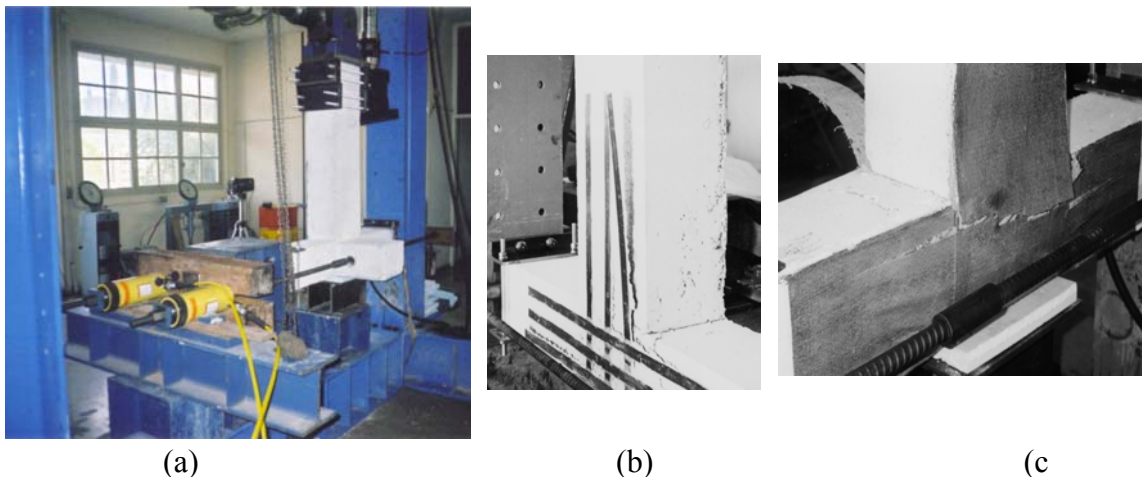


Fig. 6: Strengthening of beam-column joints: (a) Testing; (b) Poor performance (debonding) of rigid CFRP strips; (c) Satisfactory performance of flexible CFRP sheets (strain capacity of FRP achieved fully); (d)

Lateral force-displacement response of control joint; and (e) Response of joint strengthened with CFRP sheets (two layers in the vertical and two in the horizontal direction).

Shear strengthening of RC joints is a challenging task, which poses major practical difficulties. As an alternative to conventional RC or steel jackets, which require considerable labour and artful detailing, FRP materials have been used successfully as strengthening materials of exterior beam-column joints with deficiency in shear strength (Pantelides et al. 1999, Gergely et al. 2000). In a recent study, Antonopoulos and Triantafillou (2003) conducted a comprehensive experimental programme, which comprised the investigation of several design parameters through 2/3-scale testing of 18 exterior joints. Parameters under consideration included: thickness of FRP, distribution of FRP between the beam and the column, column axial load, internal joint (steel) reinforcement, initial damage, carbon versus glass fibres, sheets versus strips, effect of transverse beams. A typical result is shown in Fig. 6, which illustrates that even very thin jackets (e.g. two 0.12 mm thick CFRP sheets in the beam direction and two in the column direction) may substantially increase the shear capacity. This experimental study was followed by development of an analytical model (Antonopoulos and Triantafillou 2002) for the shear capacity of FRP-strengthened joints (through stress and strain analysis), which was found in excellent agreement with test results. Concepts for the use of FRP in shear strengthening of joints are illustrated in Fig. 7.

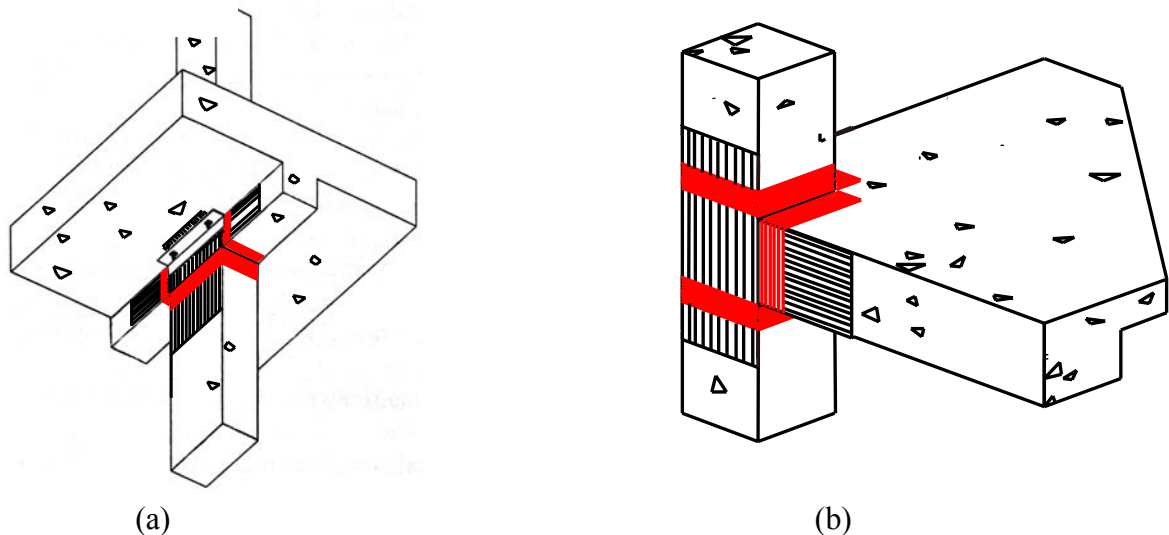
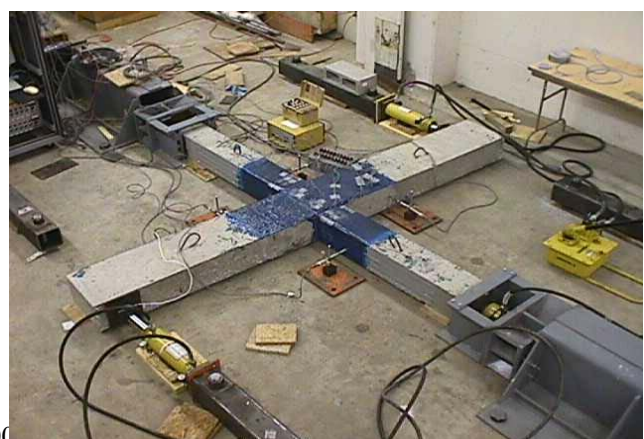


Fig. 7: Concepts for shear strengthening of (a) interior and (b) exterior beam-column joints with FRP sheets (Antonopoulos and Triantafillou 2003).

Another application of FRP related to combined strengthening of joints, as well as of critical regions of beams and/or columns is illustrated in Fig. 8. This concept has been studied in the experimental work of Castellani et al. (1999) and Prota et al. (2001), who demonstrated that an increased resistance to bar slippage within the joint, as well as increased shear resistance in the connection region is possible.



(a)

(b)

Fig. 8: (a) View of local strengthening with CFRP of a full scale dual frame-wall structure tested, repaired and re-tested at JRC (Castellani et al. 1999). (b) Testing of CFRP-strengthened beam-column connection (Prota et al. 2001).

4 Increase of member deformation capacity

(1) Introduction

Enhancement of deformation capacity of structurally deficient columns in seismic regions is of outmost importance. This enhancement is best achieved through concrete confinement. In this section the behaviour of concrete confined with FRP jackets is discussed. Then key design issues related to improved plastic hinging, lap splice clamping and prevention of rebar buckling are highlighted.

(2) Behaviour of FRP-confined concrete

Concrete is a restraint-sensitive material, which means that its response under axial loading depends heavily on the lateral restraint provided through confinement (e.g. Pantazopoulou 1995). Let us examine a cylindrical concrete column with diameter D , wrapped with an FRP jacket having thickness t_f and Modulus of Elasticity E_f in the circumferential direction, which typically coincides with the principal fibre direction. Axial loading will cause shortening of the column and lateral expansion, which, in turn, will cause the FRP jacket to extend in the circumferential direction and develop confinement pressure onto the concrete. Assuming that the stress in the FRP is σ_f , the confinement stress, σ_r , exerted on the concrete in the radial direction equals:

$$\sigma_r = \frac{2t_f}{D} \sigma_f = \frac{2t_f}{D} E_f \varepsilon_f$$

(5)

where ε_f is the circumferential FRP strain. The confinement stress reduces the shear stress components of the triaxial stress tensor in the concrete and suppresses microcracking and crack initiation/propagation.

One approach to examine the behaviour of FRP-confined concrete is to look at its volumetric response, shown in Fig. 9 (e.g. *fib* 2001). The volumetric strain in this figure is equal to the sum of the three principal strains, tensile strains and dilation considered negative. It is interesting to observe from the volume strain versus axial strain curve that for the CFRP jacket the volumetric strain first decreases, as expected, then reverts to zero and beyond a certain level of axial strain the ever-increasing confinement pressure curtails the volumetric expansion and inverts its direction. The position of the point where the confinement action starts becoming effective (that is, when the branches depart from the unconfined one) depends on the stiffness of the confining device: the GFRP-confined concrete (jacket of equal thickness) departs later than the other two. This is the point where sufficient lateral pressure develops to prevent the lateral dilation of concrete from increasing unrestrained (this is not the case with steel confinement jackets, where yielding is associated with unstable volumetric expansion).

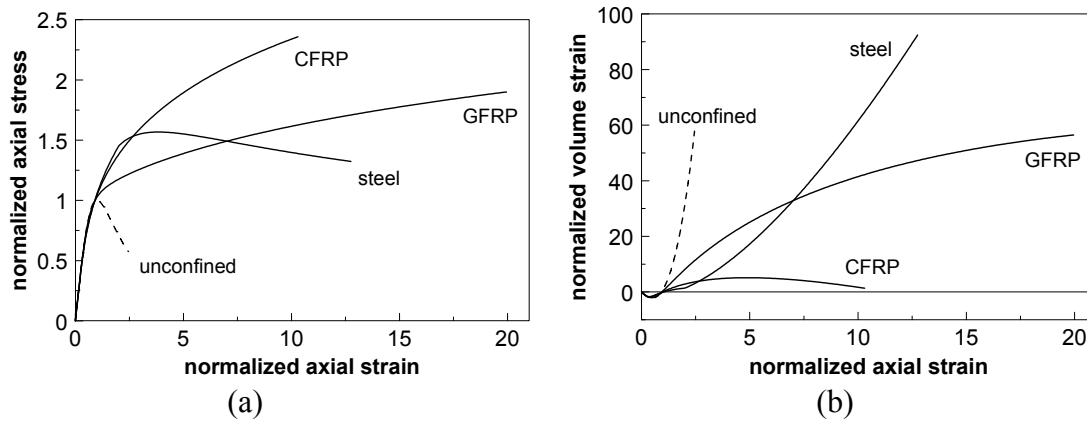


Fig. 9: (a) Stress-strain and (b) volumetric response of FRP-confined concrete.

The stress-strain response of FRP-confined concrete is illustrated in Fig. 10. The figure displays a nearly bilinear response with a sharp softening and a transition zone at a stress level near the strength of unconfined concrete. After this stress the tangent stiffness changes a little, until the concrete reaches its ultimate confined strength f_{cc} when the jacket reaches tensile failure at a stress $\sigma_f = f_j$ (and a corresponding strain ϵ_{ju}) which is, in general, less than the uniaxial tensile strength f_j . This reduction is attributed to the triaxial state of stress in the FRP (due to axial loading and confining action, but also due to bending, e.g. at corners of low radius) and to the quality of execution.

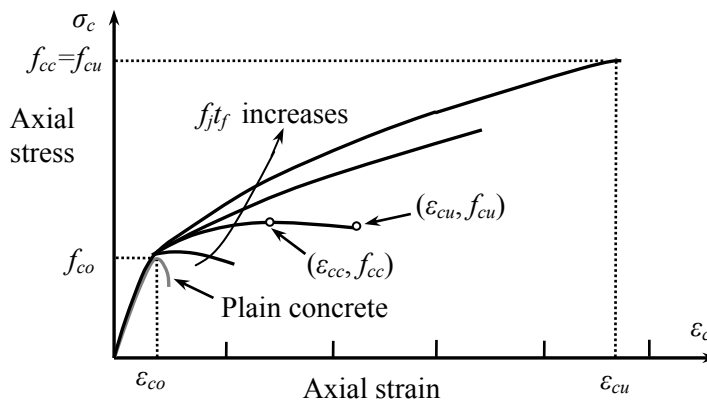


Fig. 10: Axial stress-strain response of FRP-confined concrete versus plain concrete.

If confinement is provided by applying the FRP through automated or manual winding with the fibres in a prestressed state, an additional confining stress, given by Eq. (5) with σ_f replaced by the prestress level, must be added to that caused by column shortening.

Analytical and experimental studies of the stress-strain response of FRP-confined concrete have been conducted by several researchers (e.g. Fardis and Khalili 1981, Priestley and Seible 1995, Hosotani et al. 1997, Karbhari and Gao 1997, Miyauchi et al. 1997, Watanabe et al. 1997, Harmon et al. 1998, Samaan et al. 1998, La Tegola and Manni 1999, Saafi et al. 1999, Spoelstra and Monti 1999, Wang et al. 2000, Xiao and Wu 2000, Lam and Teng 2001). Most of the FRP-confinement models give the stress at ultimate strain f_{cu} and the associated strain ϵ_{cu} as functions of f_{co} and ϵ_{co} , respectively, the confining (lateral) stress at ultimate, $\sigma_l (= 2t_f E_f \epsilon_{ju} / D = 2t_f f_j / D)$ and the jacket ultimate strain ϵ_{ju} . The effectiveness of confinement depends heavily on the jacket characteristics and increases as the stiffness and the ultimate strain increase.

The confinement model by Spoelstra and Monti (1999), combining simplicity with accuracy, is:

$$f_{cu} = f_{co} \left(0.2 + 3 \sqrt{\frac{2t_f f_j}{D f_{co}}} \right) \quad (6a)$$

$$\varepsilon_{cu} = \varepsilon_{co} \left(2 + 1.25 \frac{E_c}{f_{co}} \varepsilon_{ju} \sqrt{\frac{2t_f f_j}{Df_{co}}} \right) \quad (6b)$$

If the cross-section of the column is rectangular, confinement is less effective. Confining stress is transmitted then to the concrete through the four corners of the cross section and increases with the corner radius R (Fig. 11). The confinement effectiveness factor, α_f , is just the ratio of the confined (shaded) area of the section to the total in Fig. 11.

$$\alpha_f = \left(1 - \frac{(h-2R)^2 + (b-2R)^2}{3bh} \right) \quad (7)$$

(b, h = external dimensions of concrete section). For a more detailed treatment of the application of FRP confinement models to concrete, the interested reader is referred to *fib* (2001).

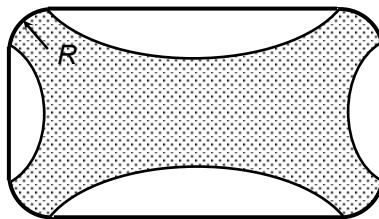


Fig. 11: Effectively confined area (defined by parabolas) in FRP-confined rectangular column.

(3) Improved plastic hinge behaviour

Plastic hinge confinement is crucial, as the unconfined compression strength of concrete is insufficient to enable the development of large displacement or chord rotation ductility factors μ_θ . Experimental results (e.g. Kobatake et al. 1993, Saadatmanesh et al. 1996, Masukawa et al. 1997, Seible et al. 1997, Sexsmith et al. 1997, Feng and Bahng 1999, Fujii et al. 1999, Fukuyama et al. 1999, Matsuzaki et al. 1999, Seible et al. 1999, Zhang et al. 1999, Saiidi et al. 2000, Sheikh et al. 2000) have demonstrated that enhancement of the ductility capacity is easily achieved by properly designed FRP jackets (Fig. 12).

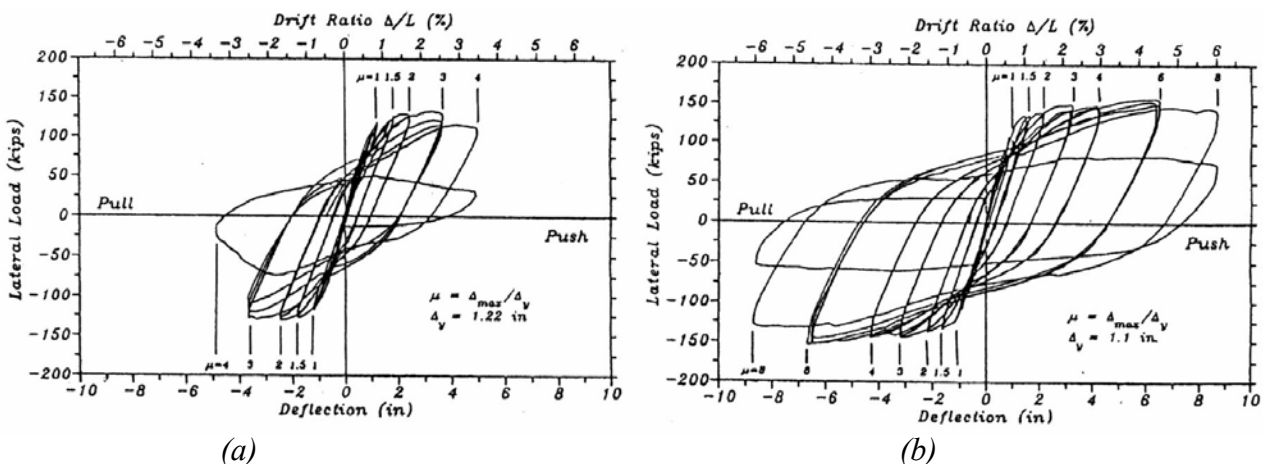


Fig. 12: Lateral force-displacement response of flexure-dominated rectangular columns: (a) as-built; (b) retrofitted with GFRP jacket at the plastic hinge region (Priestley and Seible 1995).

The method of selecting the jacket thickness for a specified ductility factor μ_θ is a relatively straightforward procedure: First the equivalent plastic hinge length L_p for a given column is calculated based on the yield stress and diameter of longitudinal rebars. From L_p and μ_θ the curvature ductility factor $\mu_\phi = \Phi_u/\Phi_y$ is established. The yield curvature Φ_y may be found from moment-curvature analysis of the cross section, whereas the maximum required curvature Φ_u may be obtained (again from section analysis) in terms of the ultimate concrete strain. Hence the required value for ε_{cu} can be established and an appropriate confinement model can be used to solve for the required FRP thickness.

Japanese researchers have followed a different approach towards assessing the displacement ductility factor of FRP-confined columns. According to this approach, the ductility factor may be related to the shear capacity, V_u , and the shear force at the flexural strength, $V_{fu}=M_u/L_s$ of the member after retrofit according to empirical equations of the type:

$$\mu_\theta = \alpha + \beta \frac{V_u}{V_{fu}} \quad (8)$$

where the constants α, β depend on the type (that is the deformability) of the fibres (Mutsuyoshi et al. 1999).

Another method for relating ductility to the FRP material characteristics was proposed by Monti et al. (2001). Key element in this method is the definition of the *upgrading index I*:

$$I = \frac{r_{tar} \mu_{\theta,tar}}{r_o \mu_{\theta,o}} \quad (9)$$

where r_{tar} = resistance (strength) of desired upgrading (target), r_o = initial resistance (before retrofiting), $\mu_{\theta,tar}$ = target ductility and $\mu_{\theta,o}$ = initial ductility. The upgrading index is related to a number of characteristics of the member before retrofiting (longitudinal reinforcement, axial load, concrete strength), as well as to the strength and ultimate strain of confined concrete. The last two properties (f_{cc}, ε_{cu}) may be related to the FRP jacket thickness and deformation capacity through an appropriate confinement model (e.g. Eqs. 6), leading to the optimum jacket design for a given value of I .

In a recent study, Tastani and Pantazopoulou (2002) proposed an empirical (based on data fit) simple lower-bound expression relating the available displacement or chord-rotation ductility factor to confining pressure σ_l provided by FRP:

$$\mu_\theta = 1.3 + 12.4 \left(\frac{\sigma_l}{f_c} - 0.1 \right) \geq 1.3 \quad (10)$$

For rectangular columns subjected to flexure in the axis parallel to the side of width b_w , the lateral confining stress equals $a_w \rho_w f_{yw} + 2a_f t f_j / b_w$, where a_w in the first term is the effectiveness coefficient of stirrups (with a volumetric ratio ρ_w and yield stress f_{yw}). Hence, for a given jacket material, Eq. (10) may be used to estimate the FRP thickness required to achieve the displacement or chord-rotation ductility factor μ_θ .

(4) Lap-splice clamping

A number of experimental studies (Ma and Xiao 1997, Saadatmanesh et al. 1997, Seible et al. 1997, Restrepo et al. 1998, Osada et al. 1999, Chang et al. 2001, Haroun et al. 2001, Saatcioglu and Elnabesy 2001) have demonstrated that FRP jackets are quite effective in preventing lap-splice failures (e.g. Fig. 13). Whilst this effectiveness is maximum for circular columns, it is reduced considerably for rectangular ones, by a factor in the order of 50% (Seible et al. 1997, Saatcioglu and Elnabesy 2001). Lap-splice clamping requires sufficient lateral pressure onto the splice region to prevent the concrete prisms that adhere to the lapped reinforcement to slip relative to each other. Tests have shown that the onset of lap-splice relative slippage starts when the dilation strains are between 0.001-0.002. Adopting the lower limit of dilation strain, that is 0.001, a rough estimate of the FRP jacket thickness as a function of the lateral stress required to keep the lap splice from debonding (through frictional restraint) in circular columns may be obtained as (Seible et al. 1997):

$$t_f = \frac{D(\sigma_l - \sigma_h)}{2E_j \times 0.001} \quad (11)$$

where σ_l represents the clamping stress over the lap splice length L_s ,

$$\sigma_l = \frac{A_s f_{sy}}{\left[\frac{p}{2n} + 2(d_b + c) \right] L_s} \quad (12)$$

and σ_h is the hoop stress in the stirrups at a strain of 0.001, which can be ignored, or the active pressure from the grouting between the FRP and the column, if provided. In Eq. (12) p = perimeter line in the column cross section along the inside of longitudinal reinforcement, n = number of spliced bars along p , A_s and f_{sy} = area and yield stress, respectively, of longitudinal reinforcement, c = concrete cover to the longitudinal rebars and d_b = bar diameter.

One possible extension of the above formulation to rectangular columns (which are of interest for buildings) could be made here by replacing D by b_w (as in the previous section) and by reducing the effectiveness of FRP jacketing through the use of a coefficient equal to the ratio of $A_{s,c}/A_s$, where $A_{s,c}$ = area of longitudinal reinforcement inside the effectively confined part of the cross section.

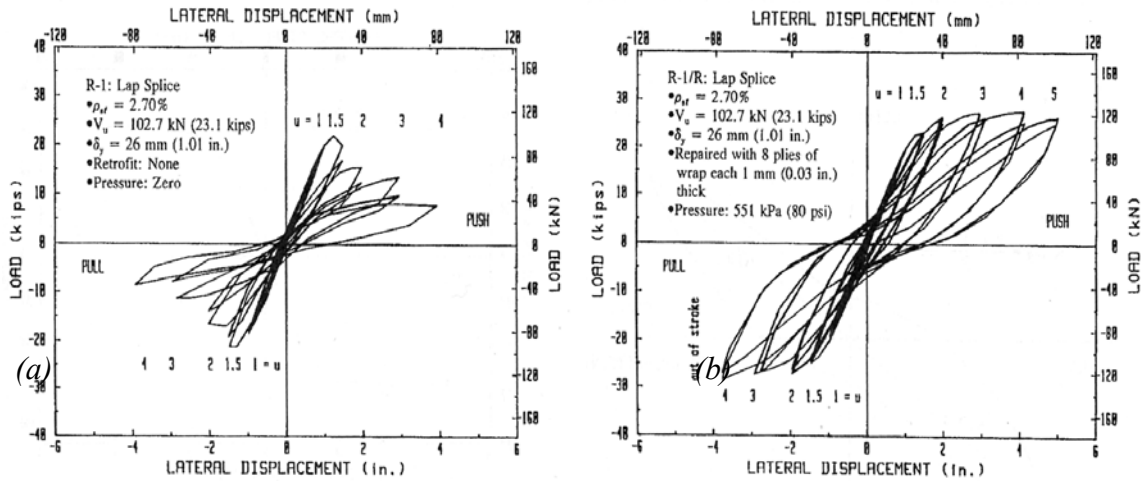


Fig. 13: Lateral force-displacement response of flexure-dominated rectangular columns with base lap-splices: (a) as-built; (b) retrofitted with FRP jacket at the lap-splice region (Saadatmanesh et al. 1997).

(5) Rebar buckling

To prevent column rebar buckling in the plastic hinge region, Priestley et al. (1995) proposed the following equation for the required volumetric transverse reinforcement ratio ρ_f of FRP ($= 4t_f/D$ in circular columns):

$$\rho_f = \frac{4t_f}{D} = \frac{0.45nf_s^2}{E_{ds}E_f} \quad (13)$$

where:

$$E_{ds} = \frac{4E_sE_i}{\left(\sqrt{E_s} + \sqrt{E_i} \right)^2} \quad (14)$$

and n = number of longitudinal rebars, f_s = longitudinal steel stress at a strain of 0.04, E_s = secant modulus of steel from f_s to f_u (ultimate stress) and E_i = initial elastic modulus of the longitudinal steel reinforcement. For rectangular cross sections, a confinement effectiveness coefficient could

be applied along the lines of the previous section, resulting in increased FRP thickness requirements.

References

- ACI 440.2R-01, 2001. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, ACI, Detroit, Michigan.
- Antonopoulos, C. and Triantafillou, T.C. 2002. Analysis of FRP-strengthened RC beam-column joints. *Journal of Composites for Construction (ASCE)*, Vol. 6, No. 1, pp. 41-51.
- Antonopoulos, C. and Triantafillou, T.C. 2003. Experimental investigation of FRP-strengthened RC beam-column joints. *Journal of Composites for Construction (ASCE)*, Vol. 7, No. 1, pp. 39-49.
- Castellani, A., Negro, P., Colombo, A., Grandi, A., Ghisalberti, G. and Castellani, M. 1999. Carbon fiber reinforced polymers (CFRP) for strengthening and repairing under seismic actions. European Laboratory for Structural Assessment, Joint Research Centre Research Report I.99.41, Ispira, Italy.
- Chang, K.C., Liu, K.Y. and Chang, S.B. 2001. Seismic retrofit study of RC rectangular bridge columns lap-spliced at the plastic hinge zone. In: *Proc. of the International Conference on FRP Composites in Civil Engineering*, Hong Kong, pp. 869-875.
- Fardis, M.N. and Khalili, H.H. 1981. Concrete encased in fiberglass-reinforced plastic. *ACI Journal*, Vol. 78, No. 6, pp. 440-446.
- Feng M.Q and Bahng E.Y (1999). Damage assessment of jacketed RC columns using vibration tests. *Journal of Structural Engineering (ASCE)*, Vol. 125, No. 3, 265-271.
- fib* (2001). Externally bonded FRP reinforcement for RC structures. Bulletin 14, prepared by the working party EBR of the Task Group 9.3.
- Fujii, S., Matsuzaki, K., Nakano, K. and Fukuyama, H. 1999. Japanese state of the art on seismic retrofit by fiber wrapping for building structures: Evaluations. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*. ACI Report SP-188. Detroit, Michigan, pp. 895-906.
- Fujisaki, T., Hosotani, M., Ohno, S. and Mutsuyoshi, H. 1997. JCI state-of-the-art on retrofitting by CFRM – research. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 613-620.
- Fukuyama, H., Suzuki, H. and Nakamura, H. 1999. Seismic retrofit of reinforced concrete columns by fiber sheet wrapping without removal of finishing mortar and side wall concrete. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*. ACI Report SP-188. Detroit, Michigan, pp. 205-216.
- Gergely, J., Pantelides, C.P. and Reaveley, L.D. 2000. Shear strengthening of RC T-Joints using CFRP composites. *Journal of Composites for Construction (ASCE)*, Vol. 4, No. 2, pp. 56-64.
- Hamilton, H.R. and Dolan, C. 2000. Durability of FRP reinforcements for concrete. *Progress in Structural Engineering and Materials*, No. 2, pp. 139-145.
- Harmon, T.G., Gould, P.L., Wang, E. and Ramakrishnan, S. 1998. Behavior of confined concrete under cyclic loading. In: Saadatmanesh H & Ehsani MR (eds) *Second International Conference on Composites in Infrastructure, ICCI '98, Tucson, AZ (Vol 1)*, pp. 398-410.
- Haroun, M.A., Mosallam, A.S., Feng, M.Q. and Elsanadedy, H.M. 2001. Experimental investigation of seismic repair and retrofit of bridge columns by composite jackets. In: *Proceedings of the International Conference on FRP Composites in Civil Engineering*, Hong Kong, pp. 839-848.
- Hosotani, M., Kawashima, K. and Hoshikuma, J. 1997. A study on confinement effect of concrete cylinders by carbon fiber sheets. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 209-216.
- Iso, M., Matsuzaki, Y., Sonobe, Y., Nakamura, H. and Watanabe, M. 2000. Experimental study on reinforced concrete columns having wing walls retrofitted with continuous fiber sheets. In: *Proceedings 12th World Conference on Earthquake Engineering*, New Zealand, pp.1865-1871.
- Japan Concrete Institute, 1998. Technical Report on Continuous Fiber Reinforced Concrete.
- JSCE (2001). *Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets*, edited by Maruyama K, Japan Society of Civil Engineers.
- Karbhari, V.M. and Gao, Y. 1997. Composite jacketed concrete under uniaxial compression – verification of simple design equations. *Journal of Materials in Civil Engineering (ASCE)*, Vol. 9, No. 4, pp.185-193.

- Kobatake, Y., Kimura, K. and Katsumata, H. 1993. A retrofitting method for reinforced concrete structures using carbon fiber. In: Nanni A (ed) *Fiber-Reinforced-Plastic (FRP) Reinforcement for Concrete Structures: Properties and Applications*. Elsevier Science Publishers, pp. 435-450.
- La Tegola, A. and Manni, O. 1999. Experimental investigation on concrete confined by fiber reinforced polymer and comparison with theoretical model. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*. ACI Report SP-188. Detroit, Michigan, pp. 243-253.
- Lam, L. and Teng, J.G. 2001. A new stress-strain model for FRP-confined concrete. In: *Proceedings of International Conference on FRP Composites in Civil Engineering*, Hong Kong, pp. 283-292.
- Lombard, J.C., Lau, D.T., Humar, J.L., Cheung, M.S. and Foo, S. 2000. Seismic repair and strengthening of reinforced concrete shear walls for flexure and shear using carbon fibre sheets. In: Humar J & Razaqpur AG (eds) *Advanced Composite Materials in Bridges and Structures*, Montreal: The Canadian Society for Civil Engineering, pp. 645-652.
- Ma, R. and Xiao, Y. 1997. Seismic retrofit and repair of circular bridge columns with advanced composite materials. *Earthquake Spectra*, Vol. 15, No. 4, pp. 747-764.
- Mander, J.B., Priestley, M.J.N. and Park, R. 1988. Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering, ASCE* **114**(8): 1804-1826.
- Masukawa, J., Akiyama, H. and Saito, H. 1997. Retrofit of existing reinforced concrete piers by using carbon fiber sheet and aramid fiber sheet. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 411-418.
- Masuo, K. 1999. Seismic retrofitting of reinforced concrete columns with wing walls using carbon fiber reinforced plastic walls. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, ACI Report SP-188. Detroit, Michigan, pp. 193-204.
- Matsuzaki, Y., Nakano, K., Fujii, S. and Fukuyama, H. 1999. Japanese state of the art on seismic retrofit by fiber wrapping for building structures: Research. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, ACI Report SP-188. Detroit, Michigan, pp. 879-893.
- Matsuzaki, Y., Nakano, K., Fujii, S. and Fukuyama, H. 2000. Seismic retrofit using continuous fiber sheets. *Proc., 12th World Conference on Earthquake Engineering*, New Zealand, pp. 2524-2531.
- Miyauchi, K., Nishibayashi, S. and Inoue, S. 1997. Estimation of strengthening effects with carbon fiber sheet for concrete column. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 217-224.
- Monti, G., Nistico, N. and Santini, S. 2001. Design of FRP jackets for upgrade of circular bridge piers. *Journal of Composites for Construction (ASCE)*, Vol. 5, No. 2, pp. 94-101.
- Mutsuyoshi, H., Ishibashi, T., Okano, M. and Katsuki, F. 1999. New design method for seismic retrofit of bridge columns with continuous fiber sheet – Performance-based design. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, ACI Report SP-188. Detroit, Michigan, pp. 229-241.
- Neale, K.W. 2000. FRPs for structural rehabilitation: a survey of recent progress. *Progress in Structural Engineering and Materials*, Vol. 2, pp. 133-138.
- Neubauer, U. and Rostásy, F. S. 1999. Bond failure of concrete fibre reinforced polymer at inclined cracks – experiments and fracture mechanics model. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, ACI Report SP-188. Detroit, Michigan, pp. 369-382.
- Osada, K., Yamaguchi, T. and Ikeda, S. 1999. Seismic performance and the retrofit of hollow circular reinforced concrete piers having reinforcement cut-off planes and variable wall thickness. In: *Transactions of the Japan Concrete Institute*, Vol. 21, pp. 263-274.
- Pantazopoulou, S. 1995. Role of expansion on mechanical behavior of concrete. *Journal of Structural Engineering (ASCE)*, Vol. 121, No. 12, pp.1795-1805.
- Pantelides, C.P., Gergely, J., Reaveley, L.D. and Volnyy, V. 1999. Retrofit of RC bridge pier with CFRP advanced composites. *Journal of Structural Engineering (ASCE)*, Vol. 125, No. 10, pp. 1094-1099.
- Priestley, M.J.N. and Seible, F. 1995. Design of seismic retrofit measures for concrete and masonry structures. *Construction and Building Materials*, Vol. 9, No. 6, pp. 365-377.
- Priestley, M.J.N., Seible, F. and Calvi, G.M. 1996. *Seismic Design and Retrofit of Bridges*, John Wiley & Sons Inc., New York.

- Prota, A., Nanni, A., Manfredi, G. and Cosenza, E. 2001. Selective upgrade of beam-column joints with composites. In: Proceedings of the International Conference on FRP Composites in Civil Engineering, Hong Kong, pp. 919-926.
- Restrepo, J.I., Wang, Y.C., Irwin, R.W. and DeVino, B. 1998. Fibreglass/epoxy composites for the seismic upgrading of reinforced concrete beams with shear and bar curtailment deficiencies. In: Proceedings 8th European Conference on Composite Materials, Naples, Italy, pp.59-66.
- Saadatmanesh, H., Ehsani, M.R. and Jin, L. 1996. Seismic strengthening of circular bridge pier models with fiber composites. *ACI Structural Journal*, Vol. 93, No. 6, pp. 639-647.
- Saadatmanesh, H., Ehsani, M.R. and Jin, L. 1997. Repair of earthquake-damaged RC columns with FRP wraps. *ACI Structural Journal*, Vol. 94, No. 2, pp. 206-215.
- Saafi, M., Toutanji, H.A. and Li, Z. 1999. Behavior of concrete columns confined with fiber reinforced polymer tubes. *ACI Materials Journal*, Vol. 96, No. 4, pp.500-509.
- Saatcioglu, M. and Elnabeh, G. 2001. Seismic retrofit of bridge columns with CFRP jackets. In: Proceedings of the International Conference on FRP Composites in Civil Engineering, Hong Kong, pp. 833-838.
- Saiidi, S., Sanders, D.H., Gordaninejad, F., Martinovic, F.M. and McElhaney, B.A. 2000. Seismic retrofit of non-prismatic RC bridge columns with fibrous composites. In: Proceedings 12th World Conference on Earthquake Engineering, New Zealand, pp. 143-150.
- Samaan, M., Mirmiran, A. and Shahawy, M. 1998. Model of concrete confined by fiber composites. *Journal of Structural Engineering (ASCE)*, Vol. 124, No. 9, pp. 1025-1031.
- Seible, F., Innamorato, D., Baumgartner, J., Karbhari, V. and Sheng, L.H. 1999. Seismic retrofit of flexural bridge spandrel columns using fiber reinforced polymer composite jackets. In: Dolan CW, Rizkalla SH & Nanni A (eds) *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, ACI Report SP-188. Detroit, Michigan, pp. 919-931.
- Seible, F., Priestley, M.J.N., Hegemier, G.A. and Innamorato, D. 1997. Seismic retrofit of RC columns with continuous carbon fiber jackets. *Journal of Composites for Construction (ASCE)*, Vol. 1, No. 2, pp. 52-62.
- Sexsmith, R., Anderson, D. and English, D. 1997. Cyclic behavior of concrete bridge bents. *ACI Structural Journal*, Vol. 94, No. 2, pp. 103-113.
- Sheikh, S.A., Iacobucci, R. and Bayrak, O. 2000. Seismic upgrade of concrete columns with fibre reinforced polymers. In: Humar J & Razaqpur AG (eds) *Advanced Composite Materials in Bridges and Structures*. Montreal: The Canadian Society for Civil Engineering, pp. 267-274.
- Shimbo, H., Yamada, K., Asai, H., Iwata, M. and Yonekura, A. 1997. Aseismic retrofit of concrete piers with CFRP ring and expansive concrete. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 363-370.
- Sirbu, G., Yoshihiro, T., Sato, Y. and Ueda, T. 1998. Shear resisting capacity of a reinforced concrete pier retrofitted with carbon fiber sheet. In: *Transactions of the Japan Concrete Institute*, Vol. 20, pp. 233-238.
- Spoelstra, M.R. and Monti, G. 1999. FRP-confined concrete model. *Journal of Composites for Construction (ASCE)*, Vol. 3, No. 3, pp. 143-150.
- Tastani, S. and Pantazopoulou, S.J. 2002. Design of seismic strengthening for brittle RC members using FRP jackets. Proc. 12th European Conf. on Earthquake Engineering, London, paper 360.
- Triantafillou, T.C. 1998a. Strengthening of structures with advanced FRPs. *Progress in Structural Engineering and Materials*, Vol. 1, pp. 126-134.
- Triantafillou, T.C. 1998b. Shear strengthening of reinforced concrete beams using epoxy-bonded FRP composites. *ACI Structural Journal*, Vol. 95, pp. 107-115.
- Triantafillou, T.C. 2001. Seismic retrofitting of structures using FRPs. *Progress in Structural Engineering and Materials*, Vol. 3, No. 1, pp. 57-65.
- Triantafillou, T.C. and Antonopoulos, C.P. 2000. Design of concrete flexural members strengthened in shear with FRP. *Journal of Composites for Construction (ASCE)*, Vol. 4, No. 4, pp. 198-205.
- Wang, Y.C., Restrepo, J.I. and Park, R. 2000. Retrofit of reinforced concrete members using advanced composite materials. University of Canterbury Department of Civil Engineering Research Report 2000-3, New Zealand.
- Watanabe, K., Nakamura, H., Honda, Y., Toyoshima, M., Iso, M., Fujimaki, T., Kaneto, M. and Shirai, N. 1997. Confinement effect of FRP sheet on strength and ductility of concrete cylinders under uniaxial compression. In: *Non-Metallic (FRP) Reinforcement for Concrete Structures (Vol 1)*. Japan Concrete Institute, pp. 233-240.
- Xiao, Y. and Wu, H. 2000. Compressive behavior of concrete confined by carbon fiber composite jackets. *Journal of Materials in Civil Engineering (ASCE)*, Vol. 12, No. 2, pp. 139-146.

Zhang, A., Yamakawa, T., Zhong, P. and Oka, T. 1999. Experimental study on seismic performance of reinforced concrete columns retrofitted with composite materials jackets. In: Dolan CW, Rizkalla SH & Nanni A (eds) Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, ACI Report SP-188. Detroit, MI, pp. 269-278.