PERFORMANCE-BASED SEISMIC RETROFITTING OF RC STRUCTURES WITH FRPS - A BACKGROUND TO THE EUROPEAN PROVISIONS

S. Pantazopoulou(1), S. Tastani(2), G. Thermou(3), T. Triantafillou(4)

(1) Professor, Dept. of Civil Engineering, Lassonde Faculty of Engineering, York University, Canada, pantazo@yorku.ca
(2) Lecturer, Dept. of Civil Engineering, Democritus University of Thrace, Xanthi, Greece, stastani@civil.duth.gr
(3) Assistant Professor, Dept. of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece, gthermou@civil.auth.gr
(4) Professor, Dept. of Civil Engineering, University of Patras, Patras, Greece, ttriant@upatras.gr

Abstract

This paper is a comprehensive state of the art background document on European seismic design provisions which was assembled in support of the development of design guidelines by the fib Committee 5.1 regarding the use of externally applied Fiber Reinforced Polymers (FRP) materials in the seismic retrofitting of reinforced concrete structures. In the context of developing design guidelines, the underlying mechanistic models that support the derivation of provisions were assembled after critical evaluation of the existing proposals and with careful reference to the available experimental evidence, the comparative assessment of past models in the literature, and requirements established from first principles.

Keywords: Seismic Strengthening, Performance based retrofitting, Deformation capacity of RC

1. Introduction

Externally applied FRPs, when used as confining jackets on lightly reinforced concrete (r.c.) members, are effective measures for increasing the deformation capacity by suppressing the premature failure modes that usually occur in the absence of proper seismic detailing. Rehabilitation of r.c. members with FRP jacketing cannot affect the demand side of the design equation (apart from suppressing premature failure modes that might have otherwise controlled the response), whereas it can significantly enhance the supply. In this context, FRP jacketing is considered to be a **local intervention** in the seismic rehabilitation of r.c. structures.

The aim of this paper is to establish a new-generation framework for the design of seismic retrofits using FRP materials. Following prevailing earthquake and design practice, the paper establishes performance-based criteria for global and local retrofit requirements so that the rehabilitated structure can develop acceptable, repairable levels of damage in a severe earthquake and minimal (limited) levels of damage in the frequent event. The aims of FRP retrofit designs are the enhancement of strength and deformation capacity as well as the mode of failure control of the structure and its individual structural members. It is intended that this paper should serve as the background for the development of European seismic retrofit provisions using FRPs.

If on first assessment it is deemed necessary to also moderate the demand, the retrofit solution should include global measures to increase the effective stiffness of the structure K_{eff}. Note that by increasing K_{eff}, the demand may be reduced in two different ways: (i) A higher effective stiffness results in a lower predominant period tending towards the left in the displacement spectrum, i.e. in the range of lower relative displacements; (ii) through a more uniform distribution of deformation demand in the structure, which ensures that the magnitude of deformation demanded of individual members is lowered. Global intervention methods include, but are not limited to, the following: 1) addition of FRP longitudinal reinforcement (NSM or externally bonded FRP laminates), 2) r.c. jacketing of selected columns in the building, 3) addition of r.c. wall elements, 4) addition of steel X-braces, 5) addition of masonry infills (not common in the West). (FRP jacketing is only pertinent for local interventions and is not included in the global strategy of the retrofit.)

For the vast majority of structures, the period after retrofit will lie between the milestone values T_B and T_D of the EN 1998-1 [1] type I earthquake design spectra. For this period range, the elastic spectral displacement demand may be estimated using,

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot T^2 / 40 \quad \text{and} \quad T_C \leq T \leq T_D: \quad S_d(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot (T_C T / 40) \qquad (1)$$

At the preliminary stage of calculation it may be assumed that this displacement will be increased by about 20 % when transferring from the spectrum to the actual structure. The displacement value may be further increased from the above value if inelasticity occurs. The total average elastic drift ratio (denoted as θ_{dem} for drift demand) for the retrofitted structure is approximated by:

$$\theta_{dem} = 1.2 \cdot S_d(T) / H_{tot} \qquad (2)$$

The target or improved period T_{trg} may be selected by requiring that the average drift demand θ_{dem} of the structure (Eq. 2) will not exceed a preset limit value, which after substituting Eq. (1) in Eq. (2) will yield the required value for T_{trg}. Such preset limit values may be 0.5 % (for performance limit A: damage limitation, $\mu_\theta \approx 1$), 1.25 % (B: repairable damage, $\mu_\theta = 2.5$) or 2 % (C: collapse prevention or life safety, $\mu_\theta > 3.5$). It is not advisable to allow for $\mu_\theta > 2.5$ for retrofitted structures. (Fig. 1 plots the base shear V against the lateral drift ratio θ of the structure, given as a multiple of the value at yielding; the ductility factor μ_θ is the multiplier of θ_y).

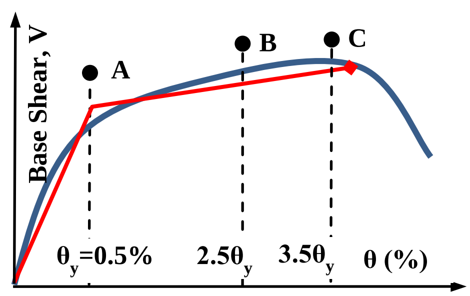

Fig. 1 - Performance limits

1.2 Determining the required stiffness

Engineered modification of the fundamental mode of lateral vibration is achieved through a weighted distribution of added stiffness over the height of the building [2-4]. In the case of the triangular shape (Fig. 2a), the solution is provided in the charts of Fig. 2b-c. These charts were derived considering a minimum storey height h_{st} = 3 m and unit storey mass m = 1 tonne; they can be used to define a target period and chosen deflection shape. Then, using the charts of Fig. 2b, the stiffness required for the first storey can be obtained directly, along with the required distribution of stiffness over the height of the retrofitted building. (Using the charts of Fig. 2c and given the number of floors in the structure, it is possible to obtain the required stiffness for all floors as a fraction of the first storey stiffness.)

The procedure described in Section 1 enables estimation of the required storey stiffness for a given building (i.e. with known distribution of mass) in order to achieve the specified target period and fundamental mode of vibration characteristics according to the designer's choice. The last step in the procedure involves selecting the global intervention method and detailing of the actual members of the building in order to achieve the stiffness addition defined.

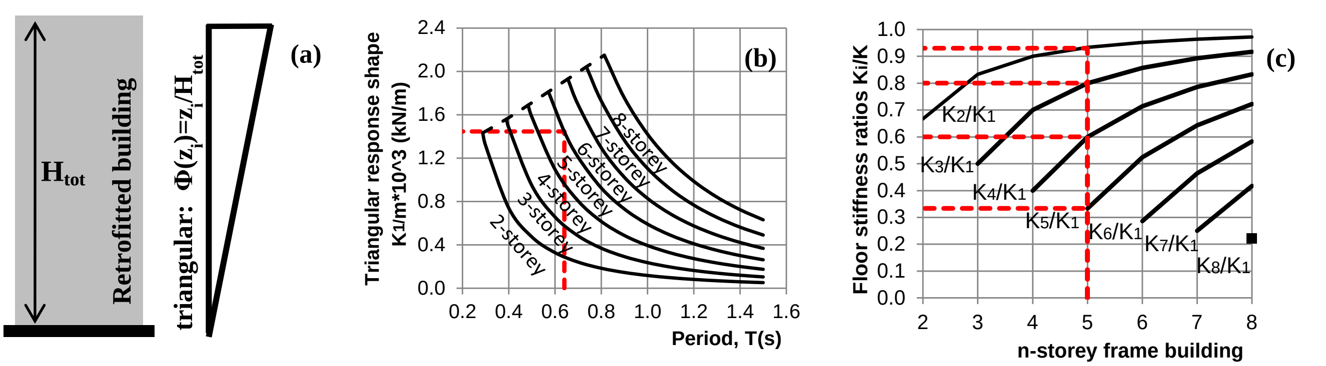

Fig. 2 – a) Triangular displacement profile. For 2 -up to 8-storey frame buildings: b) Stiffness to mass ratio for the first storey, K_1/m, versus period. c) Floor stiffness ratios k_i (=K_i:K_1).

2 Detailing of FRP interventions for seismic applications

Seismic retrofitting of r.c. structures with FRP may be carried out in order to upgrade a variety of structural deficiencies if upon assessment according to the established code framework [5] it is shown that seismic safety may be compromised at the design performance limit state. For evaluating the structure's safety and for defining the retrofit objectives, reference is made to verification of acceptable limit states as described in the reference code document. Similarly, the seismic hazard considered for the retrofit is identical to that used for new designs unless – through special provisions – the national standards enable a different importance level category to be assigned to the retrofitted structure in order to account for a residual service life different from the 50-year standard. Analysis of the retrofitted structure may be carried out to check against the established acceptance criteria, following the methods of analysis used in the assessment procedure.

Material safety factors γ_m refer to the FRP materials typically used today (GFRP, CFRP and AFRP with strengths range 1500 - 3500MPa and nominal rupture strains 1.5 - 2.5%). For retrofit design these are: a) For existing concrete and steel reinforcement, the confidence factors are used to divide mean material strength values depending on the knowledge level attained [5]. b) For FRP, the γ_m depends on the development method of the FRP material and the member classification (primary or secondary [1], i.e. for primary elements, if FRP is anchored in brittle substrate then $\gamma_f = 3$, and in the case of fully wrapped FRP layer then $\gamma_f = 1.5$).

The FRP material to be used in the retrofit solution and its arrangement depend on the overall objectives of the retrofit design. A general guideline is to aim for a uniform distribution of strength and stiffness among members on any given floor in order to minimize the risk of disproportionate damage to any single element. Major building irregularities cannot be eliminated using FRP as a strengthening technique, although the addition of FRP strips as longitudinal reinforcement can be counted as a global intervention as they can be used to increase the strength of individual members. It is essential to eliminate brittle failure modes through FRP jacketing so that the flexural capacity of the member may be fully developed and sustained up to the ductility level required by the design.

Extensive experimental evidence supports the use of FRPs as a pertinent material in seismic retrofitting, particularly for reinforced concrete beams, columns, walls and beam-column connections. FRP retrofit schemes that are well documented and support the establishment of detailing rules include the following solutions: 1) Increasing the member shear capacity by using FRP material with fibres running orthogonal to the direction of the axis of the strengthened member. 2) Increasing the ductility of end sections of beams and/or columns. 3) Improving the efficiency of lap splices. 4) Delaying the occurrence of buckling of steel longitudinal bars by using FRP material (in cases 2, 3 and 4 FRP material is wrapped around the member cross-section). 5) Increasing the diagonal tension capacity of beam-column joints by using FRP material installed with fibres located along the line of the principal tensile stresses.

In detailing the retrofit solutions, each retrofitted member is designed using capacity design principles. To secure adequate ductility, flexural yielding should control the response of the retrofitted member. So the member retrofit details should be proportioned with reference to flexural overstrength. The shear force associated with flexural yielding of the member is referred to as flexural shear demand V_{flex} (Fig. 3). When considering individual members, local strengthening schemes for individual linear members have relatively little effect on V_{flex} and depend on the confining action of the FRP reinforcement. Thus, the efficacy of a strengthening scheme in these cases depends on the magnitude of the confining pressure. The

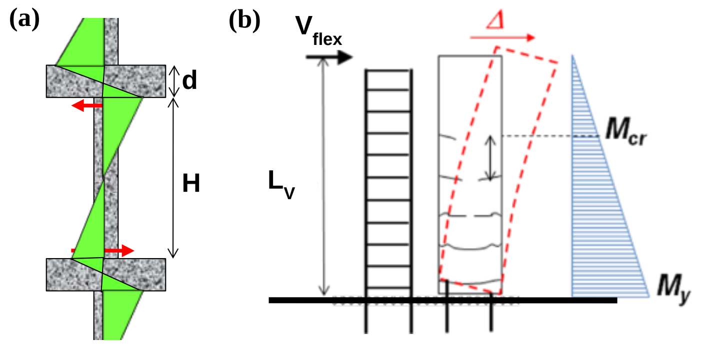

Fig. 3 - Static model used for beam-column elements undergoing lateral sway. (b) The cantilever part has the same moment distribution as the swaying column over the shear span, $L_v = H/2$ (thus, $V=M/L_V$).

role of the FRP properties in each resistance mechanism associated with the strengthening objectives of individual members listed above are reviewed briefly in the following.

2.1 Determining the displacement demand of the individual structural members

The required curvature ductility at the critical sections of members on the *ith* floor may be obtained with reasonable approximation using Eq. (3) whereas the maximum compression strain demand for the columns $\varepsilon_{cu,c}$ may be estimated from Eq. (4), [6], where $\nu_{d,max}$ is the maximum axial load ratio of a typical column for the seismic combination and ε_{sy} is the yield strain of the steel.

$$\mu_{\phi,i} = 2\mu_{\theta,i} - 1 \quad (3) \qquad \text{and} \qquad \varepsilon_{cu,c} = 2.2 \cdot \mu_{\phi} \cdot \varepsilon_{sy} \cdot \nu_{d,max} \geq 0.0035 \quad (4)$$

Ductility is achieved if the longitudinal steel reinforcement is engaged in post-yielding response prior to the occurrence of: a) delamination of concrete cover in the compression zone, b) failure of lap-splices or anchorages, c) diagonal tension failure of the member's web, d) control of bar buckling in the compression zone of a member, e) disintegration of the confined concrete core under high compression strain demands.

FRP jacketing may be used for the effective elimination of these occurrences and also to enhance the deformation and ductility capacity of a reinforced concrete member. The term FRP jacketing refers to any type of application of the material where the primary fibers are oriented transverse to the longitudinal axis of the upgraded member and on a minimum of three faces (properly anchored U-shaped and closed types exclusively) of the member's cross-section in order to facilitate a confining action against any dilation of the concrete (i.e. due to axial load, shear transverse tension or dilation produced by the bond action of a ribbed bar). A critical design parameter in all cases is the confining pressure introduced by the FRP jacket.

2.2 Confining pressure in FRP-encased concrete

The confining pressure exerted by the FRP jacket encasing a reinforced concrete member is estimated with reference to Fig. 4 (closed FRP jackets exclusively). This pressure, denoted by σ_{lat}, is given by Eq. (5).

$$\sigma_{lat} = 0.5 \cdot \left(\alpha_f \cdot \rho_{fv} \cdot E_f \varepsilon_{f,d} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st}\right) \quad (5)$$

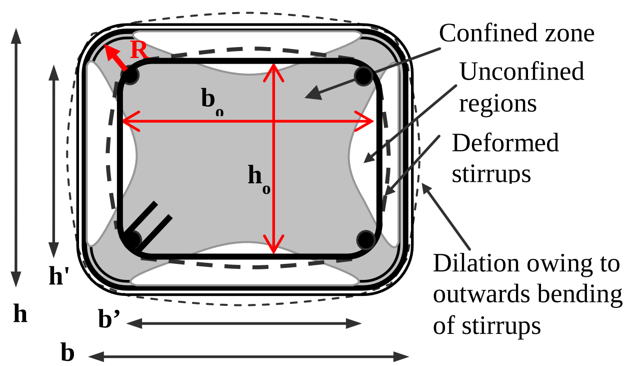

Fig. 4 – Confined section through closed FRP jacket

To account for the reduced efficiency of confinement in rectangular cross-sections, an effectiveness coefficient α_f is used to modify the FRP component of the confining stress (as per the effectiveness coefficient α_w used for stirrups [1]). Parameter ε_{fd} is the design value for the strain capacity of the transverse jacket, defined in Section 2.3. Parameters ρ_{fv} and ρ_{sv} are the volumetric ratios of transverse reinforcement (Eq. 6).

$$\rho_{fv} = 2 \cdot t_f \cdot (h+b)/(h \cdot b) \quad ; \quad \rho_{sv} = \left(A_{sw-\chi} \cdot b_o + A_{sw-y} \cdot h_o\right)/\left(s \cdot h_o \cdot b_o\right) \quad (6)$$

The effective thickness is estimated from the number of FRP layers n in the jacket and the thickness of a single layer t_o. Therefore, $t_f = t_o \cdot n^{0.85}$ for $n \geq 4$, otherwise, $t_f = t_o \cdot n$ for $n < 4$ ([6], i.e. the jacket layers are calculated as follows: From t_f (Eq. 6) calculate $n = t_f / t_o$. If $n < 4$, then the calculated number of layers is applied, but if $n > 4$, then recalculate the increased number of layers by applying $n = (t_f/t_o)^{1/0.85}$. As the number of layers increases, so the effective strain in the exterior layers is reduced due to the increased stiffness of the jacket. Therefore, the choice of alternative schemes for better use of material ought to be considered.)

2.3 Design tensile strain in FRP jacket ε_{fd}

The allowable tensile strain in the jacket ε_{fd} should not exceed the design limit $\varepsilon_{f,max} = \varepsilon_{fu}/\gamma_f$, where γ_f is taken as 1.5 for fully wrapped retrofit arrangement (refers to closed jackets), and 3 for anchorage on brittle substrate (refers to open jackets). For proportioning the FRP jacket, the axial tensile strain ε_{fd} in any FRP layer should not exceed the limit of Eq. (7). The usable design FRP strain is limited in order to protect the retrofit against premature local failures such as: (i) Rupture of the FRP at the corners occurs due to lateral dilation of concrete under high compressive strains in the compression zone of confined members. To delay the occurrence of local rupture the corners of the cross-section should be chamfered by a radius *R*. (ii) Rupture may also occur due to buckling of embedded compression reinforcement. Therefore, the design strain of the jacket is limited by:

$$\varepsilon_{fd} \le \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot \varepsilon_{f,max} \quad ; \quad \varepsilon_{f,max} = \varepsilon_{fu} / \gamma_f \tag{7}$$

Factor η_1 accounts for the radius of chamfer *R* at the corners of the member [7-8]:

$$\eta_1 = 0.25 + 2 \cdot (2R + D_b) / b' \le 1.0 \tag{8}$$

D_b is the embedded corner bar diameter. Equation (8) is valid for rectangular sections only (b' is the largest cross-section side); for circular members, $\eta_1 = 1$. Factor η_2 accounts for the development length of the wrap:

$$\eta_2 = l_b^{avail} / l_b^{min} \le 1 \tag{9}$$

where l_b^{min} is the minimum required overlap length of the exterior jacket layer (as calculated by implementing Eq. (10) for debonding failure of the FRP in a closed jacket or Eq. (11) for debonding failure of the FRP in an open FRP jacket) and l_b^{avail} is the available length of the cross-section side where the FRP is to be anchored.

- Debonding failure of the FRP in a closed jacket arrangement. The most critical layer for debonding is the external layer, since the shear strength of the adhesive in interior layers is enhanced by friction due to confinement. The minimum required overlap length of the exterior jacket layer l_b^{min} is:

$$l_b^{min} = 1.6 \sqrt{E_f \, t_o s_{ao} / \tau_a} \tag{10}$$

where τ_α is the shear strength of the adhesive at the stage of plastification and $s_{\alpha o}$ the slip of the adhesive at brittle shear failure (data for the adhesive provided by the adhesive supplier). For an adhesive that exhibits ductile shear response up to $s_{\alpha u}$, the coefficient 1.6 may be eliminated and $s_{\alpha u}$ used in lieu of $s_{\alpha o}$ in Eq. (10).

- Debonding failure of the FRP in an open FRP jacket arrangement (i.e. in the case of anchorage in a brittle substrate such as the concrete cover). The minimum development length measured from the critical section where ε_{fd} will be developed at the point – where the FRP intersects a flexural or shear crack of width w_{cr} – is:

$$l_b^{min} = 1.6 \sqrt{E_f \, t_f w_{cr} / \tau_{b1}} \tag{11}$$

The design bond strength is $\tau_{b1} = f_{ctk0.05} / \gamma_{fb}$, where $f_{ctk0.05}$ is the characteristic tensile strength of the concrete substrate and γ_{fb}=1.5, the concrete material safety factor. Design calculations may be performed for w_{cr}=0.5 mm.

Factor η_3 accounts for the redundancy of the jacket against debonding failure (for closed jackets, η_3 = 1.0, for U-type arrangements with special details at the ends to secure the jacket against debonding (e.g. adhesive anchors, NSM details, etc.), η_3 = 1.0, for U-type arrangements without special measures against debonding, η_3 = 0.85, for straight layers with special details at the ends to secure the jacket against debonding (e.g. adhesive anchors, NSM details, transverse confining wraps), η_3 = 0.9, for straight layers (parallel to the web depth) without special measures against debonding, η_3 = 0.6. (Note that this arrangement is discouraged by most relevant design codes due to the high risk of debonding, [5, 9, 10].)

2.3 Stress–strain law for FRP-confined concrete

The confined concrete strength f_{cc} and the corresponding strain at attainment of peak stress ε_{cc} in the compression zone of the encased cross-section may be calculated from the classical confinement model of Richart et al. [11] adapted to account for the greater compliance of jackets compared with conventional stirrups.

$$f_{cc} = f_{ck} + 1.5 \left(\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st} \right) \quad ; \quad \varepsilon_{cc} = 0.002 + 0.015 \left(\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st} \right) f_{ck} \tag{12}$$

The failure strain of confined concrete $\varepsilon_{cu,c}$ corresponding to a compression strength reduction in excess of 15 % is obtained from Pantazopoulou et al [7]:

$$\varepsilon_{cu,c} = \varepsilon_{c,u} + 0.075 \cdot \left(\zeta \cdot \left(\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st}\right) / f_{ck} - 0.1\right) \geq 0.0035 \quad (13)$$

When $\varepsilon_{cu,c} \leq 0.01$ $\zeta = 1$ and when $\varepsilon_{cu,c} \geq 0.02$ $\zeta = 0.6$. Coefficient ζ varies linearly between the two bounds for intermediate axial strain values and accounts for the reduced jacket effectiveness when a very high confinement is present. At such a very high limit, axial compaction of confined concrete accounts for part of the observed axial strain capacity without engaging the jacket through dilation of the core. Note that material safety factors are not used for concrete characteristic strength when determining the compression stress–strain law. Such a safety factor may have an adverse effect on the estimated hierarchy of failure when establishing capacity design principles. It is recommended that a safety factor be applied to the calculated member strength after retrofitting to account for uncertainties.

A note of caution is in order regarding the confinement models available: all models listed in the literature have been calibrated against a very large database of tests conducted on axially compressed members. Specimens were either reinforced or unreinforced. The stress–strain relationships derived do not account for the strain gradient effects that occur due to flexural moments. Using stress–strain relationships obtained from axial load tests to model the stress–strain behavior of concrete in the confined compression zone of members under combined axial load and moment is an area of inconsistency in the FRP-related literature.

2.4 Rotation capacity and displacement ductility of FRP-confined members

Based on ample experimental documentation, r.c. beams, columns and walls retrofitted with FRP jackets in the critical regions can develop significant rotation capacity and displacement ductility. Rotation capacity refers to the maximum angle that may be sustained between the chord of the member in the displaced position and the normal to the end cross-sections (Fig. 5). The ultimate chord rotation θ_u of members strengthened with FRP confinement may be estimated using one of the following procedures:

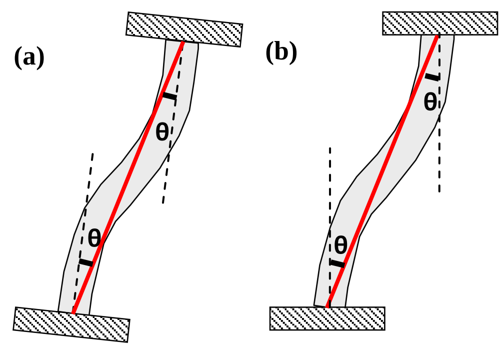

Fig. 5 - Definition of chord rotation θ when the transverse elements (i.e. beams) a) do and b) do not participate in the deformation of the storey

a) From basic mechanics:

$$\theta_u = \frac{1}{\gamma_{el}} \cdot \left[\theta_y + \left(\phi_u - \phi_y\right) \cdot \ell_{pl} \cdot \left(1 - 0.5 \cdot \frac{\ell_{pl}}{L_V}\right)\right] \quad (14)$$

where: γ_{el} = 1.5 for primary and 1.0 for secondary members (for the parameters θ_y, ℓ_{pl}, ϕ_u -the ultimate curvature of end section, evaluated by assigning at the concrete ultimate strain, $\varepsilon_{cu,c}$ from Eq. (13)-, ϕ_y -the curvature of end section at onset of yielding of tension reinforcement approximated by $2\varepsilon_{sy}/h$, see [5, 6, 12]).

b) Empirically, from the following expressions:

The $\varepsilon_{cu,c}$ determined from Eq. (13) is used to quantify the curvature ductility by reversing Eq. (4):

$$for\ \nu_{d,max} \geq 0.2: \quad \mu_\phi = 0.45 \cdot \varepsilon_{cu,c} / (\varepsilon_{sy} \cdot \nu_{d,max}) \quad ; \quad for\ \nu_{d,max} < 0.2: \quad \mu_\phi = 0.45 \cdot (\varepsilon_{cu,c} / \varepsilon_{sy}) \cdot h / (\xi \cdot d) \quad (15)$$

Here, θ_y may be estimated from $\theta_y = 1/6\phi_y H = 1/3\varepsilon_{sy}[H/h]$, where H/h is the aspect ratio of the member (H = member depth and h = cross-section depth of member). Using Eq. (3), the available μ_θ is calculated:

$$\mu_\Delta = \mu_\theta = 0.5 \cdot (\mu_\phi + 1) \quad and \quad \theta_u = \mu_\theta \cdot \theta_y \quad (16)$$

A simplification made here was to assume that $\ell_p \approx 0.5h$ and $H/h \approx 6$. The value from Eq. (16) should be multiplied by 1.5 in order to account for the contribution of the reinforcement pull-out to the rotation capacity.

3. Safety requirements for brittle members and mechanisms

3.1 Shear

The shear strength of FRP-jacketed RC members V_{Rd} should exceed the retrofitted flexural strength $V_{Ed}=M_{Rd}/Lv$ in order to preclude shear failure. The shear strength V_{Rd} of the retrofitted member comprises the contributions of the original member $V_{Rd,o}$ and the FRP jacket $V_{Rd,f}$:

$$V_{Rd} = V_{Rd,o} + V_{Rd,f} \tag{17}$$

The cyclic shear resistance $V_{Rd,o}$ of the original member decreases with the plastic part of the ductility demand, expressed in terms of the ductility factor of the transverse deflection of the shear span or the chord rotation at the end of the member $\mu_{\theta,pl} = \mu_{\theta}-1$. For this purpose, $\mu_{\theta,pl}$ may be calculated as the ratio of the plastic part θ_{pl} of the total chord rotation θ_u normalized to the chord rotation at yielding θ_y. The Equation (18a) [6] may be used for the shear strength, as controlled by yielding of the embedded stirrups (units: MN and m):

$$V_{Rd,o} = \frac{1}{\gamma_{el}} \cdot \left[\frac{h-x}{2L_V} \cdot min\left(N; 0.55A_c f_c\right) + \left[1-0.05 \cdot min(5; \mu_{\theta,pl})\right] \cdot \left(V_{Rd,c} + V_{Rd,s}\right) \right] \tag{18a}$$

Terms $V_{Rd,c}$ and $V_{Rd,s}$ represent the contributions of the concrete compression zone and the web reinforcement to the shear strength of the original member (prior to retrofitting with FRP jacket). Term $V_{Rd,s}$, as represented in the established codes of practice, is used in Eq. (18a). Note that the expression corresponds to a 45^o angle shear truss. Term $V_{Rd,c}$ is taken reduced from the code expressions in recognition of the recent understanding that only the compression zone of a cross-section participates in shear transfer [13] resulting in,

$$V_{Rd,c} = 0.41\sqrt{f_c} \cdot (b \cdot x) \quad ; \quad V_{Rd,s} = \rho_{sw-y} \cdot b_o \cdot h_o \cdot f_{y,st} \tag{18b}$$

Term $x = \xi d$ represents the depth of the compression zone at the state of sectional equilibrium at ultimate flexural capacity (accounting for the simultaneous action of the design axial load value for the seismic combination $NG+0.3Q\pm E$). Term ρ_{sw-y} is the web reinforcement ratio in the direction parallel to the shear force (design shear here is assumed to act in the y direction of the member's cross section):

$$\rho_{sw-y} = A_{sw-y} / (b_o \cdot s) \tag{18c}$$

The contribution of the FRP jacket $V_{Rd,f}$ is calculated similarly to $V_{Rd,s}$ as follows:

$$V_{Rd,f} = \rho_{f-y} \cdot b \cdot h \cdot f_{fwd} \cdot (1 + cot\, a_o) \cdot sin\, a_o \quad ; \quad f_{fwd} = E_f \cdot \varepsilon_{fd} \tag{19}$$

where α_o is the angle of FRP fibers with respect to the longitudinal axis of the member. The value of f_{fwd} depends of the type of the externally applied FRP (closed, three-sided or two-sided; the latter, being the weakest of all alternatives, is usually prohibited by several codes, i.e. [6, 9-10]) determined by the pertinent value of ε_{fd}. Term ρ_{f-y} is the FRP jacket reinforcement ratio in the direction parallel to the shear force (with design shear here assumed to act in the y direction of the member's cross section):

$$\rho_{f-y} = 2t_f / b \quad ; \quad t_f = t_o \cdot n^{0.85} \quad for \quad n \geq 4 \quad ; \quad t_f = t_o \cdot n \quad for \quad n < 4 \tag{20a}$$

If the FRP reinforcement is applied in strips of width b_f at a centre-to-centre longitudinal spacing s_f, the ρ_{f-y} is,

$$\rho_{f-y} = 2t_f b_f / (b \cdot s_f) \tag{20b}$$

The shear strength estimated according to Eq. (17) should not exceed the following limit value for shear $V_{Rd,max}$, which corresponds to crushing of the diagonal compression struts in the web of the member, modified to account for the confined concrete strength [12]:

$$V_{Rd,max} = 0.5 \cdot (1 - f_c / 250) \cdot f_{cc} \cdot b \cdot h \cdot (1 + cot\, a_o) \tag{21}$$

3.2 Lap splices

Slip of existing steel reinforcement in r.c. columns at lap splice locations may be avoided by confining the member cross-section with FRP. FRP wrapping over the embedment length of bar anchorages provides clamping, resisting propagation of cover splitting and thus enhancing the frictional mechanism of bond resistance. FRP jacketing enables attainment of high strain demands in the tension reinforcement at the critical section. The increased demand for bar development capacity cannot always be met by the anchorage/lap splice, which is often inadequate in substandard construction or inaccessible for rehabilitation. The FRP jacket layers required are intended to enhance bond strength in order to develop yielding of the embedded lapped reinforcement at the critical sections near the support. In existing structures where the available lap length Lo is known, the required bond stress may be evaluated with

$$\tau_{b1} \geq \gamma_{el} \cdot D_b \cdot f_{s,y}(4L_o) \tag{22}$$

The bond strength of lapped bars in the retrofitted member comprises contributions from concrete cover, web reinforcement and added FRP jacket [14]:

$$\tau_{b1} = 2\mu_{fr} /(\pi D_b) \cdot \left(2c \cdot f_{ctk0.05} + 0.33 \cdot A_{st}\, f_{y,st} /(N_b\, s) + 2\, t_f\, E_f \varepsilon_{sl}^f / N_b\right) \tag{23}$$

where, N_b number of tension bars (or pairs of spliced tension bars if reinforcement is spliced) laterally restrained by the transverse pressure (e.g. if a cross-section has eight bars evenly distributed around the perimeter – three bars each side –, then N_b = 3 in the cross-section region with the highest tension stresses, whereas if there are eight pairs of spliced bars around the perimeter, then again N_b = 3), c is the concrete cover, A_{st} the area of stirrup legs enclosing the N_b lapped bars (area of legs crossing splitting plane), s the stirrup spacing along member length (with only a few stirrups the stirrup term of Eq. (23) may be neglected for safety).

The effective strain $\varepsilon_{sl}{}^f$ of the FRP jacket is linked to the degree of acceptable damage along the splice length, which is reflected in the value of the coefficient of friction μ_{fr}. Based on fib Model Code 2010 [15], bond stress reaches bond strength at a slip value of 0.1 mm. For that limit, damage to the anchorage is negligible, and the corresponding coefficient of friction μ_{fr} = 1. For higher slip values, the value of μ_{fr} degrades due to plastification or cracking in the lapped length. Term $\varepsilon_{sl}{}^f$ of the FRP jacket [7, 14] is calculated by Eq. (24),

$$\varepsilon_{sl}^f = 0.05/\left(c + 0.5D_b\right) \tag{24}$$

where cover c the clear cover of longitudinal bars of diameter D_b. Equation (23) may be used to determine the confining jacket thickness required t_f (for securing the lap splice capacity of longitudinal reinforcement). In this case the required jacket thickness over the lap splice length is estimated using Eq. (25) ($A_b = \pi D_b^2/4$).

$$t_f \geq \frac{1}{2E_j \varepsilon_{sl}^f} \cdot \left[\gamma_{el} \cdot \frac{N_b \cdot A_b}{L_o} \cdot \frac{f_{sy}}{2\mu_{fr}} - N_b \cdot p_{cr} \cdot f_{ctk0.05} - 0.33 \cdot \frac{A_{st}\, f_{y,st}}{s}\right] \tag{25}$$

If the member has sustained damage during previous loading and the lap splices show signs of distress, then it is advisable to patch repair the damaged cover by replacing it with repair mortar. If no such repair is possible, then the residual, rather than the full, contributions of the cover concrete should be considered in Eq. (23). In this case it is sufficient to reduce the concrete term in Eqs. (23) and (25) to 1/3 of its initial value. Note here that as $\varepsilon_{sl}{}^f$ is very small, the calculated number of FRP layers is usually $n \geq 4$, so the effective jacket thickness should be $t_f = t_o \cdot n^{0.85}$.

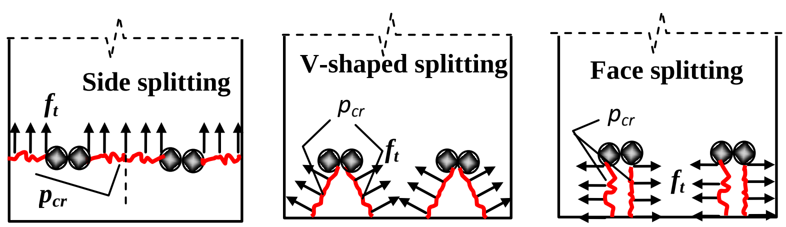

Fig. 6 - Definition of the crack-path length p_{cr}.

Further, in Eq. (25), term p_{cr} is used instead of $2c$, which appeared in the initial Eq. (23), since the potential splitting mechanisms are modified as shown in Fig. 6 in the light of the confining action of the jacket. Term p_{cr} refers to the length of cracking produced by a single bar or a pair of spliced bars at bond failure. If a V-shaped crack pattern is adopted, then $p_{cr}=2 \cdot 2^{0.5}c$, where c is the vertical cover. Note that if $N_b \cdot p_{cr} > (b-2c_h-D_bN_b)$ or $N_b \cdot p_{cr} > (b-2c_h-2D_bN_b)$ for bars or pairs of spliced bars respectively (c_h=side/horizontal cover width), then the critical splitting plane is the horizontal one that crosses all the bars. In this case the value of $c_h + 0.5 \cdot (b-2c_h-D_bN_b)/(N_b-1)$ or $c_h+0.5 \cdot (b-2c_h-2D_bN_b)/(N_b-1)$ may be used as p_{cr} in Eq. (25) for bars or pairs of spliced bars respectively.

3.3 Brittle failure of the jacket - Buckling of longitudinal bars

In lightly reinforced r.c. members, the compression strain capacity of longitudinal reinforcement is often limited by premature buckling owing to the large unsupported length of the bars (Fig. 7a). The bar slenderness ratio of compression steel bars supported laterally by stirrups is defined as $\lambda = s/D_b$. Recommended values of λ for high to moderate ductility structures are in the range 6–8. Values of $\lambda > 10$ are excessive where the bar may undergo elastic buckling prior to yielding. In such cases the susceptibility of the FRP jacket to stress concentrations limits its effectiveness as lateral support for the longitudinal bars after they reach critical conditions for buckling.

FRP jacketing may delay but cannot preclude eventual buckling of compression reinforcement. The confinement induced by jacketing provides lateral support to the cover concrete, so delamination is not prevented. The critical buckling load of compression bars diminishes after yielding in compression. FRP confinement allows the concrete in the compression zone to develop a large deformation capacity. So redistribution is possible from the longitudinal reinforcement to the concrete when the former reaches conditions of instability. Two alternative options are considered in order to calculate the required FRP confinement in order to a) eliminate the occurrence of buckling or b) increase the deformation capacity of reinforcement in the compression zones of concrete members.

In plastic hinge regions, lateral buckling is the usual form of compression reinforcement failure due to lateral shear distortion of the member in that region. A criterion for design of the required lateral restraint to be provided by the jacket is the requirement that the strain capacity of the confined concrete $\varepsilon_{cu,c}$ should exceed the critical strain $\varepsilon_{s,crit}$ at the onset of reinforcement buckling. In this case (where $\varepsilon_{cu,c} > \varepsilon_{s,crit}$), redistribution between the compressed bars at incipient buckling and the encased concrete is possible, thus postponing buckling to occur at a higher strain level [8, 14, 16]. The critical s/D_b ratio that corresponds to the rebar stress $f_{s,crit}$ is:

$$s / D_b = \psi \sqrt{E_t / f_{s,crit}} \tag{26}$$

where E_t is the tangent modulus of steel at the stress level considered (see Fig. 7b and [14]) and ψ a parameter that accounts for the buckling length ($\psi = \pi/4$ for symmetric buckling and $\psi = \pi/2$ for lateral buckling, Fig. 7c). Given the full stress–strain law of the longitudinal bars in compression (which is often assumed to be identical to that in tension for lack of detailed data), the limiting strain ductility $\mu_{\varepsilon c} = \varepsilon_{s,crit}/\varepsilon_{sy}$ is plotted against the s/D_b ratio (Fig. 7c). Parameter $\varepsilon_{s,crit}$ is the strain at which the bar will become unstable. Therefore, buckling of any individual bar segment is controlled by its strain ductility $\mu_{\varepsilon c}$-s/D_b curve unless the dependable deformation capacity of encased concrete $\varepsilon_{cu,c}$ (e.g. Eq. (13)) exceeds the $\varepsilon_{s,crit}$ value corresponding to the available s/D_b ratio.

An important consideration when detailing the FRP jacket is to ensure that the target displacement ductility of the member after upgrading $\mu_{\Delta,req}$ may be attained prior to buckling of primary reinforcement. The steps to achieve this are as follows: (i) Estimate the target displacement ductility demand at the design performance limit state $\mu_{\theta,req}=\theta_{u,target}/\theta_y$. (ii) Estimate the curvature ductility demand $\mu_{\varphi,req}$ (where $\mu_\varphi = \varphi_u/\varphi_y$) in the plastic hinge region of the member using the relationship between μ_θ, and μ_φ from Eqs. (3-4). (iii) From $\mu_{\phi,req}$, find the compression strain ductility demand $\mu_{\varepsilon c,req}$ of the compression reinforcement: $\varepsilon_{cu,c}^{req}=2.2\mu_\phi^{req}\, \varepsilon_{sy}\, v_d^{max} \geq 0.0035$. (iv) Estimate the required jacket confinement to ensure that $\varepsilon_{cu,c} \geq \{\varepsilon_{cu,c}^{req}, \varepsilon_{s,crit}\}$.

3.4 Displacement ductility of FRP-jacketed RC members

FRP jacketing can suppress all failure modes apart from flexural yielding of reinforcement. The available displacement ductility μ_Δ as a function of transverse confining pressure σ_{lat} is estimated with [14]:

$$\mu_\Delta = 1.3 + 12.4\left(0.5\left(k_f^c \rho_{fv} E_f\, \varepsilon_{f,eff} + k_{st}^c \rho_{sv}\, f_{y,st}\right)/ f_c' - 0.1\right) \geq 1.3 \tag{27}$$

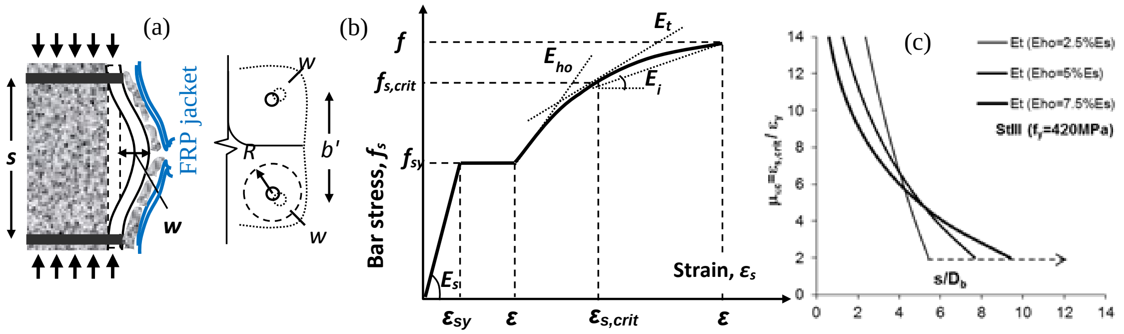

Fig. 7 - a) Symmetric buckling of fully supported steel bar, b) stress – strain diagram, c) Compressive strain ductility $\mu_{\varepsilon c} = \varepsilon_{s,crit}/\varepsilon_{sy}$ versus stirrup spacing s/D_b for steel category StIII and symmetric buckling.

In Eq. (27), the lower limit value of μ_Δ=1.3 recognizes the fact that lightly reinforced r.c. elements that overcome any premature elastic mode of failure are able to develop some limited displacement ductility (where $\varepsilon_{f,eff} = \varepsilon_{fu,d}$, $\varepsilon_{fu,d} = \varepsilon_{fu}/\gamma_f$, and γ_f is the FRP material redundancy coefficient). The above may be simplified by neglecting the contribution of stirrups if their arrangement is deemed as not conforming to modern standards.

4. Joints

Beam-column joints are regions of very high shear stress demand. The design shear force acting on the beam-column joint during seismic excitation may be estimated from the moment reversal which occurs between the end faces of the joint region, as the slope of the moment diagram over the depth of the beam or column (Fig. 8a). Joint failure occurs due to inadequate shear reinforcement or by crushing failure of the diagonal compressive strut that forms in the body of the joint (Fig. 8b). Requirements for retrofit draw from past knowledge about the behavior and design considerations of conventionally reinforced beam-column joints due to their importance in securing the integrity of the structure: the joint panel lies in the path of the vertical loads (overbearing weight of the structure) and as such, considerations of resilience and integrity of the retrofit necessarily lead to overdesign, consistent with capacity-design principles. The steep moment gradients in the joint panel facilitate reversal of moment from one face of the member to the other. In the ultimate limit state the design force in the joint is so significant that joint strength is thought to be supported primarily by the diagonal compressive strut that forms in the joint (Fig. 8b) provided that it is confined [17-18]. Current codes demand that the stirrup arrangement used in the end critical zones of the columns is also extended inside the joint panel in order to secure confinement [1]; however the effectiveness of confinement also depends on the number of free faces of the joint (that is, how many sides are unrestrained). It is notable that in reconnaissance reports joint failures are usually reported to occur in the perimeter of the building. In recognition of this fact the ACI-ASCE 352 Recommendations [19] limit the allowable shear stress input in an exterior joint to 66% of the value allowed in interior joints; the corresponding limit is at 80% in EN 1998-1 [1].

In old construction, joints are generally poorly detailed. This renders them susceptible to diagonal tension failure at relatively low levels of shear demand. Past experiments conducted in controlled laboratory conditions as well as analytical studies have demonstrated that r.c. joints in beam-column connections can be effectively strengthened with a pertinent arrangement of externally bonded FRP; analytical studies have also been developed to illustrate the mechanics of this strengthening scheme [20]. These studies support the development of rehabilitation procedures and detailing methods for strengthening of beam-column joints with FRP jacketing. However, the number of available exterior connection tests that reproduce faithfully the actual three-dimensional features of r.c. frame joints including the monolithic slab is still considered limited in light of the key role of joints in the overall structural integrity and survival in the event of a serious earthquake.

A note of caution is in order however: To be effective as a confining mechanism, FRP jacketing in beam-column joints should restrain lateral expansion of the encased strut without any risk of debonding failure or localized rupture. Because of the geometric complexity of actual 3-D frame connections that also include slabs, FRP strips must be ingeniously placed in order to achieve uniform and effective confinement of the compressive strut through the height, length and breadth of the exterior joint panel; to a large extent, this depends on the inventiveness and versatility of the engineer that supervises the retrofit. Anchorage by mechanical means or by chemical anchors is also advisable to eliminate the risk of failure by debonding.

In light of the weakness in the method necessarily imparted by the decisive dependency on the engineer's judgement as to the proper arrangement of the FRP jacket so as to effect the desirable confinement, has led to the development of two alternative options in designing FRP-based retrofits of beam-column connections. One neglects this confining contribution in the interest of conservatism and on the assumption that unless designed by specialists, this type of retrofit may prove inferior to expectations as to its confining effectiveness. This option determines the required amount of FRP reinforcement through its function as added shear reinforcement in the joint panel. This generally leads to significant amounts of added reinforcement that would need to be implemented in the form of strips (EBR or NSM). The other approach estimates the required FRP amount based on its confining action. Consistently with requirement (4) of Section 5.5.3.3 of EN 1998-1 [1] on beam-column joints, the integrity of the joints after diagonal cracking may be ensured by reinforcement crossing the diagonal crack paths and designed to support the full amount of the applied joint shear force. Thus, the required jacket thickness t_f is estimated neglecting the contribution of the diagonal strut that forms in the joint on account of the uncertain restraining action of the jacket when placed in the complex 3-D geometry of the connection. In deriving the following equations, the FRP fibres are taken oriented in the horizontal and/or vertical direction (in case of inclined fibres at an angle β with respect to the beam axis, the result of t_f obtained from Eqs. (28) is further divided by $(1+\cot\beta)\cdot\sin\beta$).

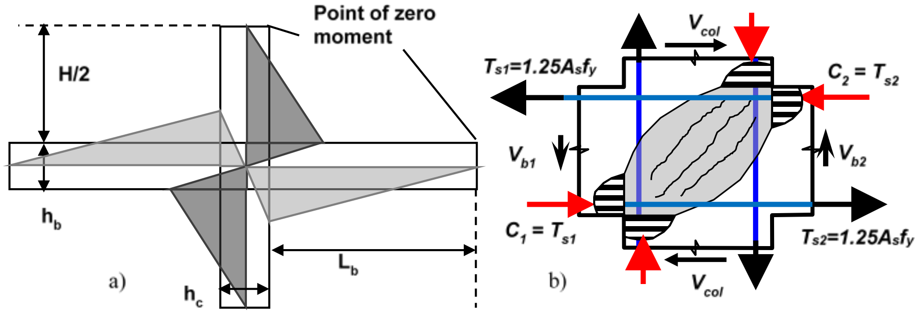

Fig. 8 - (a) Calculation of joint shear force, V_j from the gradient of flexural moments along the column or the beam line in the joint region. (b) Diagonal strut and definition of confinement requirements.

horizontally: $t_f = t_{f,h} = \gamma_{Rd} V_{j,h} /(h_b E_f \varepsilon_{fd})$; vertically: $t_f = t_{f,v} = \gamma_{Rd} V_{j,v} /(h_c E_f \varepsilon_{fd})$ (28)

In Eqs. (28) $V_{j,h}$ and $V_{j,h}$ are the design shear forces in the joint, assumed to act on a horizontal and a vertical plane through the joint respectively, and $\gamma_{Rd}=1.5$. Parameter ε_{fd} is the allowable design value of FRP tensile strain that, for the case considered, shall not be taken higher than 0.4%. An essential requirement is proper anchorage of the FRP strips. When FRP reinforcement is not properly anchored, FRP strengthening shall not be considered effective. When more than 2 FRP layers are needed, then the reinforcement shall be placed in the form of NSM strips and shall be encased transversely by properly anchored jacket layers. The calculations of $V_{j,h}$ and $V_{j,h}$ are based on EN 1998-1 Section 5.5.2.3 [1]; note that the necessary nomenclature is defined with reference to Fig. 8a.

For interior joints: $V_{j,h} = 1.25 \cdot (A_{s1} + A_{s2}) \cdot f_y - V_{col}$; For exterior joints: $V_{j,h} = 1.25 \cdot A_{s1} \cdot f_y - V_{col}$ (29)

Upper limit on beam-column joint demand: The requirement by [1] is enforced, limiting the diagonal compression induced in the joint by the diagonal strut mechanism in the presence of transverse tensile strains.

For interior beam-column joints: $V_{j,h} \le n f_{cd} \sqrt{1 - \frac{v_d}{n}} b_j h_{jc}$; $n = 0.6(1 - f_{ck}/250))$ (30a)

For exterior beam-column joints: $V_{j,h} \le 0.8 \cdot \left(n f_{cd} \sqrt{1 - \frac{v_d}{n}} b_j h_{jc} \right)$; $n = 0.6(1 - f_{ck}/250))$ (30b) where h_{jc} is the distance between extreme layers of column reinforcement, b_j is the effective joint width and v_d is the normalized axial load ratio exactly above the joint. Coefficient n accounts for the reduction in strength of the diagonal compression strut forming in the joint, due to diagonal tension cracking.

5. Conclusions

A performance-based framework for designing retrofits for RC buildings using FRP materials was developed and presented in detail. Consistent models and approaches were weaved together to cover the entire range of design considerations, including global stiffness requirements, strength hierarchies to satisfy capacity design objectives in the retrofitted structure and deformation capacities of individual structural members to meet the performance objectives of the retrofit. Interestingly, it was shown that all performance indexes may be linked to measures of the lateral confining stress exerted by FRP jackets on the encased members; however, the supporting database of experiments and attendant calibrated confinement models are particularly biased, having been obtained solely from uniaxial confinement tests with or without embedded reinforcement. It was found that information is scarce regarding the performance and deformation capacity of members retrofitted with FRP when these are subjected to cyclic moment-shear-axial load reversals, the result being some over-conservatism when defining design values for these parameters. Thus, rotation capacity, improved anchorage of confined reinforcement and shear strength of retrofitted structural members are all subjects that warrant further investigation. Detailing the anchorage of FRP strips and jackets for beam-column joint retrofits is another open issue which, although addressed analytically and with design expressions in the present work, will require particular attention during implementation in order to secure efficient confinement of the diagonal compressive struts that support the function of moment and shear transfer in this type of element.

6. References

[1] EN 1998-1 (2004), Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. European Committee for Standardization (CEN), Brussels.

[2] Thermou, G.E., Pantazopoulou, S.J., and Elnashai, A.S.: Design methodology for seismic upgrading of substandard RC structures. Journal of Earthquake Engineering, 11 (4), 2007, pp. 582-606.

[3] Thermou, G.E., Pantazopoulou, S.J., and Elnashai, A.S.: Global interventions for seismic upgrading of substandard R.C. buildings. Journal of Structural Engineering, ASCE, 138(3), 2012, pp. 387-401.

[4] Thermou, G.E., and S. J. Pantazopoulou.: Criteria and methods for redesign and retrofit of old structures. Proceedings, 10th U.S. National Conference on Earthquake Engineering, Frontiers of Earthquake Engineering, Anchorage, Alaska, 2014.

[5] EN 1998-3 (2005), Eurocode 8: Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings. European Committee for Standardization (CEN), Brussels.

[6] KAN.EPE 2013: Greek Seismic Assessment and Retrofit Code. OASP (in English, GRECO 2014).

[7] Pantazopoulou, S. J., Guadagnini, M., D'Antino, T., Lignola, G. P., Napoli, A., Realfonzo, R., Pellegrino, C., Prota, A., Balafas, I., Tastani, S., Bournas, D.: Confinement of RC elements by means of EBR-FRP Systems, Chapter 5. In: Design Procedures for the Use of Composites in Strengthening of Reinforced Concrete Structures – State of the Art Report of RILEM TC 234-DUC, (eds: Pellegrino, C., Sena-Cruz, J.), Springer, RILEM STAR Book Series, 2015.

[8] Tastani, S. P., Pantazopoulou, S. J., Zdoumba, D., Plakantaras, V., Akritidis, E.: Limitations of FRP jacketing in confining old-type reinforced concrete members in axial compression. ASCE Journal of Composites for Construction, 10(1), 2006, pp. 13–25 (corrigenda, Oct 2013).

[9] ACI 440.2R-08 (2008): Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, ISBN 9780870312854.

[10] fib Bulletin No. 35: Retrofitting of concrete structures by externally bonded FRPs, with emphasis on seismic applications, 2006, ISBN 978-2-88394-075-8.

[11] Richart, F., Brandtzaeg, A., Brown, R.: Study of failure of concrete under combined compressive stresses. Engineering Experiment Station, No. 185, Univ. of Illinois, Urbana, Ill., 1928.

[12] EN 1992-1-1 (2004), Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. European Committee for Standardization (EN), Brussels.

[13] Tureyen, A. K., Frosch, R. J.: Concrete Shear Strength: Another Perspective. ACI Structural Journal, vol. 100, No. 5, Sept/Oct 2003, pp. 609–615.

[14] Tastani, S. P., Pantazopoulou, S. J.: Detailing procedures for seismic rehabilitation of reinforced concrete members with fiber reinforced polymers. Elsevier Engineering Structures, 30(2), 2008, pp. 450–461.

[15] fib CEB-FIP (2010): fib Model Code 2010 – Final draft, vol. 1, Bulletin No. 65, International Federation for Structural Concrete, Lausanne, Switzerland.

[16] Tastani, S. P., Balafas, I., Dervisis, A., Pantazopoulou, S. J.: Effect of Core Compaction on Deformation Capacity of FRP-Jacketed Concrete. Construction & Building Materials, Elsevier, 47, 2013, pp. 1078–1092.

[17] fib Bulletin No. 24: Deformation Capacity of Old-Components, Chapter 4, State of the Art Report “Seismic Assessment and Retrofit of Reinforced Concrete Buildings”, fib Seismic Commission (C7-WG1), 2003.

[18] Pantazopoulou, S.J., Bonacci, J.: Consideration of Questions about Beam-Column Joints. ACI Structural Journal, 89(1), 1992, pp. 27-36.

[19] ACI-ASCE 352R-0-2 (2002): Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures, ISBN 9780870310874.

[20] Antonopoulos, C. P., Triantafillou, T. C.: Experimental Investigation of FRP-Strengthened RC Beam-Column Joints. ASCE Journal of Composites for Construction, 7(1), 2003, pp. 39–49.