Proceedings of the 8th U.S. National Conference on Earthquake Engineering
April 18-22, 2006, San Francisco, California, USA
Paper No. 232

FIBER REINFORCED POLYMERS IN SEISMIC UPGRADING OF EXISTING REINFORCED CONCRETE STRUCTURES

S. P. Tastani[1] and S. J. Pantazopoulou[2]

ABSTRACT

Externally-bonded Fiber Reinforced Polymers (FRPs) when used as jackets in reinforced concrete (r.c.) members are effective in increasing shear, lap-splice strength and deformation capacity. For this reason they are currently used extensively as a fast remedy in earthquake-damaged structures, or to strengthen under-designed structures in areas of high seismicity. FRP jackets impart little or no stiffness to the encased element, whereas experimental evidence indicates that the demand on bar anchorages increases after jacketing of the adjacent plastic hinges near supports. Under excessive rotation demand the jackets, being susceptible to stress concentrations, are at risk of rupture due to buckling of primary reinforcement, when the embedded stirrups are very sparse (substandard detailing). These performance issues are explored in the paper through a combined evaluation of published experimental evidence and simple mechanistic constructs that highlight the mechanical contribution of the jacket to the various failure modes of an r.c. element. Dependable deformation capacity at yield and ultimate and the various strength components are examined through collective evaluation of available tests and design lower bound expressions are derived. Criteria that should be considered as part of the upgrading strategy when FRP jacketing is used so as to control the deformation demand of the structure are also discussed.

Introduction

A common deficiency of many of the r.c. structures that get damaged during earthquakes is intrinsic lack of stiffness (e.g. in soft storey formations), combined with limited deformation capacity of the individual structural elements owing to non-ductile, old type detailing. Because excessive displacement brings out all the potential problems of an inadequate design or construction, it is necessary in repair/strengthening schemes to target for reduced displacement demand, by increasing the lateral stiffness of the structure. For this reason, global interventions need be accompanied by targeted local measures so as to increase the dependable deformation capacity of the individual members beyond the deformation demand.

FRP jacketing of damaged or under-designed r.c. members is considered a local

[1] Ph.D. Candidate, Department of Civil Engineering, Demokritus University of Thrace, Xanthi 67100, Greece; tel/fax +30-25410-79639, stastani@civil.duth.gr
[2] Professor, Department of Civil Engrg., Demokritus University of Thrace, Xanthi 67100, Greece; tel/fax: +30-25410-79639, pantaz@civil.duth.gr

intervention as it has negligible influence of the lateral stiffness of the jacketed member and cannot effect changes on the response characteristics of the overall structure. As such jackets may successfully upgrade shear strength, lap splice strength, and the overall deformation capacity of individual members. Unless the transverse jackets serve to mitigate premature local failures that would otherwise limit the pre-yield response, secant to yield stiffness remains unaltered by the repair. Among their limitations is that they are susceptible to rupture at points of localized deformation demand. When used in critical zones of members undergoing excessive deformation demand (e.g., in flexible structures) they effectively reduce shear cracking in the plastic hinge regions, driving all deformation to occur within few flexural cracks near the support. Due to confinement of the compressive zone high strain demands develop in the tension reinforcement at the critical section. This imposes an increased demand for bar development capacity that cannot always be met by the anchorage in substandard construction (Tastani and Pantazopoulou 2003).

For these reasons FRP jacketing need be explicitly embedded in the context of the integrated global strategy of seismic rehabilitation of the structure, where, survivability of the upgraded structural system depends on the magnitude of the lateral drift. In the paper the confining pressure generated by the FRP, its effectiveness, and the design effective strain that may be used in calculations are considered. The derived expressions are used to obtain the various strength terms and the deformation capacity of FRP jacketed r.c. members. Finally considerations about drift control that need be combined with the FRP jacketing are discussed.

Mechanical effects of FRP jacketing on r.c. members

As a method of seismic upgrading FRP jacketing may achieve at best a marginal increase in the flexural strength of the members without influencing their initial stiffness, up to full exploitation of the deformation capacity of longitudinal reinforcement. The actual increase in flexural strength and deformation capacity effected through FRP jackets may be quantified by approaches similar to those used for conventionally r.c. members (ACI 440.2R-02, 2002). When functioning as transverse reinforcement, the jacket is mobilized passively in tension when the encased concrete dilates laterally, owing either to excessive compression, buckling of longitudinal reinforcement, web cracking due to shear, or cover splitting along anchorages or lap splices. In all cases the jacket acts as passive confinement by restraining dilation, thereby enhancing the deformation capacity of the encased member. Depending on the mechanical function of the jacket, either the transverse pressure in the direction normal to the plane of splitting, $\sigma_{lat,y}$, or the average transverse pressure in two orthogonal directions σ_{lat}^{ave}, may be needed to quantify the mechanical function of pressure on resistance. In any given direction of action y, the total transverse pressure, $\sigma_{lat,y}$, comprises contributions of the FRP jacket and the occasional embedded stirrups:

$$\sigma_{lat,y} = \sigma^{f}_{lat,y} + \sigma^{st}_{lat,y} = 2k_{f,y}\, nt_f\, E_f\, \varepsilon^{eff}_{f} / b_y + k_{st,y} A_{st} f_{y,st} /(s\, b_y) \qquad (1)$$

Parameters $k_{f,y}$ and $k_{st,y}$ are the effectiveness coefficients for the two transverse confining systems, ε_f^{eff} is the effective tensile strain that develops in the jacket near failure (which may occur either by debonding or by rupture, whichever prevails), E_f, n, t_f are the elastic modulus, number and thickness of the FRP plies, b is the cross-section width at the splitting plane (orthogonal to the applied jacket force), A_{st} is the total cross sectional area of stirrup legs crossing the splitting plane provided by a single stirrup layer, s the longitudinal spacing of stirrups and $f_{y,st}$ their yield stress.

Effectiveness coefficients for the various response mechanisms

The effectiveness coefficients $k_{f,y}$ and $k_{st,y}$ in Eq. 1 for the two transverse reinforcement systems depend upon the function of $\sigma_{lat,y}$ in the response mechanism considered:

(1) For shear strengthening, $k_{f,y}$ depends on the development capacity of the jacket anchorage. Consider a shear crack extending at 45° along the web height d_f (Fig. 1a); transverse pressure develops in the y-direction (along the web height); b is the web width in Eq. 1. The design strain of the jacket will develop at the critical section, which is at the point of intersection with the crack. If the jacket is adequately closed, then $k_{f,y}=1$ (Fig. 1b). If, owing to cross-sectional shape of the member it is not possible to wrap the jacket around the section, thus terminating it on the web near the compression zone, (e.g. in T-beams, Fig.1a), then, only those fibers that have sufficient anchorage length L_f beyond the crack may be considered effective as shear reinforcement. In this case, the effectiveness coefficient is $k_{f,y} = (d_f - L_f)/d_f < 1$. ACI 440.2R-02 (2002) proposes methods for calculating L_f (also discussed in the following sections). In direct analogy, for open shear links, $k_{st,y}=0.5$ (FIB Bulletin 24, 2003) whereas $k_{st,y}=1$ for well anchored closed stirrups.

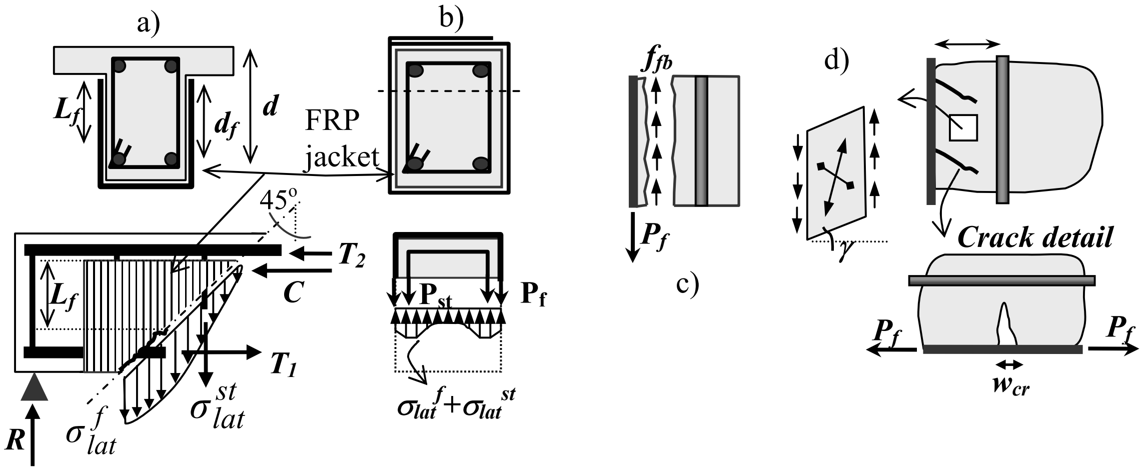

Figure 1. a) Free body diagram of FRP wrapped member at a shear crack plane, b) stress state of FRP strengthened rectangular cross section, failure of open jacket c) by delaminating from concrete and b) by diagonal tension of the cover.

(2) When strengthening for confinement, the confining pressure is the average value σ_{lat}^{ave} obtained from Eq. 1 in the two principal directions of the cross section as $\sigma_{lat\text{-}x}$ and $\sigma_{lat\text{-}y}$:

$$\sigma_{lat}^{ave} = 0.5\left(\sigma_{lat,y} + \sigma_{lat,x}\right) = 0.5\left(k_f^c\, \rho_{fv}\, E_f\, \varepsilon_f^{eff} + k_{st}^c\, \rho_{sv}\, f_{y,st}\right) \tag{2}$$

where, ρ_{fv} and ρ_{sv} are the volumetric ratios of FRP and stirrup reinforcement. The familiar expression for k_f^c approximates the volume fraction of core concrete that is effectively restrained (similar to the approach used to evaluate confinement effectiveness of stirrups k_{st}^c (Priestley et al. 1996). Therefore, $k_f^c=1-(b'^2+d'^2)/[3A_g(1-\rho_s)]$, where A_g is the gross cross section of the element, ρ_s is the ratio of longitudinal reinforcement, b' and d' the straight sides of the rectangular cross section encased by the jacket after chamfering the corners (ACI 440.2R-02, 2002, FIB Bulletin 14, 2001). For a cross section with a side aspect ratio of 3, the confinement effectiveness coefficient becomes negligible ($k_f^c \approx 0$), whereas for square and circular sections $k_f^c \approx 0.5$ and 1, respectively. The underside is that the primary function of FRP wrapping in a cross section with a large aspect ratio would be to increase its *lateral load* resistance rather than its axial load strength.

(3) In upgrading bar anchorages/lap splices by effecting transverse restraint through jacketing, the effectiveness coefficients are taken as follows: for the FRP jacket $k_{f,y}^{anch} = 1$ whereas $k_{st,y}^{anch} = 1/3$ for stirrups to account for their spacing along the anchorage length. The splitting plane may occur either starting from an anchored bar and extending towards the nearest free surface, or may cross several bars. Depending on the direction of splitting, the restraining transverse pressure (and the associated terms in Eq. 1) may be in either of the two principal directions of the section.

Derivation of the effective strain, ε_f^{eff}

The effective strain ε_f^{eff} in Eqs. 1, 2 is the usable tensile strain capacity of the FRP jacket and is only a fraction of the nominal deformation capacity of the material ($\varepsilon_{fu,d}$). The value of ε_f^{eff} depends on the mode of failure of the bonded layer that in turn is controlled by the bond strength of the substrate. In the following, ε_f^{eff} is defined depending on the jacket geometry (open or closed) and the likely mode of failure of the wrap. A jacket is considered open if it cannot be fully wrapped around the cross section of the member (as in the web of monolithic floor beams).

In open jackets the bonding substrate in the anchorage is the concrete cover. Failure may occur either by debonding of the FRP ply or by diagonal tension failure of the cover layer (Figs. 1c,d). Viability of the jacket depends on the low resistance of the cover concrete to direct tension. If debonding is suppressed by mechanical anchorage in the ends, the next likely mode of failure is diagonal cracking of the cover near pre-existing cracks. Note that the FRP sheet develops forces in tension when crossing cracks in concrete by shearing the substrate. Rather than slipping relative to its surroundings, the composite jacket drags the concrete cover in shear distortion so as to bridge the crack width, leading to premature diagonal tension failure of the concrete cover prior to realization of the jacket's tensile strength. This mode of failure is controlled by the width w_{cr}, of the cracks developing in the strengthened member under the wrap (Fig. 1d). Assuming a linear variation of jacket stresses and considering force equilibrium over the development length L_f, the critical strain ε_f^{eff} of the FRP layer at the crack and the required L_f may be defined as,

$$\varepsilon_f^{eff} = [w_{cr}\, f_{fb,d} / (2E_f\, n\, t_f)]^{0.5} \;\; ; \;\; L_f = (2w_{cr}\, E_f\, n\, t_f / f_{fb,d})^{0.5} \tag{3}$$

where f_{fb} the bond stress distribution over L_f, and f_{fbd} the average design value taken here equal to the tensile strength of the concrete, f_t'. Cover shear distortion is $\gamma = 0.5w_{cr} / c$, where c the cover thickness. This becomes prohibitively large for crack widths in excess of $0.3mm$. Thus, the results of Eq. 3 are capped by this limiting value for w_{cr}; the larger the axial stiffness of the FRP sheet, the lower the strain that may be developed over the sheet anchorage, whereas the usable fraction of its strain capacity is limited by cracking of the substrate.

In closed jackets ε_f^{eff} is calculated in a similar manner taking into account that the weak link is the adhesive resin ($f_{gl,d}$ is the shear strength of the glue at the stage of plastification) stressed in shear along the overlap length, L_f of the exterior layer. The strength of the bonded system is controlled by the limiting slip $s_{gl,u}$ of the glue at shear failure as,

$$\varepsilon_f^{eff} = [f_{gl,d}\, s_{u,gl} / (E_f\, t_f)]^{0.5} ; \;\; L_f = (s_{u,gl}\, E_f\, t_f \,/\, f_{gl,d})^{0.5} \tag{4}$$

ACI440.2R-02 (2002) proposes $\varepsilon_f^{eff} = 0.004$ and $0.75\varepsilon_{fu,d}$ for open and closed jackets, respectively. Results from compression tests with closed FRP jackets (Chaallal 2003) indicate that the material factor of 0.75 is rather high when used with rectangular cross sections due to the jacket's susceptibility to local rupture at the corners even after chamfering. A value of 0.5 has been found more conservative in this case (i.e., $\varepsilon_f^{eff}=0.5\varepsilon_{fu,d}$, Tastani and Pantazopoulou 2003).

Strength assessment of FRP rehabilitated r.c. members

In redesigning a substandard r.c. element for seismic resistance objective is to mitigate all other failure modes except for flexural, which is the least undesirable. Design forces must satisfy the following qualitative relationship:

$$V_{u,lim} = min\{V_{iflex}, V_{shear}, V_{anch}, V_{buckl}\} \qquad (5)$$

where, $V_{iflex}=M_u/L_s$ is the seismic shear force required to develop the ideal flexural resistance of the member, L_s is the shear span, V_{shear} is the nominal shear resistance, V_{anch} is the shear force when the anchorage / lap-splice reach their development capacity and V_{buckl} is the shear force when compression bars reach instantaneous buckling conditions at the critical section. The strength components in Eq. 5 may be estimated from variables of σ_{lat} calculated by Eqs. 1 and 2.

Ideal flexural capacity calculations

Flexural resistance is influenced by the concrete strength increase owing to confinement, and containment of the cover that would otherwise spall-off at ultimate. The confined concrete strength f_{cc}' and the corresponding strain ε_{cc}, in the compression zone of the encased cross section is calculated using a modified version of the classical confinement model of Richart (1928):

$$f_{cc}' = f_c' + 3\sigma_{lat}^{ave} \quad ; \quad \varepsilon_{cc} = \varepsilon_{co}\left(1 + 5\left(f_{cc}' / f_c' - 1\right)\right) \qquad (6)$$

By substitution of Eq. 2 in 6, and assuming $\varepsilon_{co}=0.002$ (strain at peak stress of unconfined concrete) Eq. 7 is obtained. The failure strain $\varepsilon_{cc,u}$ corresponding to a compression strength reduction in excess of 15% is obtained from Eq. 8 as a lower bound expression (Fib Bulletin 24, 2003). For closed jackets the ε_f^{eff} is taken as $0.5\varepsilon_{fu,d}$.

$$\begin{aligned} f_{cc}' &= f_c' + 1.5\left(k_f^c \rho_{fv} E_f \varepsilon_f^{eff} + k_{st}^c \rho_{sv} f_{y,st}\right) \\ \varepsilon_{cc} &= 0.002 + 0.015\left(k_f^c \rho_{fv} E_f \varepsilon_f^{eff} + k_{st}^c \rho_{sv} f_{y,st}\right) / f_c' \end{aligned} \qquad (7)$$

$$\varepsilon_{cc,u} = \varepsilon_{c,u} + 0.075\left(\left(k_f^c \rho_{fv} E_f \varepsilon_f^{eff} + k_{st}^c \rho_{sv} f_{y,st}\right) / f_c' - 0.1\right) \geq \varepsilon_{c,u}\,; \quad 0.003 \leq \varepsilon_{c,u} \leq 0.004 \qquad (8)$$

Shear strength calculations

Shear resistance of r.c. members subjected to displacement reversals degrades with the number of cycles and the magnitude of imposed displacement ductility, owing to breakdown of concrete's tensile and compressive resistance with increasing crack widths. Strength reduction is accounted for through a ductility dependent softening coefficient λ (Moehle 2002) as:

$$V_n(\mu_\Delta) = \lambda\left(V_s + V_c\right);\ V_s = \sigma_{lat,y}^{st}\, bd;\ V_c = 2\sqrt{f_c'}\, bd;\ \lambda = 1.15 - 0.075\mu_\Delta;\ \lambda \subseteq (0.7, 1) \qquad (9)$$

where μ_Δ the imposed displacement ductility. The transverse pressure $\sigma_{lat,y}^{st}$ contributed by any dependable stirrups is calculated from the second term of Eq. 1. In redesigning substandard r.c. members for shear resistance, Eq. 9 need be used both in assessing the residual $V_{n\text{-}res}$ prior to the FRP jacketing intervention but also in evaluating the post-upgrading resistance as,

$$V_{n-res}(q_{old}) = \lambda(q_{old})(V_s + V_c)$$
$$V_n(q_{new}) = min\{\lambda(q_{old}), \lambda(q_{new})\}(V_s + V_c) + V_w^f \quad ; \quad V_w^f = \sigma_{lat,y}^f bh \tag{10}$$

where q is the behavior index (or R, FEMA 273 1997) and $\sigma_{lat,y}{}^f$ is the transverse pressure in concrete owing to the jacket in the direction of lateral sway (Eq. 1). The shear strength of the jacketed member is the sum of the jacket contribution, $V_w{}^f$, and the contribution of the existing mechanisms, namely concrete V_c, and transverse steel V_s. In deriving Eq. 10, it has been assumed that the target μ_Δ used in the redesign of the member is equal to the behavior index, q_{new} (or R_{new}). Equation 10 recognizes that the existing mechanisms may have sustained damage during previous loading. For this reason residual rather than the full contributions of core concrete and web reinforcement are considered, by taking the minimum value of λ for these terms, based on the ductility demand either suffered during previous events, or used as target value for redesign. Based on experiments the softening coefficient is not applied on the $V_w{}^f$ as diagonal cracking is suppressed by the application of the jacket (Tastani and Pantazopoulou 2003).

Anchorage / lap-splice strength calculations

A direct consequence of member upgrading through FRP jacketing is to increase the deformation demand in the lap-splice / anchorage regions. Frequent bond related problems in existing construction include lap splicing of the main bars immediately above the floor level in the anticipated plastic hinge regions with sparse transverse reinforcement, use of smooth bars and small development lengths. To remedy anchorage problems, FRP jackets are wrapped orthogonal to the anticipated splitting cracks. The development capacity of a given anchorage length L_b is calculated from: $F=\mu \cdot \sigma_{lat} \cdot \pi D_b \cdot L_b$, where μ is the coefficient of friction at the steel-concrete interface and σ_{lat} the pressure exerted upon the lateral surface of the bar by the cover, transverse stirrups and FRP jacket. The average bond stress f_b is given by:

$$f_{b,d} = \mu\left(cf_t' + k_{st}^{anch} A_{st} f_{y,st} /(N_b s) + 2k_f^{anch} n t_f E_f \varepsilon_f^{eff} / N_b\right)/(\pi D_b) \tag{11}$$

N_b is the number of bars (or pairs of spliced bars) laterally restrained by the transverse pressure. The value of $\varepsilon_f{}^{eff}$ used in Eq. 11, is the surface strain value associated with attainment of bond strength along the bar, and it is in the order of *0.0015-0.002* (Priestley 1996). The lateral force in Eq. 5 required to develop the anchorage strength in the upgraded element is referred to as V_{anch}.

Resistance to longitudinal bar buckling in FRP-wrapped r.c. elements

In r.c. members with substandard details, attainment of flexural failure is often precluded by buckling of compression reinforcement owing to the large unsupported length of the bars: stirrup spacing in the range of *200mm - 300mm* is not uncommon in old construction. At this distance, the spacing to bar diameter ratio s/D_b is *10 - 15* for a D_b*=20mm* bar, much in excess of the upper limit of *6 - 8* recommended for high to moderate ductility structures (FIB Bulletin 24 2003). FRP jackets are susceptible to rupture when bar buckling is imminent. For plastic hinge regions with severe shear demand sideways bar buckling is the likely failure pattern. The critical buckling stress $f_{s,crit}$ is related to s/D_b through: $s/D_b=0.785(E_r/f_{s,crit})^{1/2}$ where E_r is the double-modulus of steel at the stress level considered (FIB Bulletin 24 2003). From this relationship, given the full stress-strain diagram of the bar, the limiting strain-ductility curve ($\mu_{\varepsilon c}=\varepsilon_{s,crit}/\varepsilon_y$) may be plotted as a function of s/D_b. $\varepsilon_{s,crit}$ is the axial strain at the onset of instability for the given s/D_b. The example in Fig. 2 refers to steel with yield stress f_y*=400MPa,* initial strain hardening slope of *30GPa* and a yield plateau to a strain of *0.005*.

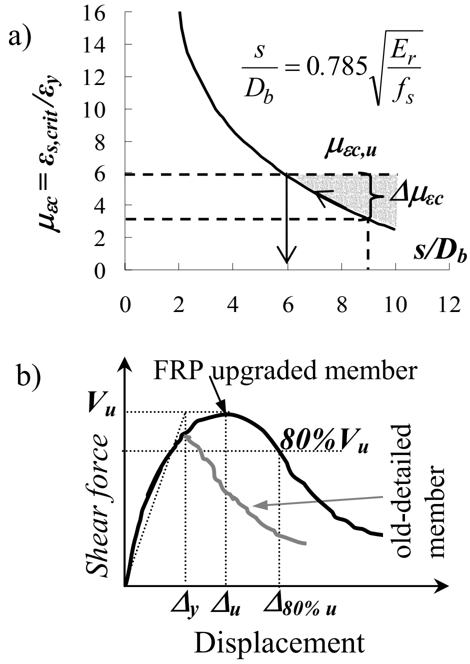

Figure 2. a) Relationship between $\mu_{\varepsilon c}$ and s/D_b ratio b) Definition of deformation indices.

Buckling of any individual bar segment is controlled by its strain-ductility curve, unless the dependable deformation capacity of encased concrete, $\varepsilon_{cc,u}$ (from Eq. 8) exceeds the $\varepsilon_{s,crit}$ value corresponding to the available s/D_b. In that case redistribution between the compressed bars at incipient buckling and the encased concrete is possible, thereby postponing buckling to occur at a higher strain level. Therefore, by increasing the strain capacity of concrete through jacketing to levels higher than $\varepsilon_{s,crit}$, the effective s/D_b is reduced, as depicted in Fig. 2. The dependable strain ductility of compression reinforcement is:

$$\mu_{\varepsilon c} = max\left\{\varepsilon_{s,crit} / \varepsilon_y , \varepsilon_{s,cu} / \varepsilon_y\right\} \tag{12}$$

The lateral force in Eq. 5 corresponding to the development of buckling strain $\varepsilon_{s,crit}$ in the compression reinforcement is $V_{buckl} = M_{buckl}/L_s$ where M_{buckl} is obtained from equilibrium of moments in the critical section. In detailing the jacket it is important to ensure that the target displacement ductility of the member after upgrading, $\mu_{\Delta req}=\Delta_u^{target}/\Delta_y$, may be attained prior to buckling of primary reinforcement. To check this the resulting curvature ductility demand $\mu_{\phi,req}$ (= $\phi_{u,req}/\phi_y$) in the plastic hinge region of the member is obtained from $\mu_{\Delta,\rho\varepsilon\theta}$:

$$\mu_{\Delta,req} = 1 + 3\left(\mu_{\phi,req} - 1\right)\left(1 - 0.5 l_p / L_s\right) l_p / L_s ; \qquad l_p = 0.08 L_s + 0.022 f_y D_b \tag{13}$$

Lacking a better approximation the length of plastic hinge l_p in Eq. 13 is taken as per established expressions (Priestley 1996). This may require revision for FRP-jacketed members where the contribution of pullout is significant.

From $\mu_{\phi,req}$ the compression strain ductility demand, $\mu_{\varepsilon c,req}$, of compression reinforcement may be estimated and compared to the dependable value resulting from Eq. 12. For example, for symmetric displacement reversals, $\mu_{\varepsilon c,req}=1.1\mu_{\phi req}-1$ (FIB Bulletin 24 2003). If the jacket layers required to satisfy this criterion are excessive, then it may be advisable to opt for increased storey stiffness so as to effect a reduction in the displacement ductility demand $\mu_{\Delta,req}$.

Deformation capacity assessment for FRP encased members

Results from over seventy published tests are used to assess the response of FRP jacketed r.c. prismatic members under reversed cyclic loading (Tastani and Pantazopoulou, 2003). For each specimen the experimental load – displacement envelope is used to define yield and ultimate displacement and lateral load strength, as illustrated in Fig. 2b: the characteristic points in the envelope correspond to *80%* of the peak load, V_u. Figure 3a plots experimental estimates of yield displacement (defined as per Fig. 2b) after being normalized by the calculated result from two popular models: 1) according to classical mechanics ($\Delta_y=\phi_y L_s^2/3$) and 2) including the

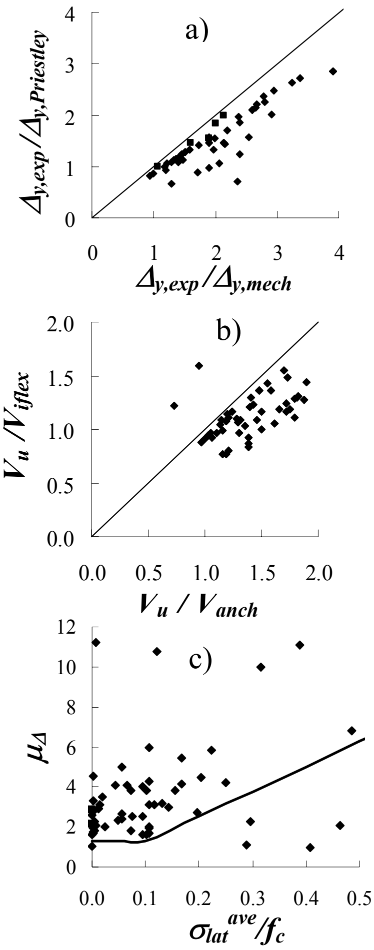

Figure 3. a) Comparison between experimental and analytical yield displacements, b) Normalized experimental strengths, c) Correlation of Eq. 14 with test data.

flexural-slip component of the yield-displacement as proposed by Priestley (1996) ($\Delta_y=\phi_y(L_s+0.022f_yD_b)^2/3$). Clearly, both analytical estimates fall well below the experimental value for yield displacement; the worse estimates resulting from the classical model. Underestimation of yield displacement indicates that the actual slip component is larger than calculated.

Figure 3b plots the reported (experimental) strength V_u of each upgraded member normalized by the V_{iflex} on the y-axis, and by the calculated V_{anch} on the x-axis. Again, most of the experimental points fall below the equal value line underscoring the localization of deformation demand that occurs in the anchorage, which, after jacketing, becomes the weak link of the upgraded member. The tests confirm that it is possible to suppress all premature modes of failure so that flexural yielding may prevail, through FRP jacketing. This is manifested by the ductility in the load-displacement curve of the upgraded member. Here, tests results were correlated collectively with an empirical lower bound expression for the available displacement ductility, μ_Δ as a function of transverse confining pressure σ_{lat}^{ave} ($\mu_\Delta \geq 1.3$ for poorly detailed members):

$$\mu_\Delta = 1.3 + 12.4\left(\frac{k_f^c \rho_{fv} E_f \varepsilon_f^{eff} + k_{st}^c \rho_{sv} f_{y,st}}{2f_c^{'}} - 0.1\right) \qquad (14)$$

Figure 3c compares the experimental values with the analytical estimates of Eq. 14 obtained using $\varepsilon_f^{eff}=\varepsilon_{fu,d}$. Experimental points lying below the lower bound curve correspond to repair cases where the postrepair yield displacement used to quantify dependable ductility from the load displacement envelope was the apparent value, markedly greater than the true displacement at first steel yielding.

Conclusions: Global considerations when using FRP jackets for seismic upgrades

Most of the strength terms in Eq. 5 depend on the anticipated deformation demand in the member after repair. Once the strength of the weakest mechanism is exhausted, localization of deformation is expected to occur in that particular behavior mode, which becomes the fuse of member response upon increased deformation demand. Collective evaluation of the available experimental evidence demonstrated that jacketing of deficient r.c. members increases their nominal deformation capacity, but imposes a more severe demand upon the anchorage. A large component of the drift attained in jacketed column tests is due to lumped rotation owing to slippage of longitudinal bars from the support. Bar buckling is postponed in jacketed members as the compression strain capacity of the encased concrete core is increased, thereby enabling redistribution of bar stresses to concrete upon attainment of bar instability (Tastani 2005).

Recalling that FRP-jackets cannot increase member stiffness, it is useful to employ pertinent criteria in order to identify whether the upgrading measures need to involve storey stiffening along with local interventions through jacketing. Relevant response indices that may be used as diagnostic tools in assessing the global adequacy of the structural morphology are the magnitude of the fundamental structural period, drift at yield of the vertical element and the fundamental translational mode-shape of the structure that may reveal the existence of soft-storeys. In designing the upgrading scheme, the seismic demand need be determined in displacement terms. Prerequisite is idealization of the structure as an equivalent single degree of freedom system (ESDOF) through a selected empirical approximation of the predominant shape of lateral vibration and calculation of the corresponding stiffness (secant to yield).

For immediate results the ESDOF properties may be used with the YPS (Yield Point Spectra) of the design earthquake in order to evaluate the anticipated displacement demand and corresponding displacement ductility (Aschheim and Black 2000). Assuming the equal displacement rule the elastic spectral displacement is also the target displacement of the inelastic system: $\Delta_u=S_d$. For a preliminary assessment of the suitability of the upgrading scheme it is acceptable to adopt an upper limit of *2%* for the lateral drift of the structure at the design earthquake; for larger displacement levels second order effects that are usually not efficiently mitigated by concrete encasement need be explicitly addressed in the upgrading strategy. The critical displacement limit, $\Delta_{u,crit}=2\%H$ (where H the building height) corresponds to a spectral limit of Δ_{crit}^*. The vertical line in the ADRS drawn at displacement Δ_{crit}^* defines a design boundary. Acceptable solutions are to the left of the vertical line and above the YPS associated with the limiting ductility of the system. By also implementing stiffening schemes in the structure, the radial line is effectively rotated counterclockwise in the ADRS, thereby reducing the design value of Δ_u, with an attendant mild increase in the required V_y^*. Note that a larger increase in capacity may be required to also reduce the target μ value. The final step in the design is to apply the analytical expressions for each of the ultimate limit states discussed as per the qualitative Eq. 5, thereby linking the target indices of behavior to jacket dimensions.

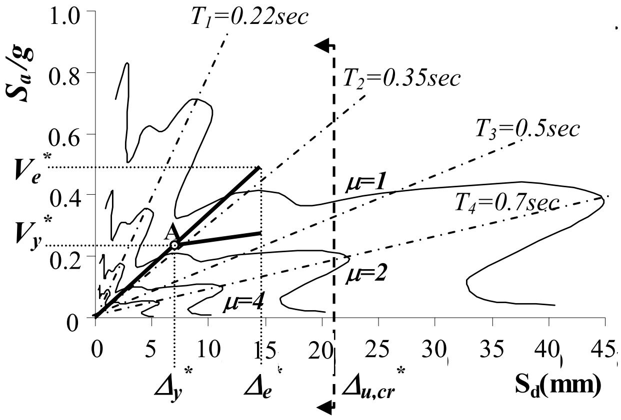

Figure 4. Isoductile YPS: definition of values
(* identifies the ESDOF system).

Example: design of jacketing for a substandard r.c. column

The procedures described can be used to detail the jacket required to upgrade the seismic resistance of a substandard element to a desired level of displacement ductility. Consider an old-type reinforced concrete column in double bending, having a cross section of *400x700mm* , clear height of *5m*, symmetric tension and compression reinforcement ratios of *0.90%*. Nominal material strengths are $f_c'=25MPa$, $f_{y,s}=400MPa$ (main bars) and $f_{y,st}=220MPa$ (stirrups). Rectangular stirrups (diameter of 6mm) spaced at $s=300mm$ are provided ($s/D_b=50$). Initial column shear resistance was calculated as $V_n= 30\%V_c+V_s=51.60kN$, the ideal flexural capacity at column yielding and the corresponding displacement as $V_{iflex}=103.4kN$, and $\Delta_y= 36mm$. Thus, shear failure would prevail at a displacement of $\Delta_{sh}=18mm$, well before flexural yielding.

Consider an upgrading scheme with CFRP jacketing so as to enhance member lateral drift capacity to *2*%, corresponding to displacement ductility of $\mu_\Delta=2.5$ (wraps with $t_f=0.13mm$, $f_f=3500MPa$, $\varepsilon_{fu,d}=0.015$, and $E_f=230GPa$). The jacket is detailed using the procedures described in the preceding. Results are listed collectively in Table 1 for each design action (shear, anchorage, rebar buckling). Jackets are fully wrapped (closed), thus, the effective strain used in the calculations is $\varepsilon_f^{eff}=50\%\varepsilon_{fu,d}$ for shear and rebar buckling. In Table 1, each mode of failure considered leads to a different number of required layers. The more severe requirement in terms of jacket thickness is associated with the anchorage, oversupplying the demands of the other response mechanisms. Note that whereas shear was the likely mode of failure in the initial state of the member, theoretically a single jacket layer would suffice to upgrade column shear strength to levels exceeding the ideal flexural strength.

Table 1. Required jacket layers for each design action (n_f to be rounded off to next integer).

Confinement for $\mu_{\Delta req}=2.5$ $l_p \approx 350\ mm$ (Priestley '96)				Bar Buckling ($\varepsilon_{cc,u}=0.011$, Eq. 8)			Shear in-crease		Lap-Splice above Base, Eq. 11
σ_{lat}^{ave}/f_c	k_f^c	$\rho_{fv}\%$ Eq.14	n_f	$\mu_{\varepsilon c,avail}$ Eq.12	$\mu_{\phi,req}$ Eq. 13	$\mu_{\varepsilon c,req}=1.1\mu_{\varphi,req}-1$	V_w^f (kN)	n_f	avail. $L_b=20D_b$
0.2	*0.37*	*0.8*	7.5	$\varepsilon_{cc,u}/\varepsilon_y=5.6$	4.6	4.1<*5.6*	67.3	0.4	$f_{bd}^{avail}=3\ MPa$ $f_{bd}^{req}=4.35\ MPa$ $\sigma_{lat}^f/N_b=110\ (N/m)$, $n_f=7.4$ (ε_f^{eff}=0.13%)

References

Tastani S. and Pantazopoulou S.J., 2003. Strength and deformation capacity of brittle r.c. members jacketed with frp wraps. *CD-ROM Proceedings of the fib-Symposium on Concrete Structures in Seismic Regions*, Technical Chamber of Greece (Ed.), Athens – Greece.

American Concrete Institute, 2002. *Guide for the design and construction of externally bonded frp systems for strengthening concrete structures (ACI 440.2R-02)*. Farmington Hills, Michigan.

International Federation for Concrete, 2003. *Seismic assessment and retrofit or r.c. buildings (FIB Bulletin 24)*. Report by Task Group 7.1, Chapter 4, , Lausanne Switzerland.

Priestley M., Seible F., Calvi M., 1996. *Seismic design and retrofit of bridges*. J. Wiley & Sons Inc., N.York.

International Federation for Concrete, 2001. *Externally bonded FRP reinforcement for r.c. structures (FIB Bulletin 14)*. Report by Task Group 9.3, Lausanne Switzerland.

Brẽna S., Wood S. and Kreger M., 2003. Full-scale tests of bridge components strengthened using carbon FRP composites. *ACI Structural Journal* 100 (6), 775-784.

Chaallal O., Shahawy M. and Hassan M., 2003. Performance of axially loaded short rectangular columns strengthened with carbon FRP wrapping. *ASCE J. Comp. for Construction* 7 (3), 200-208.

Richart F., Brandtzaeg A. and Brown R., 1928. A study of the failure of concrete under combined compressive stresses. *Engrg. Experiment Station Bull. #185*, Univ. of Illinois, Urbana.

Moehle J., Elwood K. and Sezen H., 2002. Gravity load collapse of building frames during earthquakes. *S.M. Uzumeri Symp.Behavior and Design of Conc. Str. for Seismic Performance*, ACI-SP 197.

Federal Emergency Management Agency, 1997. *NEHRP Guidelines for the seismic rehabilitation of buildings (FEMA Report 273)*, Washington D.C.

Tastani S., Pantazopoulou S., Zdoumba D., Plakantaras V., Akritidis E., 2005. Limitations of FRP-jacketing in confining old-type r.c. members in axial compression. in print, *ASCE J. of Comp. for Construction*.

Aschheim M. and Black E., 2000. Yield point spectra for seismic design and rehabilitation. *EERI Earthquake Spectra* 16 (2), 317-336.