SEISMIC UPGRADE OF REINFORCED CONCRETE COLUMNS WITH FRP

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1 INTRODUCTION

The strengthening of vertical elements in reinforced concrete, either columns or bridge piers, has different implications depending on whether the strengthening measure is carried out on a conventional structure or on a structure in a seismic area.

For conventional structures, the objective is usually to increase the bearing capacity, and therefore the strengthening measures aims either at enlarging the cross sectional area or at enhancing the compressive strength of concrete by applying a confining action. Such measures are generally applied in buildings where live loads have increased consequent to a change in use. In the case of bridge piers, which can usually rely on adequate safety levels with respect to the vertical loads, confining measures are applied in cases when concrete is heavily damaged or if required by a live load increase (e.g., third lane construction, etc.).

In the structures built in seismic areas according to obsolete codes, the flexural capacity is generally adequate, as a result of the conservative design assumptions inherent in the elastic design approach. It is known that obsolete codes focused on the strength aspects while only making implicit reference to the concepts of ductility and dissipation capacity, and, which is more important, gave no provisions to ensure stability of the response in the post-elastic range. Ductility is the property of being able to deform through several cycles of displacements much larger than the yield displacement, without significant strength degradation. Displacement ductility as high as 6 to 8 may be needed sometimes.

Existing structures built according to obsolete codes – as assessed either from original project drawings or through *in-situ* inspections after destructive seismic events – systematically show insufficient transverse reinforcements and thus lack the confinement necessary for ensuring a ductile response. In Figure 1 the lateral collapse mechanism of a column with insufficient transverse reinforcement is shown.

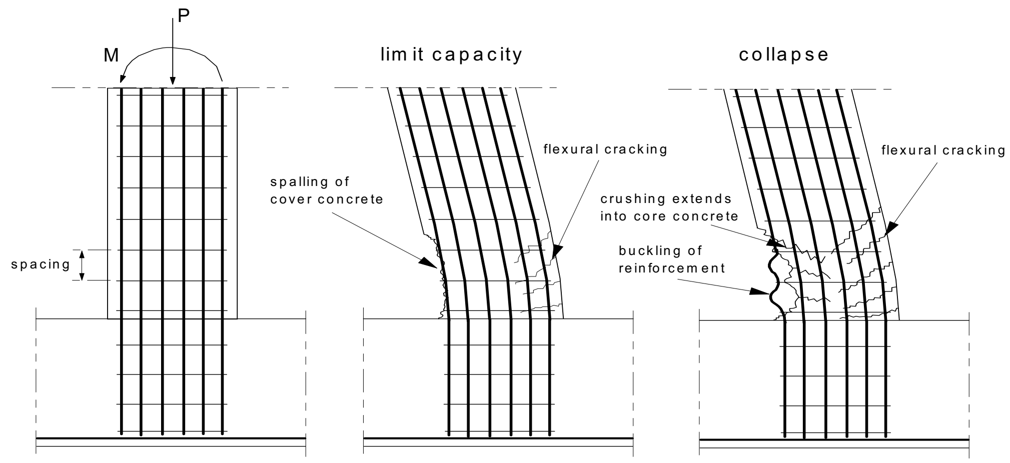


Figure 1. Lateral collapse mechanism of an under-designed column.

At displacement ductility 2 to 3, spalling of the cover concrete occurs in the plastic hinge zones, where inelastic deformations concentrate. Unless the core concrete is well confined by close-spaced transverse hoops or spirals, crushing extends into the core, the longitudinal reinforcement buckles, and rapid strength degradation follows. This behavior can even be accelerated when transverse reinforcement is lapped in the cover concrete, as is often the case in old constructions. The hoops then loose effectiveness at lap locations, when concrete spalls.

Common retrofitting techniques of columns typically aim at increasing the available ductility by enhancing the confinement action in the potential plastic hinge region. However, enhancement of the flexural strength can be sought in lap-spliced zones or when longitudinal reinforcement is terminated prematurely.

It is already well known that confinement of concrete enhances its strength and ductility. Therefore, improved confinement will increase the ability of a column to withstand repeated cycles of loading beyond the elastic limit and tend to prevent column failure due to degradation of flexural capacity. Debonding of longitudinal reinforcement lap-splices and formation of plastic hinges at regions of termination of longitudinal reinforcement can also be prevented by adequate confinement.

When necessary, retrofitting techniques are sometimes directed at increasing flexural strength. This retrofit method should be used carefully: increased flexural capacity will increase the forces transferred to the foundation and the superstructure/column connections, and will also result in increased column shear force. Since failure of the foundation or brittle shear failure of the columns are usually more critical than excessive flexural yielding, this method should only be used when loss of flexural strength results in a collapse mechanism, and not without taken precautions.

Concrete and steel jacketing have had an extensive use in practice and have proved to be effective measures for retrofitting existing columns. Yet, the engineering community has recently looked for alternatives, with the objective of improving easiness of transportation and construction and to reduce maintenance cost due to corrosion of steel. Advanced composite materials in FRP are now recognized to represent an effective alternative retrofit technique for columns.

In the last years, in California, USA, more than 500 bridge piers have been wrapped with advanced composite materials (Seible et al. 1995, Xiao et al. 1995). Similar programs are currently under way in Japan (Hoshikuma and Unjoh 1997, JSCE 1995). In Europe, where notable interest exists (*fib* 2001), the subject is still in an interlocutory phase, mainly because of the lack of established and accepted design rules, which slow down the process of promoting FRP as an 'official' construction material.

In the following sections, a brief review of available strengthening techniques for columns is presented.

1.1 CONCRETE JACKETING

A concrete jacket consists of a comparatively thick layer of reinforced concrete cast around a column. Extensive longitudinal and transverse reinforcement is added in the new layer of concrete, improving the flexural strength and ductility. Firstly, the cover concrete is removed to expose the main reinforcing bars. In addition, chipping away the concrete cover of the original member and roughening its surface can improve the bond between the old and new concrete. U shaped steel links are then welded to the exposed bars; weldable steel is preferable to avoid brittleness. Additional bars are then welded to the U shaped links to form the longitudinal reinforcement. Stirrups are added as required and concrete is poured after the erection of timber formwork.

This is one of the most commonly applied methods of repair and strengthening of concrete members. Apart from welding, it does not require specialist knowledge. The main drawback is the uncertainty with regard to bond between the jacket and the original member.

This straightforward principle has been proven in tests in New Zealand, but is rather expensive, the constructability is poor and it arises aesthetical problems. Concrete jacketing has been commonly used in Japan, mostly for enhancing the flexural strength.

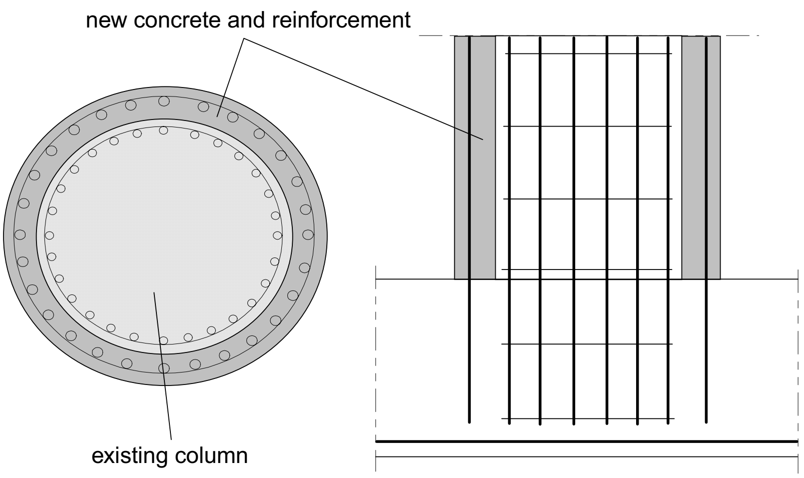


Figure 2. Concrete jacketing.

1.2 STEEL JACKETING (CAGING AND ENCASING)

In general, the techniques where either steel plate adhesion or steel welding in reinforced concrete is involved are fast and effective. In particular, considering the choice of the steel cage elements cross-sections, as shown in Figure 3, from a practical point of view, it is a cost-effective application of light cross-sections without reducing the effectiveness of the technique, as far as a satisfactory pretensioning degree of the transversal elements is applied. The simplicity and speed of the method's application provide a solution for critical intervention time immediately after a strong earthquake, particularly for special buildings such as hospitals and schools. An external steel cage is constructed from longitudinal angle sections and transversal steel strips. In practice, the strips are often laterally stressed either by special wrenches or by preheating to temperatures of about 200-400°C, prior to welding. Any spaces between the steel cage and the existing concrete are usually filled with non-shrinking mortars. When required to provide corrosion or fire protection to the cage, a covering with concrete or shotcrete can be provided.

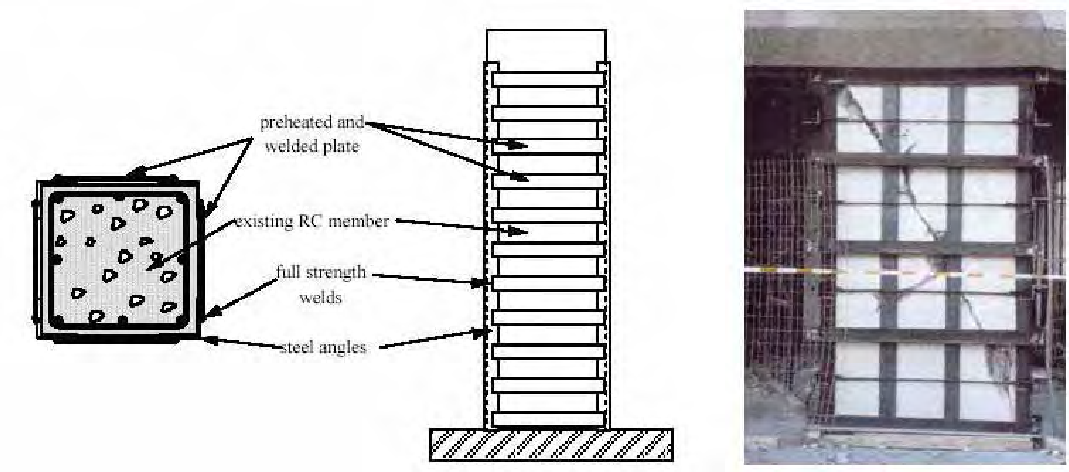


Figure 3. Steel jacketing (caging) of a square section column.

When strengthening circular section columns, an alternative approach involves the total encasement of the column with thin steel plates placed at a small distance from the column surface, with the ensuing gap being filled with non-shrinking grouts (Figure 4).

A steel jacket usually consists of two half shells of steel placed around the column, with a clearance of about 10-25 mm, and welded together after placing. The gap between the jacket and column is filled with pure cement grout. A space of about 50 mm is left between the jacket and the joining member, or the footing, to avoid the possibility of direct load-carrying action of the jacket at large drift angles, which would cause local buckling in the jacket.

The jacket provides a passive confinement action. Lateral confining pressure is induced in the concrete as it dilates laterally under high axial strain levels in the compression zone of the column, due to the hoop strength and stiffness of the jacket. Also in the tension zone the jacket is effective, due to dilating splice-induced vertical cracks, and residual lateral dilation from previous load cycles. The jacket can be considered as equivalent to continuous hoop-reinforcement.

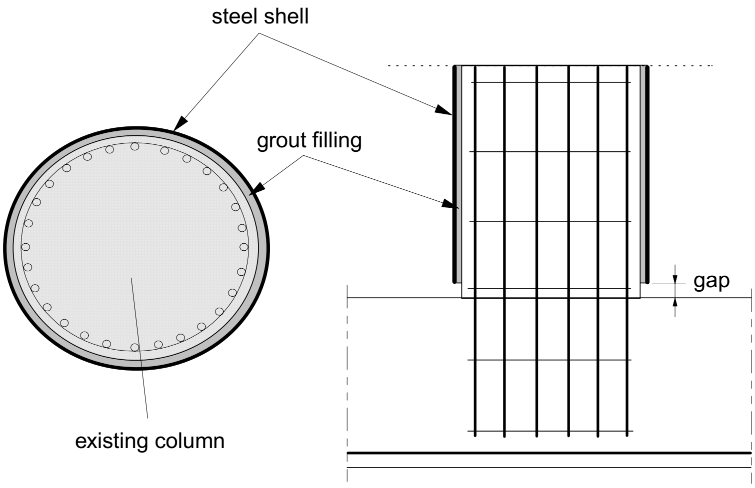


Figure 4. Steel jacketing (encasing) of a circular section column.

The performance of columns retrofitted with steel jackets has been thoroughly tested, and has proven to be a very effective method. Performance of columns failing in flexure, as a result of the deficiencies mentioned earlier, can be converted to ductile flexural response at least as good as new columns designed to current design approaches.

Both techniques require only a small increase of the member cross-section and are less disruptive than the concrete jacketing technique.

Steel jackets have been extensively implemented. In California, USA, steel jacket retrofit is normally directed at increasing the confinement action in potential plastic hinge regions, so to improve the column ductility. In Japan, it is mostly meant as additional longitudinal reinforcement of the column, for enhancing its flexural strength.

Some years after the first applications, it is recognized that steel jackets arise sometimes problems relative to both construction difficulties and most of all corrosion. Construction is difficult because of the placing and welding, and construction costs are high because of heavy-weight transportation and placing. Corrosion can be a problem at both ends of the jacket, so maintenance is very important.

1.3 FRP WRAPPING

Until a few years ago, the only available techniques for upgrading vertical structural elements were those presented in the previous sections. Only recently, have fibre reinforced polymers (FRP) been recognised as an effective strengthening technique for degraded or inadequate reinforced concrete members.

The remarkable properties of FRP, such as high specific strength, and mostly also high specific stiffness, low thickness and weight, and resistance to corrosion, allow them to be applied in a construction site without serious difficulties.

An FRP jacket can consist of active or passive layers, or a combination, of different FRP materials. Normally carbonfiber and/or fiberglass are used, sometimes also aramid-fibers like Kevlar™ or Twaron™, in combination with a resin matrix, usually epoxy. Numerous combinations can be made, which is one of the main advantages of FRP jackets.

Experimental studies and pilot applications have demonstrated that, by wrapping vertical elements with FRP jackets placed on one or more layers, a confining action on the concrete is obtained that enhances both strength and ductility of the whole element. In the case of columns, FRP composite jacketing techniques have been shown to have performance capabilities comparable to and in some cases better than columns retrofitted through the application of the above-presented conventional methods.

The confinement action so obtained is of the “passive” type, that is, it develops only consequent to the transverse dilation of the compressed concrete core that stretches the confining device, which thus applies an inward confining pressure (Figure 5). “Active” confinement action can be obtained by pre-tensioning the sheets prior to application. Generally, carbon fibres (CFRP) are preferred in those cases where the objective is the bearing capacity increase of the column, while glass fibres (GFRP) are more suitable, thanks to their higher deformability, to cases where a ductility increase is sought instead.

A detailed description of all available methods for FRP wrapping is presented in 4.

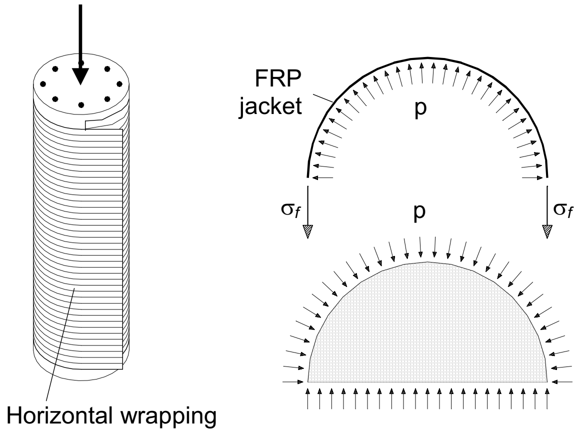


Figure 5. Confining columns with FRP, taking advantage of the transverse dilation of the compressed concrete core under axial load.

1.4 PRESTRESSING WIRES OR EXTERNAL HOOPS

A fourth retrofit principle is placing vertical prestressing steel cables around the column. This principle has been suggested primarily for bridge piers. To the author's knowledge, implementation has not been carried out anywhere in the world.

2 CONCRETE STRENGTH AND DUCTILITY

In the last three decades the behaviour of confined concrete has been deeply studied in countless researches, whose results are now well known and established. All of those studies referred concrete confined by steel, which, after yielding, exerts a constant confining pressure. This allowed all researchers to relate the confined concrete properties as if under hydrostatic pressure, expressed in terms of the steel yield strength, thus avoiding to tackle the complex problem of concrete dilation and of its interaction with the confining device itself.

This standpoint had to change with the innovative introduction of FRP confining devices: FRP is an elastic material, and as such it does not yield; as a consequence, it exerts a continuously increasing confinement action on concrete. The response of FRP-confined concrete turns out to be completely different from the steel-confined one, and this opened the way to a remarkable research effort that in the last few years has produced a number of valuable studies, both experimental and analytical, with the common aim of clarifying all new aspects in this phenomenon.

The strength increase in confined concrete originates from a known fact: unconfined concrete under uniaxial compression up to 90% of its strength reduces in volume; beyond this value, it dilates. A confining pressure opposing such dilation remarkably improves its performance.

The most relevant findings of all experimental studies on FRP-confined concrete are condensed in Figure 6 and Figure 7. In Figure 6 the (normalized) behavior of (both Glass and Carbon) FRP-confined concrete are compared to the more familiar steel-confined concrete. All stress and strain quantities are normalized with respect to f'_{co} (unconfined concrete strength) and ε_{co} (strain at f'_{co}), respectively.

In the stress-strain relation, Figure 6 top-left, it is seen that the (G-C)FRP-confined concretes show an ever-increasing branch, as opposed to the steel-confined one, which, after reaching the peak strength, decays on a softening branch. Concrete degradation is proportional to the lateral strain: the increasing confinement action of the elastic FRP limits the lateral strain thus delaying the degradation; on the other hand, when steel yields, which occurs at 2.5 normalized axial strain, degradation of concrete takes place, because steel offers a zero stiffness to the lateral dilation of concrete. The increasing confinement action of the elastic FRP limits the lateral strain thus delaying degradation. As regards the ultimate strain, and implicitly ductility, it should be noted that, notwithstanding the low ultimate strain of the FRP-jackets, the concrete ultimate strain is comparable (CFRP) or even greater (GFRP) than that obtained with steel.

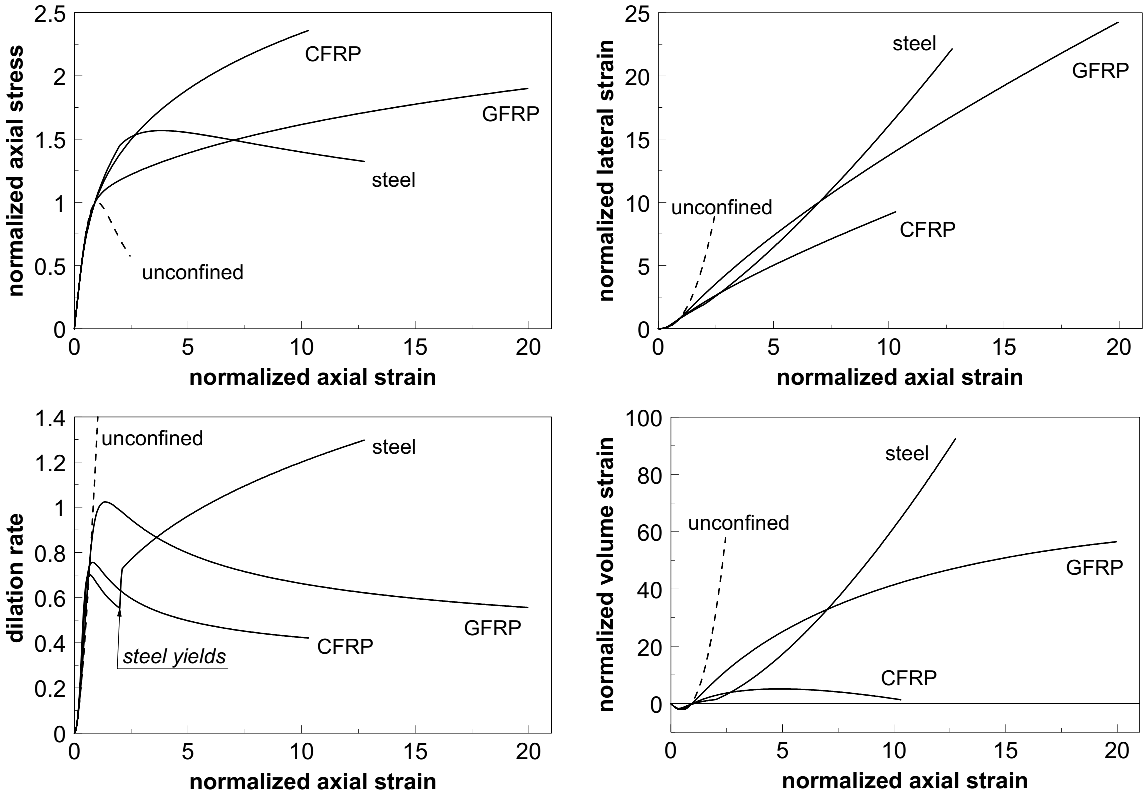


Figure 6. Schematic behaviour concrete confined with steel, CFRP and GFRP: axial stress vs. axial strain (top, left), lateral strain vs. axial strain (top, right), volume strain vs. axial strain (bottom, left), dilation rate vs. axial strain (bottom, right).

As regards the concrete ultimate strain, which influences the ductility attained through confinement, it should be noted that, notwithstanding the low values of ε_{ju} of the FRP-jackets, in these cases the concrete ultimate strain is comparable or even greater than that obtained through the use of a ductile confining device, *i.e.* steel.

This different behavior suggests (Spoelstra and Monti 1999) that, when FRP jackets are used, the ultimate axial strain of concrete is only weakly governed by the ultimate confinement pressure (proportional to $\rho_j \, f_{ju}$), whereas it is mostly dependent on the ultimate deformation. This is proven by the fact that the fiberglass-confined specimen shows an almost twice as large deformability than the carbonfiber-confined one, although the ultimate confinement pressure of the latter is roughly 50% larger.

In Figure 6 (top, right), the lateral strain vs. axial strain relation is shown. It can be observed that the slope of the branches depends on the stiffness of the confining device (which can also be observed in the previous diagram): steel and CFRP start with almost the same slope, but after steel yields at 2.5 normalized axial strain, it departs towards higher lateral strains. GFRP shows a more stable behavior, in the sense that it starts with a higher slope (meaning that concrete has a higher initial lateral dilation), which however remains constant until the jacket fails. CFRP reduces the initial lateral strain, but its effectiveness has a shorter duration, due to its lower extensional ultimate strain ε_{ju}.

This can be better appreciated in Figure 6 (bottom, left), where the dilation rate is expressed as function of the axial strain. The dilation rate $\mu = \Delta\varepsilon_l / \Delta\varepsilon_c$ is defined as the rate of increase of the lateral strain $\Delta\varepsilon_l$ to the corresponding axial strain increment $\Delta\varepsilon_c$. It is seen that when steel yields a discontinuity occurs, due to the abrupt change in modulus; after this, the dilation rate increases indefinitely. On the other hand, for the two FRP, it constantly decreases towards an asymptotic value. Note that 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 departs later than the other two. This is the point where a sufficient lateral pressure develops that prevents the lateral dilation of concrete from increasing unrestrained.

In Figure 6 (bottom, right), it is interesting to observe from the volume strain vs. 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.

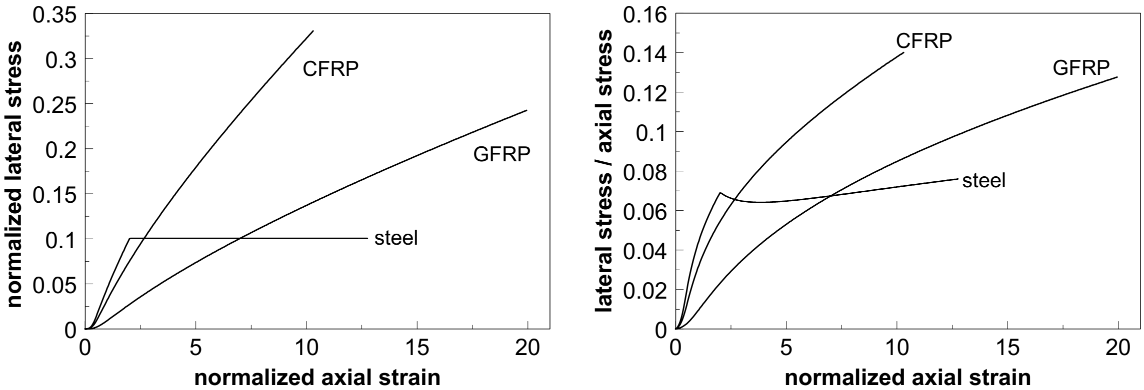


Figure 7. Comparison of confinement effectiveness on concrete confined with steel, CFRP and GFRP.

In Figure 7, left, the confinement effectiveness (lateral stress vs. axial strain) for all three types of jackets is compared. It is explicitly shown what expected, that is, before yielding the steel jacket exerts a higher confining action, which however remains constant after yield, whereas the FRP jackets show a monotonically increasing confinement, thus arriving at applying a confinement action twice (GFRP) or thrice (CFRP) that of steel, with the same volumetric ratio ρ_j. In Figure 7, right, it is interesting to compare the jacket effectiveness expressed in terms of ratio of the lateral stress to the *current* axial stress. The steel jacket effectiveness after yield is only due to the softening behavior of concrete, whereas in the other two cases it is the elastic behavior of the FRP jackets that increases the ratio. Here, it should be evident how the two FRP materials reach almost the same level of effectiveness, but at different axial strain levels, which renders more attractive the use of GFRP jackets that also exploit ductility while maintaining the same effectiveness of CFRP jackets.

The idea emerges from these graphs that CFRP should be used to provide concrete with higher strength increase and moderate ductility, whereas GFRP should be used to provide higher ductility and moderate strength increase. This findings will be the basis of the design-oriented considerations developed in the next section.

3 IDEALISATIONS USED IN DESIGN AND ANALYSIS

Phenomena underlying the strength and ductility increase of FRP-confined concrete are still the object of countless experimental and analytical studies (for a review, see also Monti 2001). Therefore, a general consensus on the equations to adopt in the design of upgrading measures is yet to be reached. However, some indications in that direction are contained in a recently issued *fib* Bulletin (2001). It should be noted that reliability-based studies are still under way (Monti and Santini 2002) which should provide calibrated values of the partial safety coefficients to be applied to the mechanic quantities of FRP. In the absence of such indications, the equations reported in the following should be considered as descriptive of the phenomenon and not as design equations. In precautious and conservative way, a safety coefficient γ_f=1.5 can be applied to the characteristic value of the FRP strength f_{fk}, so that a possible design value can be: $f_f = f_{fk}/\gamma_f$, while the elastic modulus E_f, which has a negligible variation from sample to sample, can be considered, for practical purposes, as deterministic (and therefore equal to that declared by the manufacturer). For the ultimate strain and the elastic modulus of the FRP jacket, see the considerations developed in 3.1.

As already discussed, two cases of FRP-strengthening should be distinguished: one in which it is required to increase the axial bearing capacity of the reinforced concrete element, and one in which a ductility increase is sought instead. In these two cases it is necessary to identify the value of two quantities that govern the design of the FRP jackets: in the former case the strength, in the latter the ultimate strain of concrete.

3.1 EFFECTIVE FRP PROPERTIES

The properties of FRP-confined concrete depend on both the ultimate strain ε_{fu} and the elastic modulus E_f of the FRP used for wrapping it. Generally, these two quantities, once the FRP is placed, attain values (characterised by the index: j = jacket) ε_{ju} and E_j that are lower than those declared by the manufacturer, referred instead to the sheets themselves. The reasons for such reduction are to be sought in the following phenomena:

- The presence of a triaxial stress state (Figure 8) that depends on the superposition of three effects on the FRP jacket: 1) the lateral expansion of concrete that produce a circumferential stress, 2) the bond with the concrete surface, which produces an axial stress, and 3), of lower importance, the outward pressure exerted by the expanding concrete on the internal face of the jacket. This reduces the ultimate strain ε_{ju} of the FRP jacket. Thus, in designing, in place of the ultimate strain ε_{fu} given by the manufacturer, the value: $\varepsilon_{ju} = \min[0.9 f_f/E_f = 0.9 f_{fk}/(\gamma_f E_f)\,, 0.9\varepsilon_{fu}/\gamma_f]$ should be taken instead. Values greater than this should be motivated by adequate experimental evidence.
- The execution quality: if the fibers are not properly aligned, due either to the presence of voids or to an inadequate surface preparation, part of the circumferential strain is employed to stretch the fibers and this produces a non-homogeneous stress state in the material. Besides, the fibers can be damaged in correspondence of local irregularities in the contact surface. This phenomenon brings to a reduction of the elastic modulus

of the jacket, generally in the order of 10%, so that one has: $E_j = 0.9E_f$. This inconvenience can be overcome by slightly pre-tensioning the sheets before application.

- The curvature of the jacket, especially in correspondence to insufficiently rounded column corners. This phenomenon is accounted for as described in 3.5, through a coefficient k_s .
- Influence of the thickness in the presence of multiple layers. This phenomenon can be neglected because its effect is of lower entity with respect to the others.

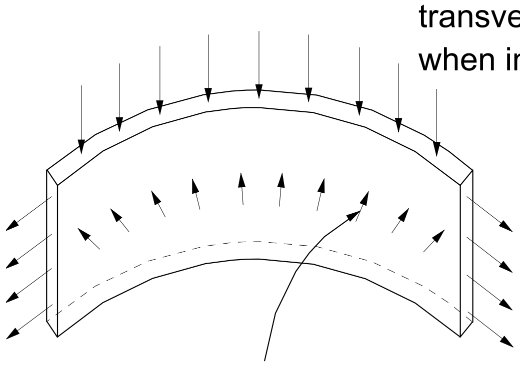


Figure 8. Triaxial stress state in FRP jackets.

3.2 EVALUATION OF THE CONFINING PRESSURE

Confinement with steel.

It is known that in an axially loaded concrete cylinder and confined with steel stirrups, the maximum confining pressure f_l is computed in terms of the transverse reinforcement ratio ρ_{st} and of its yield strength f_y, as:

$$f_l = \frac{1}{2} k_e \, \rho_{st} \, f_y \qquad \text{with} \qquad \rho_{st} = \frac{4\, A_{st}}{s\, d_s} \tag{1}$$

where k_e = arching- effect coefficient (usually 0.8), s = stirrups spacing, A_{st} = stirrups area, and d_s = diameter of the confined concrete core.

Confinement with FRP.

In the case of concrete elements continuously confined with FRP sheets, with the fibers aligned circumferentially, the maximum confining pressure f_l is obtained for ε_{ju} = effective ultimate strain of the FRP jacket (as determined in 3.1):

$$f_l = \frac{1}{2} \rho_j \, E_j \, \varepsilon_{ju} = 2 \frac{E_j \, \varepsilon_{ju} \, t_j}{d_j} \qquad \text{with} \qquad \rho_j = \frac{4 t_j}{d_j} \tag{2}$$

where ρ_j = FRP ratio, E_j, t_j, d_j = elastic modulus, thickness and diameter, respectively, of the FRP jacket.

Notice that the previous relation is valid only in the case of axially loaded cylindrical elements and continuously confined with FRP. For different configurations (*e.g.*, elements with rectangular cross-section, discontinuously confined with FRP strips), the reduction coefficients described in 3.5 should be applied.

3.3 EVALUATION OF THE ULTIMATE STRENGTH OF FRP-CONFINED CONCRETE

The evaluation of the ultimate strength of concrete is useful in those cases where the load bearing capacity of rc elements strengthened with FRP should be estimated. In all proposed equations, presented in the following, the confined concrete ultimate strength f'_{cc} is evaluated based on both the unconfined concrete strength f'_{co} and the confining pressure f_l, given by (2).

Most of the predictive equations are based on the equation of Richart et al. (1929):

$$\frac{f'_{cc}}{f'_{co}} = 1 + k_1 \frac{f_l}{f'_{co}} \tag{3}$$

where, for the parameter k_1 that measures the confinement effectiveness, the following expressions have been proposed:

$k_1 = 2.1\left(f_l / f'_{co}\right)^{-0.13}$ (Karbhari and Gao 1997)

$k_1 = 6 f_l^{-0.3}$ (Samaan et al. 1998)

$k_1 = 2.2\left(f_l / f'_{co}\right)^{-0.16}$ (Saafi et al. 1999)

$k_1 = 3.5\left(f_l / f'_{co}\right)^{-0.15}$ (Toutanji 1999).

Spoelstra and Monti (1999) proposed the following equation:

$$\frac{f'_{cc}}{f'_{co}} = 0.2 + 3\sqrt{\frac{f_l}{f'_{co}}} \tag{4}$$

while Saadatmanesh et al. (1994), Purba and Mufti (1999), use the equation of Mander et al. (1988):

$$\frac{f'_{cc}}{f'_{co}} = 2.254\sqrt{1 + 7.94\frac{f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.254 \tag{5}$$

3.4 EVALUATION OF THE ULTIMATE STRAIN OF FRP-CONFINED CONCRETE

The evaluation of the concrete ultimate strain ε_{cu}, increased by the confinement exerted by the FRP, is fundamental when estimating the ductility of vertical elements seismically under-designed and subsequently upgraded with FRP. In the following the two more used expressions are recalled.

Seible et al. (1995), propose an expression for ε_{cu}, used for steel-confined concrete:

$$\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_{st} f_y \varepsilon_{su}}{f'_{cc}} \qquad \text{(with steel)} \tag{6}$$

where ε_{su} = stirrups ultimate strain, having yield strength f_y and with transverse reinforcement ratio ρ_{st}. The above expression has been adapted to the case of FRP:

$$\varepsilon_{cu} = 0.004 + \frac{2.5 \rho_j \, E_f \, \varepsilon_{ju}^2}{f'_{cc}} \qquad \text{(with FRP)} \tag{7}$$

where E_f = FRP elastic modulus, ε_{ju} = effective ultimate strain of the FRP jacket (as determined in 3.1, with the modifications in 3.5 when necessary); in both the above equations f'_{cc} is computer with (5), while f_l is computed with (1) in the former case and with (2) in the latter.

An alternative expression, to be used in the case of FRP-confined concrete, has been proposed by Spoelstra and Monti (1999):

$$\varepsilon_{cu} = \varepsilon_{co}\left(2 + 1.25\,\frac{E_c}{f'_{co}}\,\varepsilon_{ju}\sqrt{\frac{f_l}{f'_{co}}}\right) \tag{8}$$

where f_l is computer with (2), f'_{co} = unconfined concrete strength, and ε_{ju} = effective ultimate strain of the FRP jacket (as determined in 3.1, with the modifications in 3.5 when necessary). Notice the dependence on the elastic modulus of concrete E_c and on the strain of concrete at peak strength $\varepsilon_{co} \approx$ 0.002.

Note that, by expressing stresses in MPa and assuming for the elastic modulus of concrete: $E_c = 5700\sqrt{f'_{co}}$ (Eurocode 2, 1992), the previous equation simplifies into:

$$\varepsilon_{cu} = 0.004 + 14.25\,\frac{\varepsilon_{ju}}{f'_{co}}\sqrt{f_l} \tag{9}$$

where f_l is computed with (2), with the modifications reported in 3.5 when necessary.

3.5 MODIFICATIONS FOR NON-CIRCULAR CROSS-SECTIONS AND CONFINEMENT WITH STRIPS

In the case of rectangular or square cross-sections, in which the corners have been rounded with radius R_c (comprised between 5 mm and 40 mm) before wrapping with FRP, the confinement effectiveness is significantly reduced. It has been proposed (Mirmiran et al. 1998) to adopt in these cases a modified confinement pressure $f'_l = k_s f_l$ (with $f_l = 2\, f_f\, t_j / D$) through the coefficient:

$$k_s = \frac{2\,R_c}{D} \tag{10}$$

where *D* is the longer side of the cross-section. More recent studies (Monti and Renzelli 2002), which however need refinement, describe in a more accurate and rational manner the effects of confinement on rectangular sections.

When the concrete cover of the elements to strengthen is not sufficient, it is difficult to round the corners with a high radius R_c. In these cases, Priestley et al. (1994) propose to inscribe the rectangular section in one of different geometry (usually circular or elliptical, as for example shown in Figure 12). For elliptical sections, the confinement pressure is given as function of the semi-axes *a* and *b*:

$$f_l' = \frac{f_f\, t_j\,[1.5(a+b) - \sqrt{ab}]}{2\,a\,b} \tag{11}$$

In cases where the strengthening is not in form of continuous wraps, because realized either with discontinuous spiral or with spaced rings, it has been proposed (Saadatmanesh et al. 1994) a reduced value of the confining pressure $f_l' = k_g f_l$ (with $f_l = 2\, f_f\, t_j / D$), by means of a coefficient:

$$k_g = \frac{(1 - s_f / 2d)^2}{1 - \rho_{sc}} \tag{12}$$

where s_f is the rings spacing and ρ_{sc} is the ratio of the longitudinal reinforcement area to the area of concrete confined by stirrups.

4 FRP WRAPPED COLUMNS

4.1 INTRODUCTION

Different types of column FRP wrapping systems have been investigated and developed based on material types, form and process of application (Figure 9). They can be classified into five categories based on the method of processing/installation (Karbhari et al. 2001):

1. wet lay-up process using fabric, tape or individual tow;
2. prepreg in the form of tow, tape or fabric;
3. prefabricated shells;
4. resin infusion processes;
5. external composite cables or prefabricated strips.

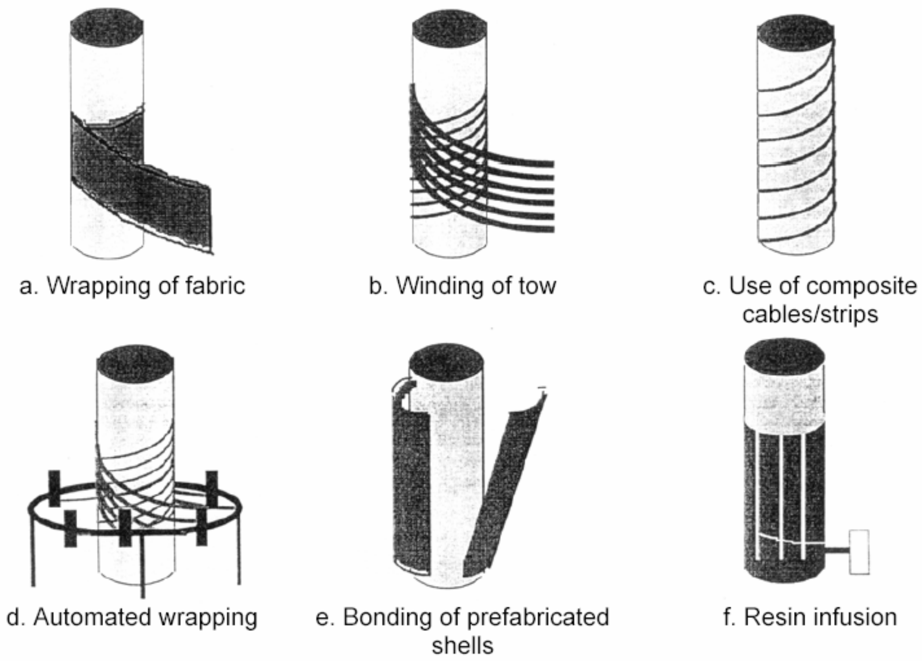


Figure 9. Methods of FRP wrapping of RC columns (Karbhari et al. 2001).

In technique a) the column can be wrapped with either mono- or multi-layer FRP sheets, or even with FRP strips placed in spirals or rings (Figure 10a). Applications of this technique are amply reported for both buildings columns and bridge piers (see for ex., ACI 1996, Neale and Labossiere 1997, Tan 1997). Laying-up of prepreg tape is a straightforward, very fast construction principle, but it is more difficult to control, since it is carried out by hand completely, and there are concerns related to the quality control of resin mix, attainment of good wet-out of fibers with uniform resin impregnation without entrapment of excessive voids, good compaction of fibers without excessive wrinkling of the predominantly hoop-directed fibers, control of cure kinetics and achievement of full cure, and aspects related to environmental durability during and after cure.

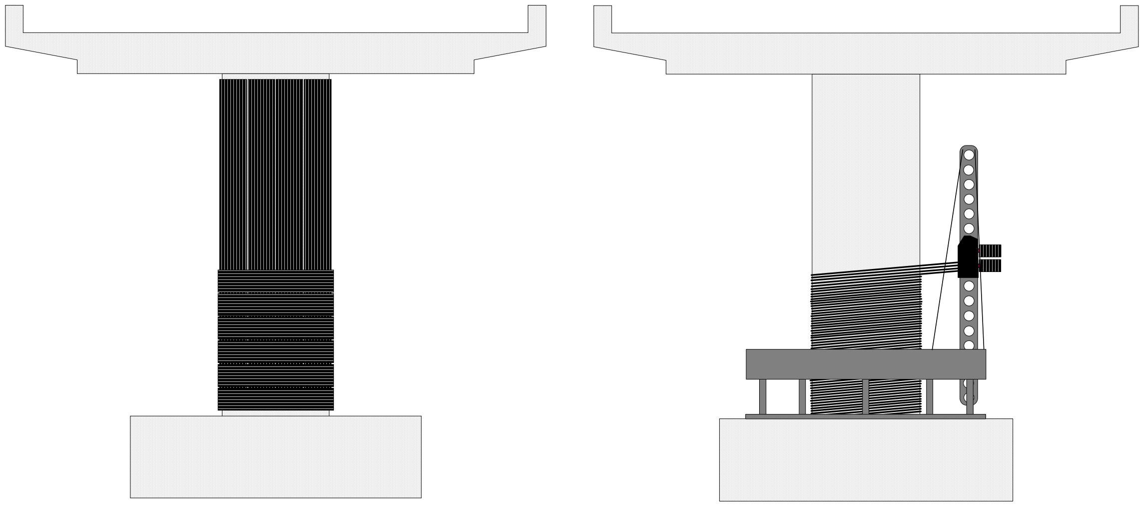
In the case of wet winding tow or tape (b), the process may be automated, although resin impregnation is still through the use of wet bath and/or spray, and many concerns are the same as those described for the lay-up process. Moreover, the necessity of curing under elevated temperatures (usually in the range of 80-150°C) can cause problems if the substrate concrete is very moist resulting in water vapour driven blistering in the curing composite jacketing.

Method (c), involving the use of cables or prefabricated strips, has not been investigated to a large extent as yet.

In technique d) a wrapping machine is used for automatically winding the fibers around the column (Figure 10-b, Figure 11-b). The machine, built for the first time in Japan in the 80ies (ACI 1996), has been designed for the upgrade of bridge piers, but it can be used for buildings columns, as well. The automated wrapping device is set up around the column. The fibers, wound on reels, are placed in the fiber winding head, and moving upwards are wound around the column, pre-impregnated with the resin. After winding, a curing blanket is placed. Winding angle, fiber volume fraction, and thickness are fully computer controlled. With this device it is possible to wind the fibers while pre-tensioning them, so to obtain an active jacketing system, independent of the concrete lateral dilation. The disadvantage of such device is that, in the presence of non-leveled soils, preliminary calibration operations are required, which sensibly slow down its use. The same effect can be actually obtained with the other systems, by injecting either expanding mortar or epoxy in pressure between the jacket and the column surface.

Technique e) consists in the application around the column of two prefabricated half shells (Figure 11a) that can be, depending on its cross-sectional shape, either circular (for ex., Nanni and Norris 1995) or rectangular (for ex., Ohno et al. 1997). In alternative, full circular shells with a vertical slit can be opened and placed around the column (for ex., Xiao and Ma 1997). This is a very simple system in any *in situ* application, and affords a high level of materials quality control due to controlled factory-based fabrication of shells. However, the shells must be realised with strict tolerance with respect to the piers dimensions. In case of multiple layers, they must be properly positioned to ensure the desired collaboration of the entire jacketing system. Prefabricated shells can be used as formworks, also functioning as transverse reinforcement; in the case of rectangular sections, the confining action of the shells is less effective and it is generally preferred to modify the column cross-sectional shape (as explained hereafter).

Finally, in the case of resin infusion (f), the dry fabric is applied manually and resin is then infused using vacuum with cure being under ambient conditions.



a) Prepreg tape lay-up b) Automated fiber wrapping system

Figure 10. Schematic examples of FRP wrappings.

a) b)

Figure 11. (a) Positioning prefabricated FRP shells, and (b) automated wrapping system.

It is worth recalling that the increase in strength obtainable by FRP-wrapping is not significant as that in ductility. However, when necessary, upgrading techniques of columns aim at enhancing the flexural capacity. In this case, capacity design criteria should be applied (as, for ex., expressed in Eurocode 2 (CEN 1991)), because an increased flexural capacity in the column introduces higher forces in the beam-column joint (or in the footings) and it amplifies the shear action in the column itself. Because these resisting mechanisms are of the brittle type, it is mandatory to avoid them, by strengthening them as well.

The purpose of confining a column is therefore either enhancing its load carrying capacity or its ductility under lateral actions, such as those induced by earthquakes. Both cases will be presented in the following section 4.2. Moreover, the confining device offers a transverse constraint to the longitudinal bars, preventing them from buckling, and avoids the spalling of the concrete cover. This technique can also be used to prevent premature slippage of rebars in lap-splicing zones, or to avoid rebars pullout in anchorage zones, as presented in section 4.3.

It is extremely important to understand that, in case of members with rectangular cross-section, the confining action is less effective than for the circular. In fact, due to the long distance between the corners, the composite does not actually confine the internal concrete structure if just applied to the surface. In these cases, reinforcing fibres are often loose and unable to provide confinement. If the aspect ratio is larger than 2, it is appropriate to inscribe the section within an elliptical shape cast in (preferably light) concrete, which is subsequently FRP-wrapped (Figure 12). If this solution is not viable, due to the evident weight increase, the corners must be rounded in order to avoid excessive stress concentrations in the sheets folded around them (the radius is about 15 to 25 mm, depending on the specifications given by the FRP jacket supplier and on the available concrete cover thickness). Design issues relevant to the above described cases are dealt with in Section 3.5.

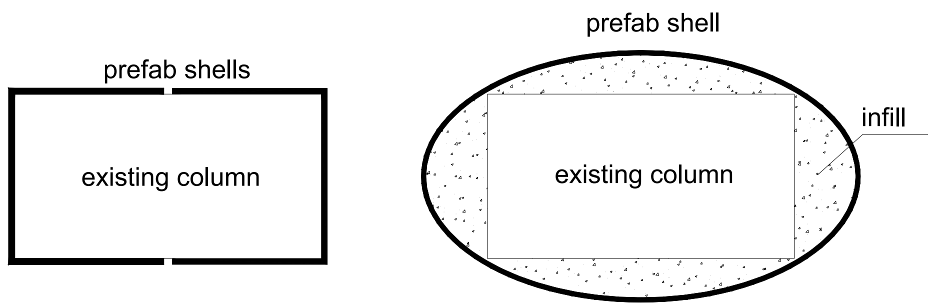


Figure 12. Ovalisation of a rectangular section prior to wrapping.

4.2 LOAD CARRYING CAPACITY AND DUCTILITY

4.2.1 Load carrying capacity upgrade

The design of the FRP quantity is carried out starting from the value of the design load N_d. The required value for the FRP-confined concrete strength is obtained as:

$$f'_{cc} = \frac{N_d}{A_c} \tag{13}$$

where A_c is the concrete area to be confined.

Once the sought value of the confined concrete strength is computed, one of the equations presented in 3.3 can be used, as for example:

$$\frac{f'_{cc}}{f'_{co}} = 0.2 + 3\sqrt{\frac{f_l}{f'_{co}}} \tag{14}$$

in order to obtain the required confining pressure f_l, and applying, for cross-sections different from the circular, the appropriate coefficients presented in 3.5; having computed f_l, from:

$$f_l = \frac{1}{2}\rho_j E_j \varepsilon_{ju} = 2\frac{E_j \varepsilon_{ju} t_j}{d_j} \qquad \text{with} \qquad \rho_j = \frac{4t_j}{d_j} \tag{15}$$

the thickness t_j of the FRP jacket is easily found.

4.2.1.1 Example: upgrading of the compressive strength of a RC column

It is desired to design an FRP wrapping (with fibers laid horizontally) of a whole RC column having rectangular section, in order to obtain a 20% increase of the vertical load bearing capacity (Figure 13). Before application of the wrapping, in order to avoid concentration of stresses at the corners, these have been rounded with a radius of R_c.

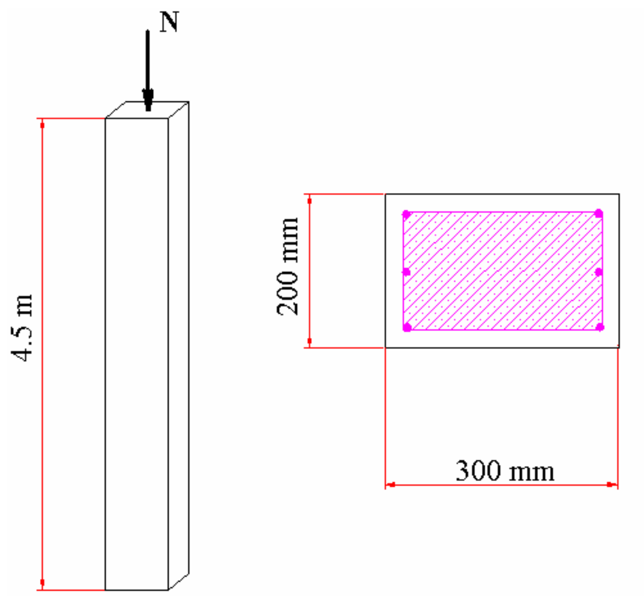


Figure 13. Example of a RC column strengthened with FRP to obtain a 20% increase of the load carrying capacity.

Column properties		
h	Height	4.50 m
b	Section width	0.30 m
d	Section depth	0.20 m
f'_{co}	Unconfined concrete strength	25 MPa
f'_{cc}	Required confined concrete strength (20% increase with respect to f'_{co})	30 MPa
R_c	Radius of rounded corners	40 mm

Properties of CFRP[1]		
E_{fk}	Elastic modulus	230000 MPa
ε_{fu}	Ultimate strain	1.5 %
γ_f	Partial safety factor	1.5
[1] For strength upgrade CFRP is preferred, see Figure 3.5		

- Confining pressure f_l needed to obtain a confined concrete strength $f'_{cc} = 1.20 f'_{co}$, evaluated based on (4):

$$\frac{f'_{cc}}{f'_{co}} = 0.2 + 3\sqrt{\frac{f_l}{f'_{co}}} \rightarrow f_l = \frac{1}{9}\left(\frac{f'_{cc}}{f'_{co}} - 0.2\right)^2 \cdot f'_{co} = \frac{1}{9}\left(\frac{30}{25} - 0.2\right)^2 \cdot 25 = 2.7 \text{ MPa}$$

- Reduction of the effective confining pressure due to the effect of rectangular section (see considerations in 3.3.5):

$$k_s = \frac{2R_c}{b} = \frac{2 \cdot 40}{300} = 0.27$$

- Confining pressure to apply to account for the loss due to the corners rounding (see considerations in 3.3.5):

$$f'_l = f_l \frac{1}{k_s} = 2.7 \frac{1}{0.27} = 10 \text{ MPa}$$

- Properties of the selected CFRP (see considerations in 3.3.1, with $\gamma_f = 1.5$):
  - design elastic modulus of CFRP:
    $E_j = 0.9 E_{fk} = 207000$ MPa
  - design ultimate strain of CFRP:

$$\varepsilon_{ju} = \min\begin{cases} 0.9\, f_f / E_f \\ 0.9\, \varepsilon_{fu} / \gamma_f \end{cases} = 0.009$$

- The FRP jacket thickness is obtained from (2):

$$t_j = \frac{f_l}{2} \frac{d_j}{E_j \varepsilon_{ju}} = \frac{10}{2} \frac{300}{207000 \cdot 0.009} = 0.8 \text{ mm}$$

Considering for each layer a typical thickness of about 0.17 mm, 5 CFRP layers are needed in order to obtain a 20% increase of vertical load bearing capacity in the RC column with rectangular section.

4.2.2 Ductility upgrade

In the following, three criteria are proposed that aim at upgrading the ductility of vertical structural elements with insufficient transverse reinforcement..

According to Seible et al. (1995) the deformation capacity of plastic hinge zones of a circular element with diameter d_j can be enhanced by confining the concrete core with composite material having thickness:

$$t_j = 0.09 \frac{d_j \left(\varepsilon_{cu} - 0.004\right) f'_{cc}}{\phi_f \cdot E_j \cdot \varepsilon_{ju}^2} \tag{16}$$

where, for design purposes, the confined concrete strength f'_{cc} can be taken equal to 1.5 times the unconfined one f'_{co}; E_j, ε_{ju} are, respectively, the elastic modulus and the ultimate strain of the jacket in circumferential direction (for which, the considerations in 3.1 apply, with the modifications in 3.5 when necessary), ϕ_f is a reduction factor of the sectional capacity (usually taken as 0.9), ε_{cu} is the ultimate concrete strain needed to reach the required sectional ductility. It should be noted that the latter value is not readily related to the target ductility, so that it is necessary to perform moment-curvature analyses from which the concrete strain corresponding to the ductility level required is obtained.

Japanese researchers (Mutsuyoshi et al. 1999) 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 to the moment capacity M_u of the member after retrofit, according to empirical equations of the type:

$$\mu_\Delta = \alpha + \beta\left(V_u a / M_u\right) \leq 10 \tag{17}$$

where a is the shear span and the constants α, β depend on the type (that is the deformability) of the fibres.

An alternative design equation has been proposed (Monti et al. 2001) for the ductility upgrade of circular columns having diameter D. The equation stems from the definition of a *section upgrading index* $I_{\sec} = \delta_{\sec}^{tar} \big/ \delta_{\sec}^{ava}$, representing the ratio of the target sectional ductility (to be obtained through upgrading) and the initially available sectional ductility (evaluated through assessing the existing section).

The proposed equation yields the needed confining pressure, as function of the ultimate strain ε_{ju} of the FRP jacket (for which, the considerations in 3.1 apply, with the modifications in 3.5 when necessary):

$$f_l = 0.4\ I_{\sec}^2 \frac{f_{cc,st} \cdot \varepsilon_{cu,st}^2}{\varepsilon_{ju}^{1.5}} \tag{18}$$

and it is expressed as function of two quantities: 1) the strength $f_{cc,st}$ of the existing steel-confined concrete, determined through (Mander et al. 1988, see section 3.3):

$$\frac{f'_{cc}}{f'_{co}} = 2.254\sqrt{1 + 7.94\frac{f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.254 \tag{19}$$

with f_l computed in terms of the transverse reinforcement ratio ρ_{st} and of its yield strength f_y, as (see section 3.2):

$$f_l = \frac{1}{2} k_e\ \rho_{st}\ f_y \qquad \text{with} \qquad \rho_{st} = \frac{4\,A_{st}}{s\,d_s} \tag{20}$$

where k_e = arching- effect coefficient (usually 0.8), s = stirrups spacing, A_{st} = stirrups area, and d_s = diameter of the confined concrete core; and 2) the ultimate strain $\varepsilon_{cu,st}$ of the existing steel-confined concrete, computed through (see section 3.4):

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_{st}\ f_y\ \varepsilon_{su}}{f'_{cc}} \tag{21}$$

where ε_{su} = stirrups ultimate strain, having yield strength f_y and with transverse reinforcement ratio ρ_{st}.

These two quantities are to be evaluated in the initial phase of assessing the current state of the structure (*assessment*), in the most critical section of the element to be upgraded in ductility, by considering only the confinement provided by the existing ties, whose quantity must be measured *in situ*.

The design of the FRP jacket with this method can be summarised as follows:

1. Compute $f_{cc,st}$ from (5) with f_l from (1), and $\varepsilon_{cu,st}$ from (6),
2. Assign the upgrading index $I_{\sec}$, that is, of how much the initial available ductility should be increased $\delta_{\sec}^{ava}$ in the section to be upgraded (evaluated from available computer programs) in order to obtain the target ductility $\delta_{\sec}^{tar}$ (as defined in the design phase),
3. Select the material for the FRP racket (and thus the properties E_j and ε_{ju}),
4. Compute the needed confining pressure from (18),
5. Compute the FRP thickness from (2) by applying if needed the considerations in 3.5.

4.2.2.1 Example: seismic upgrading of a RC column with circular section

This example is representative of a case of seismic upgrading of RC column improperly designed and that presents a scarce ductility in the base section, in the zone of potential formation of plastic hinge under earthquake (Figure 14). A jacketing is designed around the column base (with horizontally laid fibres) in order to obtain an increase in curvature ductility equal to 4 times that available before strengthening.

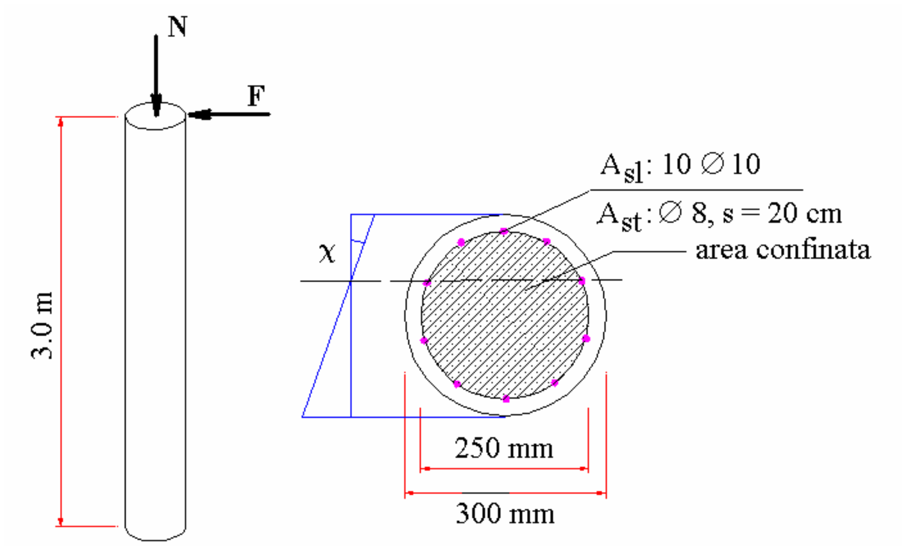


Figure 14. Example of a RC column strengthened with FRP to obtain a 400% increase of the curvature ductility (seismic upgrading).

Column properties		
h	Height	3.0 m
$d = d_j$	Section diameter = confining jacket diameter	0.30 m
d_s	Ties-confined core diameter	0.25 m
N	Axial load	100 kN
f'_{co}	Unconfined concrete strength	25 MPa
f_{sy}	Steel reinforcement strength	430 MPa
	Longitudinal reinforcement: 10 ϕ 10; ties: ϕ 8 at 200 mm	

Properties of GFRP[1]		
E_{fk}	Elastic modulus	65000 MPa
ε_{fu}	Ultimate strain	2.8 %
γ_f	Partial safety factor	1.5
[1] For ductility upgrading GFRP is used, see Figure 3.5		

- Ratio of transverse reinforcement:
  $$\rho_{st} = \frac{4A_{st}}{s\, d_s} = \frac{4 \cdot 0.5}{20 \cdot 25} = 0.004$$
- Initial available sectional curvature ductility (computer through computer program):
  $$\delta_\chi^{ava} = 6.90$$
- Upgrading index (the available ductility is made 4 times larger):
  $$I_{\sec} = \frac{\delta_\chi^{tar}}{\delta_\chi^{ava}} = 4$$
- Properties of the selected GFRP material (with the considerations in 3.1, with $\gamma_f = 1.5$):
  - design elastic modulus of GFRP:
    $$E_j = 0.9 E_{fk} = 58500 \text{ MPa}$$
  - design ultimate strain of GFRP:
    $$\varepsilon_{ju} = \min\begin{cases} 0.9\, f_f / E_f \\ 0.9\, \varepsilon_{fu} / \gamma_f \end{cases} = 0.0168$$
- needed confining pressure exerted by the GFRP jacket:

  <u>Procedure</u>:
  1) compute the confining pressure exerted by the steel ties f_l with (1):
  $$f_l = \frac{1}{2} k_e \rho_{st} f_y = \frac{1}{2} 0.8 \cdot 0.004 \cdot 430 = 0.7 \text{ MPa}$$
  2) compute the steel ties-confined concrete strength $f'_{cc,st}$ with (5):

$$\frac{f'_{cc,st}}{f'_{co}} = 2.254\sqrt{1+7.94\frac{f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.254$$

that is :

$$\frac{f'_{cc,st}}{f'_{co}} = 2.254\sqrt{1+7.94\frac{0.7}{25}} - 2\frac{0.7}{25} - 1.254 = 1.18$$

from which :

$$f'_{cc,st} = 1.18 \cdot f'_{co} = 1.18 \cdot 25 = 29.5\ \text{MPa}$$

3) compute the steel ties-confined concrete ultimate strain $\varepsilon_{cu,st}$ with (6):

$$\varepsilon_{cu,st} = 0.004 + \frac{1.4\rho_{st} f_y \varepsilon_{su}}{f'_{cc,st}} = 0.004 + \frac{1.4 \cdot 0.004 \cdot 430 \cdot 0.02}{29.5} = 0.0056$$

4) compute the confining pressure exerted by GFRP f_l with (18):

$$f_l = 0.4\, I_{\sec}^2 \frac{f_{cc,st} \cdot \varepsilon_{cu,st}^2}{\varepsilon_{ju}^{1.5}} = 0.4 \cdot 4^2 \frac{29.5 \cdot 0.0056^2}{0.0168} = 2.7\ \text{MPa}$$

- The FRP jacket thickness is obtained from (2):

$$t_j = \frac{f_l}{2}\frac{d_j}{E_j \varepsilon_{ju}} = \frac{2.7}{2}\frac{300}{58500 \cdot 0.0168} = 0.4\ \text{mm}$$

Considering that each layer has a typical thickness of about 0.17 mm, 3 layers of GFRP in order to obtain a ductility 4 times larger than the initially available one.

4.3 LAP-SPLICE FAILURE

From recent surveys on old buildings and bridges in seismic regions, it has been recognized that these structures very often had the longitudinal reinforcement insufficiently lap-spliced in the potential plastic hinge region (Figure 15). In pre-70ies designs, lap lengths were typically 20-35 times the bar diameter, which is too less to develop the ultimate strength of the bars, and thus the theoretical flexural strength of the column. Also for cases with larger lap-splice lengths, if the transverse reinforcement is inadequate to produce too low confinement pressure to inhibit splice failure, collapse is likely to occur if the column section is required high or even moderate ductility levels.

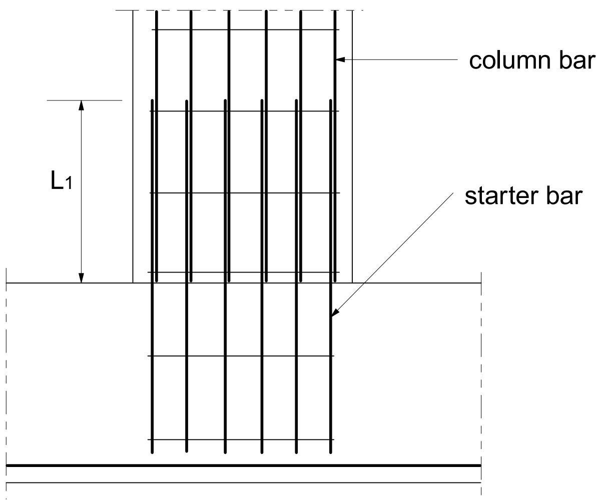


Figure 15. Typical column base with lap-spliced longitudinal bars.

Sometimes the longitudinal reinforcement is terminated at midheight of the column, based on the moment envelope. When this is designed without accounting for the effective tension shift due to diagonal shear cracking, unwanted plastic hinging might occur in this region. This can lead to severe damage, as was the case in Japan in the Urakawa-oki earthquake in 1982. In this case the termination at mid-height was provided with an inadequate development length of only 12 times the bar diameter.

Experimental evidence indicates that the flexural strength of columns with lap-spliced longitudinal reinforcement in the potential plastic hinge region degrades rapidly to a value equal to that which can be sustained by the axial compression force on the column, without contribution from reinforcement, using a reduced section size taken to the inside of the layer of longitudinal reinforcement, as is shown in Figure 16. The corresponding strength is:

$$M_R = P\left(\frac{D'}{2} - x\right) \tag{22}$$

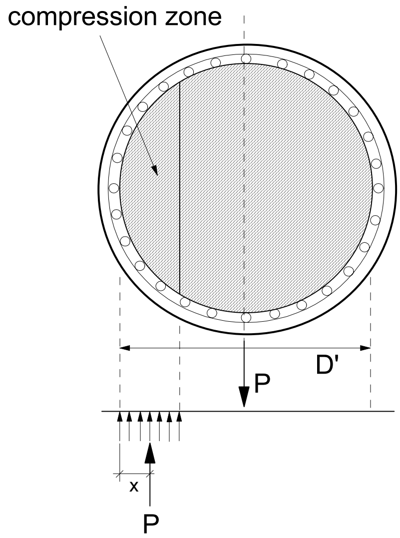


Figure 16. Residual moment capacity of columns after lap-splice failure.

4.3.1 Modeling of flexural strength and ductility of column sections

From a study on typical hysteresis loops of circular columns, the following model describing the flexural strength and ductility of column sections is proposed, as shown in Figure 17.

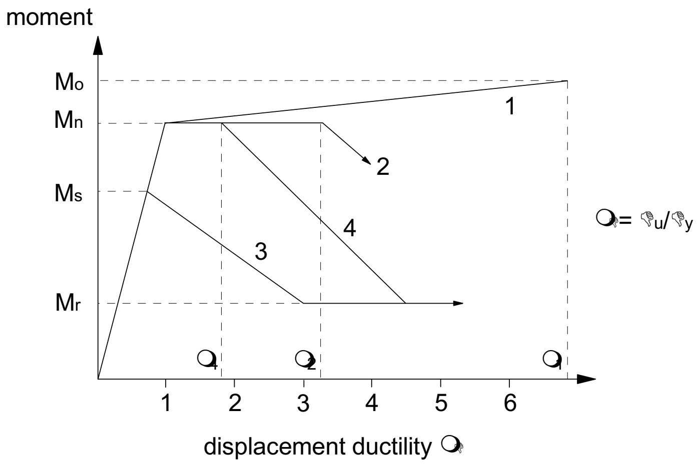


Figure 17. Flexural strength and displacement ductility of columns.

Line (1) is a bi-linear representation of a comparatively well confined column. The nominal moment capacity is reached at $\mu_\Delta = 1.0$, and an overstrength moment capacity M_0 attained at μ_1. The overstrength moment M_0 exceeds M_n as a result of strain-hardening of flexural reinforcement, and confinement effects.

Line (2) represents a poorly confined column without lap-splices in the plastic hinge region. The maximum strength does not exceed M_n, and the maximum displacement ductility factor μ_2 is found in correspondence of the attainment of $\varepsilon_{cu} = 0.005$ (poorly confined concrete). Typical values of $\mu_2 = 3.0$ will be found. When the ductility limits of lines (1) and (2) are reached, strength degrades rapidly due to crushing of the concrete core and buckling of the longitudinal reinforcement.

Line (3) represents degradation of a column with lap-splices, where the nominal moment capacity M_n will not be achieved. Strength starts degrading at less than $\mu_\Delta = 1.0$ from a maximum strength M_S to the residual strength M_r.

Line (4) represents degradation of a column with lap-splices, where the nominal moment capacity M_n will be achieved, and degradation occurs when a ductility level μ_4 is developed corresponding to an extreme concrete compression strain of $\varepsilon_c = 0.002$ (unconfined concrete).

The displacement and curvature ductility capacity of a column will be limited by the ultimate displacement Δ_u and ultimate curvature ϕ_u that can be sustained by the columns without collapse. The definition of these values is somewhat subjective, but a well-adopted approach for reinforced concrete columns is to define these values as the displacement (curvature) corresponding to either the first fracture of the confining reinforcement in a plastic hinge, which results in rapid degradation of performance, or to a 20% decrease in the lateral load (moment) capacity after the maximum strength has been reached, as shown in Figure 18.

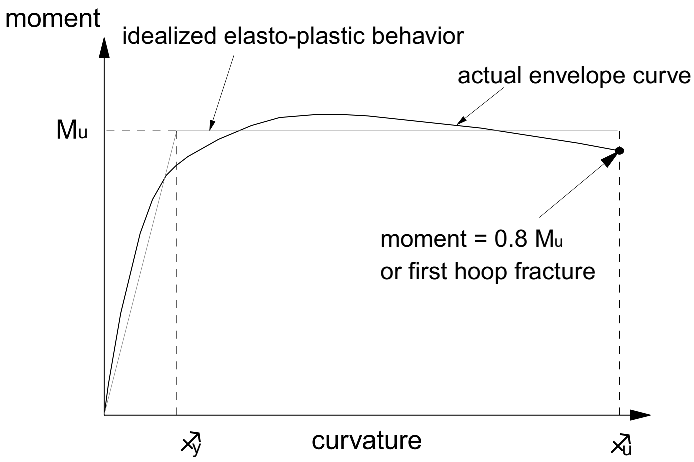


Figure 18. Definition of ultimate curvature.

The failure mechanism suggested by Priestley et al. (1994) to describe the behavior of lap splices at critical sections of columns, involves the development of vertical cracks parallel to the column bars. For these cracks to develop, it can be shown that a second crack surface inside the column bars and parallel to the plane of reinforcement must develop to facilitate the lateral dilation implied by the vertical cracks. For failure to occur, it may be hypothesized that the area associated with a column bar must separate from the concrete attached to the starter bar, as shown in Figure 19.

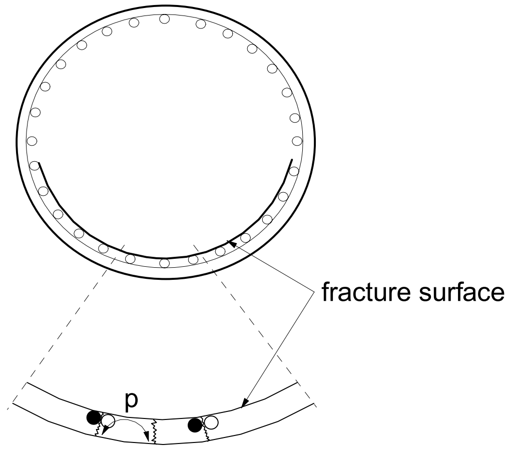


Figure 19. Failure mechanism of lap-splices in circular columns

The tensile stress necessary to fracture this surface may be assumed equal to the direct tension strength:

$$f_t = 0.33\sqrt{f'_c} \tag{23}$$

For a circular column with n longitudinal bars, cover c, bar diameter d_b, rebar pitch circle diameter D' and lap length l_s, the total tensile force on the rupture surface at failure will be

$$T_b = f_t \left[\frac{\pi D'}{2n} + 2(d_b + c) \right] l_s = f_t \; p \, l_s \tag{24}$$

where p is the perimeter of the crack surface. The parameter T_b is the maximum force that can be developed in the column bar unless significant transverse confinement is present. If $T_b > A_b F_y$, the bar can develop its yield force, and the initial ideal flexural strength may be developed. If $T_b < A_b F_y$ bond failure will occur at less than the flexural strength, with rapid strength degradation under cyclic loading. Even if (24) indicates that the bar strength can be fully developed, available ductility will be small. Bond failure seems inevitable in poorly confined lap-splices when these are cyclically loaded.

However, if adequate confinement is available, the cracks can be contained, and the frictional resistance across the crack provided by the clamping pressure may be sufficient to develop the bar strength. Tests indicate that there is a limit to the dilation strain that can be permitted before bond slip occurs. This strain is approximately $\varepsilon_d = 0.001$. To ensure that bond failure does not develop regardless of ductility level, the confining stress corresponding to a radial dilation of $\varepsilon_d = 0.001$ should be sufficient to develop the tensile strength of the bar. For circular columns with transverse hoops or spirals of cross sectional area A_b at spacing s, the effective confining stress at $\varepsilon_d = 0.001$, assuming $E_S = 200$ GPa will be

$$f_l = \frac{400 A_b}{D' s} \quad \text{MPa} \tag{25}$$

where D' is the diameter of the hoop or spiral. Allowing a coefficient of $\mu = 1.4$ on the crack surface, the tensile force capable of being developed in the bar is, by analogy with (24):

$$T_b = 1.4 f_l \left[\frac{\pi D'}{2n} + 2(d_b + c) \right] l_s \tag{26}$$

For no bond failure at high ductility's,

$$T_b \geq A_b\, f_u \approx 1.5 A_b\, f_y \tag{27}$$

Combining the two previous equations, a formula for the minimum required effective confinement pressure to inhibit splice failure is obtained:

$$f_l \geq \frac{1.5 A_b\, f_y}{1.4\left[\frac{\pi D'}{2n} + 2\left(d_b + c\right)\right] l_s} \tag{28}$$

Where A_b is the cross sectional area of one longitudinal bar.

There is a lower limit to the lap length l_S below which bond failure will develop regardless of the confining stress provided by transverse reinforcement. Results from tests indicate that the effective bond stress may be taken as:

$$\mu_u = 1.5\sqrt{f'_c} \quad \text{MPa} \tag{29}$$

Thus the minimum splice length for which it should be possible to develop the ultimate bar strength is

$$l_{s,\min} \cong \frac{1.5 A_b\, f_y}{\pi d \mu_u} \cong \frac{0.25 d\, f_y}{\sqrt{f'_c}} \tag{30}$$

To develop just the bar yield strength, the coefficient should be 0.167 rather than 0.25. Eq. (30) may also be used to determine the sufficiency of anchorage of bars in different characteristic locations, such as in knee-joints. If a column plastic hinge is expected at the top, eqs. (24), (26) and (27) will apply. If the column will not form a plastic hinge at the top, the yield strength, rather than the ultimate strength of the bar, may be used in (27).

As mentioned above, splice failure can be predicted by an assessment of the tensile stress capacity across a potential splitting failure surface. After cracking develops on this interface, splice failure can be inhibited providing adequate confining pressure is assured without excessive dilation. Tests have indicated that the critical radial dilation strain is ε_d = 0.001. Priestley et al. (1994) derived the following well accepted expression, which is derived from (28), for calculation of the minimum required confinement pressure, to avoid bond failure of lap-spliced longitudinal reinforcement:

$$f_l \geq \frac{A_b\, f_y}{\left[\frac{\pi D'}{2n} + 2\left(d_b + c\right)\right] l_s} \tag{31}$$

where d_b, A_b and f_y are the diameter, area and yield stress of one spliced longitudinal bar, D' is the pitch circle diameter of the reinforcement, n is the number of bars, c is the cover concrete thickness and l_s is the starter bar length. This confinement pressure should be available at the critical radial strain of ε_d = 0.001, and can be related to the characteristics of the used retrofit concept:

$$f_l = \frac{\rho\, f_s}{2} \tag{32}$$

$$f_s = E_j\, \varepsilon_d \tag{33}$$

$$\rho = \frac{4t_j}{D} \tag{34}$$

Therefore, the minimum required effective thickness of a passive steel or FRP jacket can be expressed as:

$$t_j \geq \frac{Df_l}{2E_j\,0.001} = \frac{500Df_l}{E_j} \tag{35}$$

where E_j is the effective modulus of elasticity in the direction of the transverse section dilation. This relation is valid for both steel and FRP confined lap-spliced sections. When this equation is believed to be correct, it is clear that a passive fiberglass jacket, and in some extent also a carbon jacket, are not the best solutions, due to the (much) lower stiffness in comparison to steel. Consider a fiberglass jacket with E_j = 40,000 MPa, then a thickness 5 times larger than with a steel jacket is needed. When FRP-materials are used, it is clear that an active layer is more convenient. For a combination of an active and passive FRP jacket, (35) becomes:

$$t_a\,E_a + t_p\,E_p \geq 500D\left(f_l - f_a\right) \tag{36}$$

where t_a, t_p, E_a and E_p are the thicknesses and moduli of elasticity of the active and passive layers respectively, and f_a is the active confining stress induced in the column.

According to Priestley et al. (1994) tests indicate that where ratios of longitudinal reinforcement do not exceed 2.5 % and axial load ratios are less than ν= 0.15, the following approximate approach will provide satisfactory performance.

4.3.2 Confinement of plastic hinges with lap-splices

Provide f_l = 2.0 MPa at a dilation strain of ε_d = 0.001. With this, the minimum effective required thickness of a jacket can be expressed as:

for steel and passive FRP: $$t_j \geq \frac{Df_l}{2E_j\,0.001} = \frac{1000D}{E_j} \tag{37}$$

for active/passive FRP: $$t_a\,E_a + t_p\,E_p \geq 500D\left(2 - f_a\right) \tag{38}$$

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