The required storey stiffness K of the retrofitted structure that comprisestRcRC jacketed columns, I_w RC walls, I_x spans of X-brace metallic pairs, I_mw masonry walls and I_cf columns strengthened with longitudinal FRP strips(EBR or NSM)is equal to:

$$K_i = \sum_{j=1}^{\ell cRC} K_j^{J} + \sum_{j=1}^{\ell_w} K_j^{w} + \sum_{j=1}^{\ell_x} K_j^{X} + \sum_{j=1}^{\ell_{mw}} K_j^{mw} + \sum_{j=1}^{\ell_{cf}} K_j^{f} \tag{8–2}$$

The contributions of each of these techniques/elements to the storey stiffness,K,are listed in a study[6].7 and are summarised here for completeness in Appendix 8.1.Only the possible contribution of FRP to the stiffness terms, K_i^f,is considered in the following detailing paragraphs in the main body of the chapter.

In Eq.(8-2),variables tRctw,tmwandt.are defined as follows:ERc is the number of columns retrofitted with RC jackets in a single storey;lwis the number of RC walls added for stiffening the structure in the direction of action;I is the number of X-brace pairs added in the storey to add stiffness-in the direction of action;tmw is the number of infill masonry panels added in the storey in the direction of action;and I is the number of columns strengthened with longitudinal FRP strips(externally bonded or NSM).

8.2 Practical implementation of global measures

From the preceding discussion it follows that in practical implementation the displacement demand and the pattern of its distribution may be essential prerequisites to the application of externally applied FRP for seismic retrofitting.Steps in this direction are the following5: (1)determine if a global intervention is required,(2)target for an improved period estimate, (3)target for an improved shape of the fundamental mode,and (4)determine the required stiffness.Detailed considerations on these prerequisite steps are given in Appendix 8.2.

8.3 Strategies in FRP interventions for seismic applications

Seismic retrofitting of RC structures with FRP may be used in order to upgrade a variety of structural deficiencies,if upon assessment according to the established code framework (EN 1998-32005)[8]it is shown that seismic safety may be compromised at the design performance limit state.Both for evaluating the structure's safety and in defining the retrofit objectives,reference is made to verification of acceptable limit states as described in the reference code document.

Similarly,the seismic hazard considered for the retrofit is identical to that used for new designs,unless the National Standards enable through special provisions,assigning a different importance level category to the retrofitted structure so as to account for a residual service life different from the 50-year standard.

Analysis of the retrofitted structure may be used to check against the established acceptance criteria,following the methods of analysis used in the assessment procedure.

Material safety factors refer to FRP materials typically used today(e.g.GFRP,CFRP and AFRP with strengths ranging from 1500 to 3500 MPa and nominal rupture strains from 2.5% down to 1.5%).For retrofit design these are:(a)For existing concrete and steel reinforcement, the confidence factors are used to divide mean material strength values depending on the knowledge level attained (EN 1998-32005)8.(b)For FRP:The material safety factor depends on the application method of the FRP material and the member classification, "primary" or "secondary" as per EN 1998-19, as listed in Table 8-1 (according to EN 1998-1 secondary members are those members whose stiffness and resistance account for less than 15% of the total storey stiffness, and hence they may be neglected in the response analysis, even though they shall be designed to withstand the deformations of the structure under the design seismic loads without loss of vertical load carrying capacity).

Safety factors exceed the corresponding values which were adopted in Chapter 3 for accidental actions ($\gamma=1$); the reason for this difference is that in the performance-based assessment and retrofit adopted in the Seismic Provisions, the performance limit state corresponds by design to significant inelastic deformations. Note that the deformation capacity of the member in FRP-based retrofits depends on the resilience of the retrofit scheme at displacement ductilities that easily exceed the value of 3.5, and corresponding curvature and confined material strain ductility in the order of 6 or higher.

Table 8-1 FRP material safety factors γ

FRP is anchored on:	Primary member	Secondary member
Brittle substrate	3.0	
Fully wrapped FRP layer (i.e. anchorage by lap-splicing the ends of the layer in closed jacket)	1.5	1.25

*This is the case where the FRP material terminates at, and is anchored on concrete or masonry; e.g. longitudinal FRP reinforcement in beams or slabs, and two-sided or three-sided jacket layers. The safety factor is very high in this case if the anchorage is unconfined; the designer may effectively reduce the safety factor thereby using a larger fraction of the material's strain capacity through the multipliers a_1 and a_2 as a benefit of good practice [by adequately confining the anchorages through placement of transversal EBR confining layers or chemical anchors, see Eq.(8-7) for primary reinforcement, and Eq.(8-15) for jacketing].

8.3.1 Objectives in FRP retrofitting

The FRP material to be used in the retrofit solution and its arrangement depend on the overall objectives of the retrofit design. A general guideline is to aim for a uniform distribution of strength and stiffness among members in any given storey, in order to minimise the risk of disproportional damage of any single element. [The curvature at yielding of a linear RC member is approximately equal to $\varphi_y=2\varepsilon_y/h$, where h is the cross section depth and ε_y the characteristic yield strain of the reinforcing steel. Thus the chord rotation (see definition in Section 8.3.2.3.1) at yielding is $\theta_y=\varphi_y\cdot H/6=\varepsilon_y(H/h)/3$, where H/h is the aspect ratio of the member. Therefore two members having very different aspect ratios yield at very different relative drift ratios.] The implication is that during an earthquake, for any given magnitude of lateral displacement, members with different aspect ratios in a single storey reach very different states of damage. The same effect is observed if the structure has plan irregularities that cause torsional response. Clearly, major building irregularities cannot be eliminated using FRP as a strengthening technique, although the addition of FRP strips as longitudinal reinforcement belongs to the global interventions as it can be used to increase strength and stiffness of individual members.

Thus, a good strategy is to selectively retrofit members that belong to the lateral load resisting system so as to achieve similar relative drift ratios at yielding, and to also enhance deformation capacity through confinement. It is essential to eliminate brittle failure modes through FRP jacketing, so that the flexural capacity of the member may be fully developed and sustained up to the ductility level required by the design.

Extensive experimental evidence supports the use of FRP as a suitable material in seismic retrofitting applications, particularly in reinforced concrete beams, columns, walls and beam-column connections10.11,12.FRP retrofit schemes that are well documented and support the establishment of detailing rules include the following solutions:

1.Increasing the capacity of beam-column joints to diagonal tension by using FRP material installed with fibres located along the directions of principal tensile stresses.

2.Increasing the member shear capacity by using FRP material with fibres running orthogonal to the direction of the axis of the strengthened member.

3.Increasing the rotational capacity and the ductility of end sections of beams and/ or columns by using FRP material wrapped around the member cross section.

4.Improving the efficiency of lap splices by using FRP material wrapped around the member cross section.

5.Delaying the occurrence of buckling of steel longitudinal bars by using FRP material wrapped around the member cross section.

6.Increasing the flexural strength of a linear member by using externally bonded or near-surface mounted FRP strips in the role of primary reinforcement -that is,by the addition of reinforcement running parallel to the longitudinal axis and attached near the tension side of the strengthened member.

As interventions(1)-(5)listed above cannot significantly alter the reference flexural strength and stiffness of the retrofitted member,all these techniques are classified as local measures (or local interventions).It is required that localised strengthening shall not reduce the overallductility of the structure.For all FRP local interventions and particularly for the strengthening of beam-column joints,in-situ supervision and quality control are crucial in order to ensure that installation procedures are carefully adhered to so as to ensure that performed interventions are fully effective as theoretically expected.

In detailing the retrofit solutions each retrofited member is designed using principles of Capacity Design.To secure adequate ductility,flexural yielding should control the response of the retrofitted member.So the member retrofit details should be proportioned with reference to flexural overstrength.The shear force associated with flexural yielding of the member,from the static relationship depicted in Fig.8-1,is referred to as flexural shear demand,Yilex

When considering individual members,case(6)in the list of intervention measures given is a global measure,effectively increasing VeOn the other hand,local strengthening schemes of individual linear members,i.e.cases(1)-(5)above,have relatively little effect on Vnex and they depend on the confining action of the FRP reinforcement.Thus,the efficacy of the strengthening scheme in these cases depends on the magnitude of the confining pressure.To illustrate the procedures for detailing,the role of the FRP properties in each mechanism of resistance associated with the strengthening objectives of individual members listed above, will be reviewed briefly in the following.

In order to perform the necessary design calculations and to control the failure modes,a static model is envisioned for each member undergoing retrofit depicted in Fig.8-1.In this static model,the member develops constant shear force along its length reaching maximum flexural moment at the end points of its deformable length where partial restraint to rotation may exist.For example,during lateral sway,a column is considered to develop maximum moment at the base and at the top cross section at the beam soffit;a beam is considered to develop maximum moments at the end supports,at the face of the columns;a flexural wall is considered to develop maximum moment at the base.

When using FRP materials for retrofitting RC members, the following principles shall be taken into account:

- Removal of all brittle collapse mechanisms (Section 8.3.2.1).
- Removal of all storey collapse mechanisms ("soft storey") (Section 8.3.2.2)
- Enhancement of the overall deformation capacity of the structure, to exceed the displacement demand by the design seismic hazard, through one of the following response mechanisms (Section 8.3.2.3):
  - Increasing the rotational capacity of the potential plastic hinges without changing their position (Section 8.3.2.3.1)-(Scenario I).
  - Relocating the potential plastic hinges implementing capacity design principles (enforcing a hierarchy of resistances at convergent members, e.g. strong column -weak beam connection, weak column-strong foundation connection and so on) (Section 8.3.2.3.2)-(Scenario II).
- Through proper detailing measures to secure resiliency of the retrofit against debonding and local rupture.

To implement these FRP retrofitting strategies, global displacement demands need to be determined (Section 8.2 and Appendix 8.2) and to be subsequently converted to local deformation demands (Section 8.3.2) of the members to be retrofitted through the above scenarios.

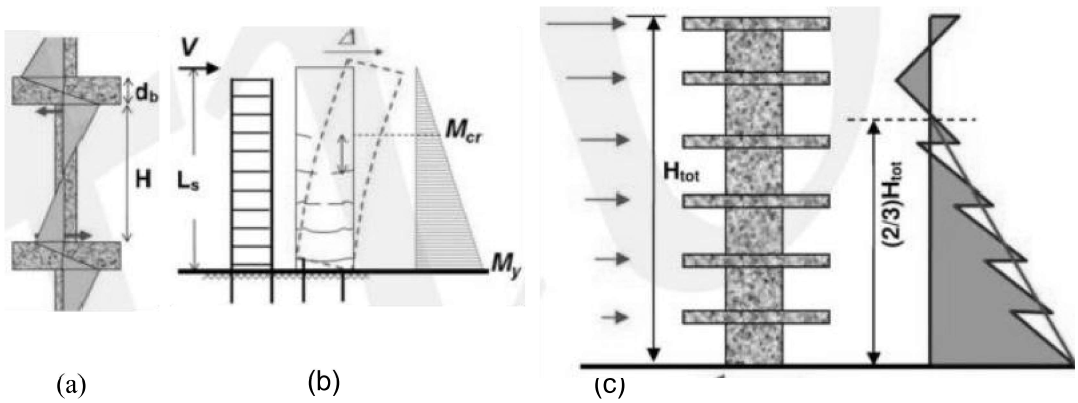


Fig. 8-1: (a) Static model used for beam-column elements undergoing lateral sway. (b) The cantilever part to the right has the same moment distribution as the swaying column over the length from the point of contra-flexure to the face of the support. Owing to this similarity the cantilever member is used to illustrate the concept of shear span, L=H2, and the relationship between shear demand and flexural moment strength: V=M/L. (c) Static model for structural walls. The straight line defines an "equivalent" linear moment diagram to relate shear demand with moment strength: V=M/(2H/3)

8.3.2 Determining the displacement demand of the individual structural members

The global retrofit objectives are defined in terms of target fundamental period, target response shape and global behaviour factor, q.

With reference to Fig. A8.2-1, it is recommended that q should not exceed the value of 2.5 for ordinary structures. Higher values should be avoided if the structure has been designed to previous standards, or has irregularity in plan or in height. Lower values of q are acceptable.

To illustrate how global considerations of the retrofit design may be used to determine local design requirements, the following steps are considered:

-The required displacement ductility μ of the structure may be estimated from:

$$\mu_\Delta = \begin{cases} q & \text{for } T>T_C \\ 1+\frac{T_C}{T}(q-1) & \text{for } T<T_C \end{cases} \tag{8-3}$$

where T_C is the end of the plateau of the Type I spectrum for the design soil conditions EN 1998-12004[9]. Depending on the target response shape, required displacements at the individual floors of the structure are determined from EN 1998-38:

$$\mu_{\Delta,i} = \mu_{\theta,i} = \frac{\Delta_i}{\Delta_{y,i}} = \frac{\mu_\Delta \cdot \Delta_y \cdot \Delta\Phi_i}{\Delta_{y,i}} \tag{8-4}$$

where φ,is the coordinate of the target response shape in the i-th floor and $\Delta\Phi=\Phi-\Phi_1$.

-The required curvature ductility at the critical sections of members in the i-th floor may be obtained with reasonable approximation from:

$$\mu = 2\mu - 1 \tag{8-5}$$

whereas the maximum compression strain demand for the columns, εcu, may be estimated from GRECO[1]3:

cuc=2.2μ。E,·Vdmax≥0.0035 (8-6)

In Eq.(8-6), parameter vamxis the maximum axial load ratio of the typical column for the seismic combination (defined in Section 8.4.1) and e is the yield strain of steel. Using the calculated compression strain demands, the required amount of confining reinforcement may be obtained from pertinent stress-strain models for FRP-confined concrete, which relate the thickness of the FRP jacket with the compression strain capacity of encased concrete.

8.3.2.1 Removal of all brittle collapse mechanisms

Brittle collapse mechanisms to be removed as well as FRP strengthening methodologies are as follows:

-Shear failures shall be avoided; all members that present shear deficiency shall be strengthened.

-Failure due to loss of bond in steel overlapping areas; the areas where overlapping length of the longitudinal bars is not enough shall be confined with FRP wrapping.

-Failure due to buckling of steel longitudinal bars; areas where plastic hinges are expected to form shall be confined with FRP wrapping when existing steel transverse reinforcement cannot prevent the post-elastic buckling of compressed longitudinal bars.

-Failure due to tensile stresses in beam-column joints; FRP strengthening shall be applied.

8.3.2.2 Removal of all storey collapse mechanisms

Storey collapse mechanisms usually occur following the formation of plastic hinges at column top and bottom locations of structures with no vertical walls (e.g. in swayframes with flat slabs). In such a case, FRP strengthening can be performed to enhance the column flexural capacity with the intent of precluding the formation of plastic hinges. In no case should the removal of the storey collapse mechanisms be guided by the sole objective of increasing the capacity for higher storey displacements.

8.3.2.3 Enhancement of the overall deformation capacity of a structure

The ultimate deformation capacity of a structure is a measure of its capability to sustain seismic forces while carrying its self-weight. This capacity depends on the plastic deformation capacity of each single resisting member (beams, columns and walls) and can be computed with nonlinear static analysis methods (push-over analysis).

8.3.2.3.1 Increasing of the local rotational capacity of RC members

The deformation capacity of beams and columns may be measured through the rotation θ of the end section with respect to the line joining the latter with the section of zero moment (chord rotation) at a distance equal to the shear span L_s=M/V (in buildings with a "shear" type mode of lateral deflection this rotation is also equal to the ratio of the relative displacement between the two above mentioned sections to the shear span, referred to as relative storey drift ratio; however, if beams also participate in the deformation of the storey, then the relative drift ratio defined in the preceding far exceeds the column rotation due to the rigid body rotation of the base as depicted in Fig. 8-7).

The deformation capacity of RC members in the plastic range is limited by the failure of compressed concrete. FRP confinement increases the ultimate deformation of compressed concrete and enhances the ductility of the strengthened member.

8.3.2.3.2 Capacity design criterion

The application of the capacity design criterion (hierarchy of resistance) implies the adoption of mechanisms of behaviour in the structure such as to prevent by design the formation of all potential plastic hinges in the columns. In "weak column - strong beam" situations, which are typical of structures designed for vertical loads only, columns are under-designed due to lack of longitudinal reinforcement. In such a case, it is deemed necessary to increase the column capacity under combined bending and axial load toward a "strong column-weak beam" situation.

8.4 FRP as a means of enhancing strength and deformation capacity

8.4.1 Increasing flexural strength of RC members by adding longitudinal FRP reinforcement

With the addition of FRP strips on the tension side of a member parallel to its longitudinal axis the objective is to enhance flexural strength and, to a limited degree, stiffness of the member. In this capacity the FRP reinforcement works as tension reinforcement; if upon reversal of the load, the FRP layers fall within the compression zone of the member, then their contribution to strength may be neglected, until adequate data exist to corroborate any other design decision.

Steps for detailing the retrofit: Consider the cross section shown in Fig. 8-2: A prismatic cross section with known initial geometry and material properties is given. The cross section carries an average design axial load, NG+0.3Q-E≤Nc+0.3Q≤Nc+0.3Q+obtained from the seismic design combination. For this axial load, which is used to calculate the axial load ratio v:a[v=Nc+0.3Q/(f:b·d)], and the longitudinal reinforcement ratios, p, and p_{32} the reference flexural strength of the cross section is MR。(moment and axial load are considered to be acting at the centroid of the cross section).

To increase the flexural strength up to a required value MEFRP strips shall be either externally bonded or embedded in near-surface grooves in the tension sides of the member as shown in Fig.8-2. The added reinforcement may be confined by a transverse jacket (this should be provided if the shape of the cross section allows it). Figures 8-2 and 8-3 depict all possible combinations for illustration purposes-i.e. NSM or EBR longitudinal strips, either confined by a transverse jacket (see top region in cross sections in Fig.8-2), or left unconfined (see bottom region in cross sections of Fig.8-2). The maximum allowable stress of the longitudinal FRP depends on the chosen arrangement as explained below.

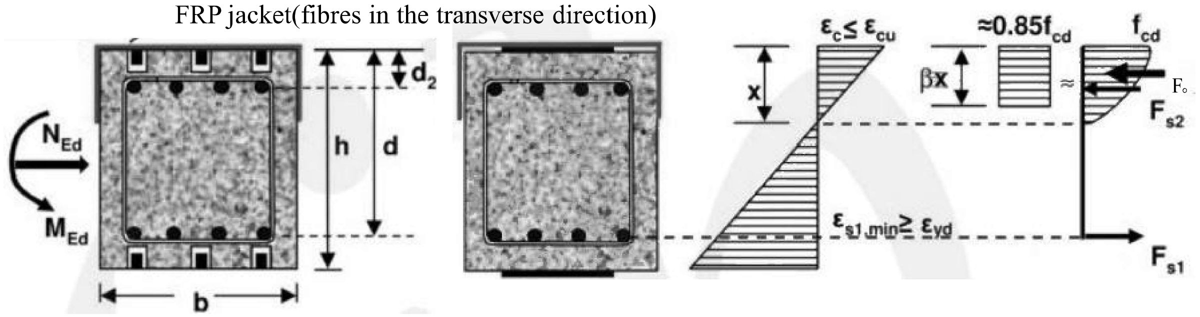


Fig.8-2 Various types of flexural strengthening of prismatic column/beam cross section. The U-shaped line illustrates the jacket arrangements implied by the various values of a, and a_2 in Eq.(8-7). Clearly this should not be interpreted as a recommended arrangement of the jackets. Wherever possible the jacket should be wrapped fully around the cross section

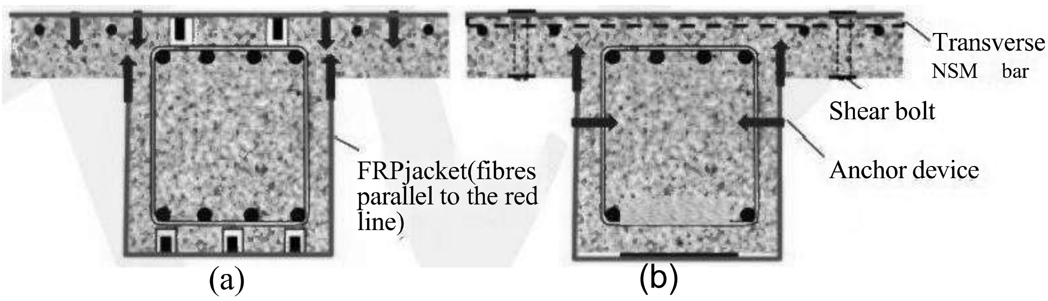


Fig.8-3 Various types of flexural strengthening of T-beam cross sections by addition of longitudinal NSM or EBR reinforcement. The outer solid line illustrates jacket arrangements required to secure the flexural intervention. Due to the presence of slab longitudinal reinforcement, addition of top reinforcement is more rarely needed [{see(a); here it is important to provide transverse top reinforcement to secure the participation of slab reinforcement in beam flexural strength [see dashed line in(b)].

Essential requirements for this type of retrofit are:

1. The extreme layer of embedded longitudinal tension steel reinforcement should undergo yielding at the ultimate limit state(es1,min>Ey).

2. Maximum compressive strains in unconfined concrete in the compression zone, ε, shall not exceed εcu·

3. Tension strains in the FRP longitudinal reinforcement, ε, shall not exceed the design limit εd

For (simplified) dimensioning of the longitudinal reinforcement, the axial design tensile strain εin the FRP layer shall not exceed the limit:

$$\varepsilon_{fd}=\min\left\{a_1\cdot a_2\frac{\varepsilon_{fuk}}{\gamma_f};\eta\cdot\varepsilon_{fuk}\right\} \tag{8-7}$$

where

a_1=1.0 for EBR FRP.

a_1=1.4 for NSM FRP.

a_2=1.0 if no transverse confinement has been applied over the FRP reinforcement.

a_2=1.4 if transverse confinement has been applied over the FRP reinforcement.

η=as defined in Section 3.6.2 [the second term in Eq.(8-7) is enforced as an upper limit in the design strain as per Section 3.7.5 for accidental combinations; sfuk is the characteristic strain of the FRP material].

Y,=3.0 (taken from Table 8-1 for primary reinforcement on a brittle substrate).

For the sake of illustration, the dimensioning procedure is considered for the case when the cross section fails as a result of concrete crushing (e.=ε。). The procedure for the case of FRP failure follows similar principles.

Requirements 1-3 above may be expressed as limits on the normalised depth of compression zone of the cross section after retrofit, ξ=x/d:

$$\frac{\varepsilon_{cu}}{\varepsilon_{cu}+\varepsilon_{fd}}\left(1+\frac{d_2}{d}\right)\le\xi\le\frac{\varepsilon_{cu}}{\varepsilon_{cu}+\varepsilon_{yd}}=\xi_{bal} \tag{8-8}$$

To calculate the required area of the added tension reinforcement A,=p;b·d, the values of ξand p are solved from the equilibrium requirements:

-Sum of axial forces =0(v>0 for compression)

0.85f·β·ξ+(P,2-p3)f-Veaf =p.·E,·E→ (8-9)

$$\rho_f=\frac{0.85\beta\cdot\xi-\left(\rho_{s1}-\rho_{s2}\right)\frac{f_{yd}}{f_{cd}}-v_{Ed}}{\frac{f_{fd}}{f_{cd}}}\Rightarrow\rho_f=\frac{0.85\beta\cdot\xi-\left(\rho_{s1}-\rho_{s2}\right)\frac{f_{yd}}{f_{cd}}-v_{Ed}}{\frac{E_f\cdot\varepsilon_{cu}}{f_{cd}\cdot\xi}\left(1+\frac{d_2}{d}-\xi\right)}$$

Parameterβis the depth of the equivalent rectangular stress block of concrete compressive stress, normalised by the depth of compression zone, x. The intensity of the stress block is assumed equal to 0.85f

– An upper limit is obtained for p, by substituting $\xi=\xi_{bal}$ in the above. For f_{yk} = 500 MPa (andε=0.0035), 5ba=0.0035/(0.0035+0.0025/1.15)=0.62; forfk=400 MPa ξ_{bal} = 0.0035/(0.0035+0.002/1.15)=0.66; and for f=220 MPa ξba=0.0035/(0.0035+0.0011/1.15)=0.78.

- For the retrofit to be possible it is also required that:

$$0.85\beta\cdot\xi_{bal}\ge v_{Ed}+\left(\rho_{s1}-\rho_{s2}\right)\frac{f_{yd}}{f_{cd}} \tag{8-10}$$

To find the required area of the FRP in order for the strengthened cross section to have a flexural strength of Md>Min the presence of a design axial load $N_{Ed} = v_{Ed} \cdot b \cdot d \cdot f_{cd}$,the following procedure is used.First,the sum of moments is considered about the centroid of the FRP:

M:a;=Ma+N:a(h−y。)=HE b • d^2 • f. (8-11a)

where y is the distance from the centroid of the cross section to the extreme compressive fibre.Note that both MEand the normalised value μHEdare defined with reference to the centroid of the FRP layer (this is the significance of the subscript"f").Mea is calculated first,given the design values of moment and axial load of the retrofitted cross section. Next,μed is obtained from Mafter normalizing with $b \cdot d^2 \cdot f_{cd}$. This is set equal to the normalised moment of internal forces about the same point of reference,given by:

$$\mu_{Ed,f} = 0.85\beta \cdot \xi\left(1+\frac{d_2}{d}-0.4\xi\right)+\left(\rho_{s2}-\rho_{s1}\frac{d_2}{d}\right)\frac{f_{yd}}{f_{cd}} = \tag{8-11b}$$

$$0.85\beta \cdot \xi\left(1+\frac{d_2}{d}-0.4\xi\right)+\mu_{Ro}$$

where term μR in Eq.(8-11b)only depends on geometric and mechanical characteristics of the original cross section.The FRP layer is calculated so that the total value for HEd meets the strength demand of the retrofitted cross section.The first term in the right-hand side of Eq.(8-11b),given by:

$$\Delta\mu_{Ed,f} = 0.85\beta \cdot \xi\left(1+\frac{d_2}{d}-0.4\xi\right) \quad \textit{where } \triangle Hedj=Heds\text{-}HR \tag{8-11c}$$

is tabulated for easy reference in Table 8-2 for usual values of d_2/d.

Table 8-2 Normalised moment △u:for various values of maximum allowable FRP strain ε

d_2/d	0.05		0.10		0.15	
Eid	ξ	△!Lai	ξ	△μai	ξ	△Hadi
0.0022	0.645	**0.347**	**0.675**	**0.381**	**0.706**	**0.417**
0.0024	0.623	**0.339**	**0.652**	**0.372**	**0.682**	**0.407**
0.0026	0.602	**0.331**	0.631	**0.364**	0.660	**0.398**
0.0028	0.583	**0.324**	0.611	**0.356**	0.639	**0.389**
0.0030	0.565	**0.317**	0.592	**0.348**	0.619	**0.380**
0.0035	0.525	**0.300**	0.550	**0.329**	0.575	**0.360**
0.0040	0.490	**0.285**	0.513	**0.312**	0.537	**0.341**
0.0045	0.459	**0.271**	0.481	**0.297**	0.503	**0.325**
0.0050	0.432	**0.258**	0.453	**0.283**	0.474	**0.309**
0.0055	0.408	**0.246**	0.428	**0.270**	0.447	**0.295**
0.0060	0.387	**0.236**	0.405	**0.258**	0.424	**0.282**
0.0065	0.368	**0.226**	0.385	**0.248**	0.403	**0.271**
0.0070	0.350	**0.217**	0.367	**0.238**	0.383	**0.260**
0.0075	0.334	**0.208**	0.350	**0.228**	0.366	**0.250**
0.0080	0.320	**0.200**	0.335	**0.220**	0.350	**0.240**
0.0085	0.306	**0.193**	0.321	**0.212**	0.335	**0.232**

0.0090	**0.294**	**0.186**	**0.308**	**0.205**	**0.322**	**0.224**
0.0095	0.283	**0.180**	0.296	**0.198**	0.310	**0.216**
0.0100	0.272	**0.174**	0.285	**0.191**	0.298	**0.209**
0.0105	0.263	**0.169**	0.275	**0.185**	0.288	**0.202**
0.0110	0.253	**0.163**	0.266	**0.179**	0.278	**0.196**
0.0115	0.245	**0.159**	0.257	**0.174**	0.268	**0.190**

Values in Table 8-2 are calculated by taking βequal to 0.8,which corresponds to the ultimate strain of the extreme compressed fibre of the cross section, $\varepsilon_{cu} = 0.0035$.Ranges of parameters outside the table represent cases where the resulting normalised depth of compression zone,ξ,does not satisfy the limits set by Eq.(8-8),and therefore this type of strengthening would not be advisable as it will embrittle the cross section [for ξvalues above the upper limit of Eq.(8-8),in the shaded part]or it willlead to debonding along the anchorage of the added FRP reinforcement [for ξvalues below the lower limit of Eq.(8-8)]. The Table 8-2 is entered with the value of d_2/d and the required normalised moment increase △u:(calculated about the centroid of the longitudinal FRP).The estimated value of ξ(from the table)is substituted in the expression for p,[Eq.(8-9)]to yield the required area of FRP tension reinforcement.The strain that develops in the FRP is given in the left column,which may be checked against the allowable ε value determined from Eq.(8-7).The shaded part indicates concrete crushing for embedded tensile reinforcement with $f_{yk} = 500$ MPa;values are increasingly conservative in the range of higher FRP strains,i.e.,for lower ξ values,to account for the fact that the actual stress in compression reinforcement is lower than the assumed yielding limit.More strict criteria(e.g.a minimum required value of tensile strain in the extreme layer of steel reinforcement $\varepsilon_{s1,min} = 0.004$)will extend the shaded portion of the table downwards.

Additional requirements:

-FRP reinforcements used as post-installed primary reinforcement to strengthen RC structural members for seismic applications shall be anchored so as to be able to support their force at the critical section where moment is maximum (e.g.at the upper and lower cross sections over the free length of a column,at the end cross sections of a beam,and at the base cross section of a structural wall).The required effective anchorage length available on each side of a critical section shall satisfy the requirements for development length given in Chapter 6.

-When FRP reinforcement is used to increase the flexural capacity of a member, it is important to verify that the member will be capable of resisting the shear forces associated with the increased flexural strength.

-The increased flexural strength corresponds to an increased flexural shear demand, VEa=MRd/L,(Fig.8-1).The retrofitted member shall be checked in shear,and additional shear reinforcement shall be provided to ensure that factored shear resistance,VRd exceeds V

-The stiffness increase attained by the addition of the FRP reinforcement may be quantified by the magnitude of the effective E·I,of the j-th member's strengthened cross section,associated with the onset of yielding of the embedded longitudinal tension steel.Thus,the translational stiffness of a column is:

$$EI_j = \frac{M_{y,j}}{2\varepsilon_{sy}/h} \qquad K_j = 12\frac{E\cdot I_j}{H_j^3} = 6\frac{h}{H_j}\frac{M_{y,j}}{\varepsilon_{sy}\cdot H_j^2} \tag{8–12}$$

Note that H /h is the aspect ratio of the member and M,is the moment resistance of the j-th strengthened member, at the onset of yielding of longitudinal reinforcement.

Additional detailing considerations:

-When the member flexural capacity is increased, particular care shall be taken to properly anchor the adopted FRP reinforcement, as outlined in Sections 6.2.1(EBR) and 6.2.2(NSM).

-Longitudinal fibres used for strengthening RC members subjected to combined bending and axial load shall be properly confined to avoid debonding and concrete spalling under cyclic loads.

8.4.2 Increasing the deformation capacity of reinforced concrete members through FRP jacketing

To increase the deformation capacity of an RC member any type of undesirable brittle failure should be eliminated. The member shall be designed to develop ductility during seismic load reversals. Ductility is achieved if the longitudinal steel reinforcement of the member is engaged in post-yielding response prior to the occurrence of any of the following:

1. Delamination of concrete cover in the compression zone.
2. Failure of lap-splices or reinforcement anchorages.
3. Diagonal tension failure of the member's web (shear).
4. Bar buckling in the compression zone.
5. Disintegration of the confined concrete core under high compression strain demands.

FRP jacketing may be used to effectively eliminate these occurrences and also to enhance the deformation and ductility capacity of a reinforced concrete member. The term FRP jacketing refers to any type of application of the material where the primary fibres are oriented transversally to the longitudinal axis of the upgraded member and at a minimum of three faces (U-shaped and closed types exclusively) of the member's cross section so as to provide confining action against any dilation of the concrete (i.e. due to axial load, shear transverse tension or dilation produced by the bond action of a ribbed bar). Interventions that may be necessary to achieve this objective were termed local measures, and were listed in Section 8.3.1. A critical design parameter in all cases is the confining pressure introduced by the FRP jacket.

The confinement model adopted in the following sections is not the same as the one adopted in Chapter 6, which is applicable in the analysis of columns under nearly concentric loading, i.e. where flexure is limited. In fact, all confinement models listed in the literature, including the one adopted in Chapter 6, have been calibrated against a very large database of tests conducted on axially compressed members (Fig. 8-6a). Specimens were either reinforced or unreinforced. Tests conducted correspond to the point where the axial load-moment interaction diagram intersects the axial load axis (Fig. 8-6c). As already discussed in Chapter 6, the derived stress-strain relationships do not account for the strain gradient effects that occur due to flexural moments, hence the confinement models adopted in this chapter is not the same as that in Chapter 6, which applies to columns under predominantly axial loading.

8.4.2.1 Calculation of confining pressure in FRP-encased concrete

The confining pressure exerted by the FRP jacket encasing a reinforced concrete member is estimated with reference to Fig.8-4(see also Fig.6-12).FRP stresses and confinement exerted on the encased cross section vary from the corners to the centre.The average confining pressure,σ,acting along the x axis,may be estimated considering equilibrium on a plane intersecting the cross section along line A-A.Similar is the calculation of the average confining pressure,σ,acting in the y direction.

σ、=PhvxE,·EGuh+Pswxfy.s (8-13a)
FRP component contribution of links

σy=PwyE·Euh+Pswy yst (8-13b)
FRP component contribution of links

where phx is the FRP web reinforcement ratio provided in the x direction(2t,/b)for continuous jacket having an effective thickness of t.Similarly,Prwy=2t,/h.The effective thickness is calculated from the number of FRP layers placed in the jacket,n,and the thickness of a single layer,t,ast=n·t。for n=1,2 or 3,and ast,=n0.85t。forn≥4.Hence, the reduced effectiveness of a jacket with many layers is taken into account.

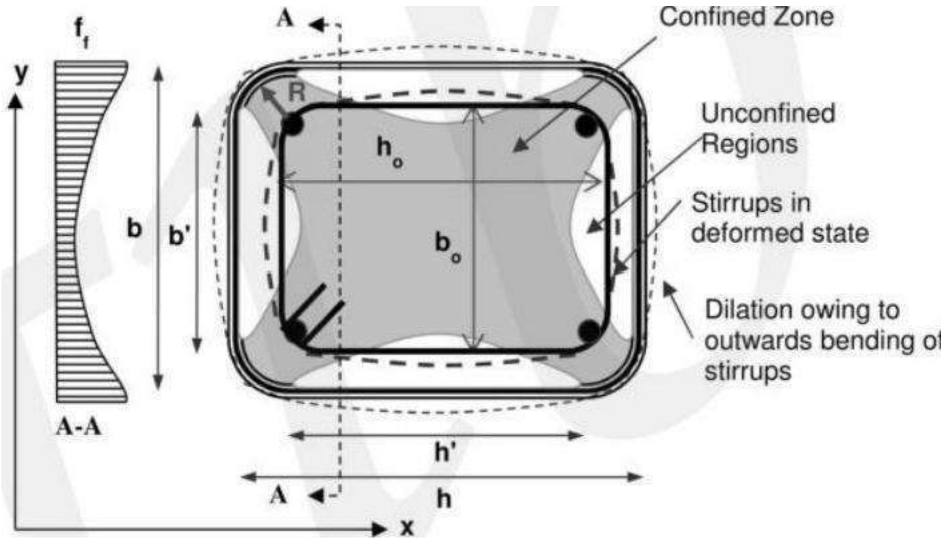


Fig.8-4 Definition of terms of estimation of confining pressure

Parameter eh is the ultimate strain of the jacket in the hoop direction,defined in Section 8.4.2.3.Parameter pswx is the transverse(web)steel reinforcement ratio in the x direction:for links oriented in the x direction placed along the member length at a clear spacing s,having a total sectional area of Aswx,this is defined by:Pswx=Asw×/s·b。. Similarly,Pswy=Aswy/s-h。

A uniform lateral pressure is assumed to confine the FRP-encased concrete in compression. This pressure,denoted by o is the average of o and o defined above.To account for the reduced efficiency of confinement in rectangular cross sections an effectiveness coefficient, αp is used(see Section 6.2.3.3)in order to moderate the FRP component of confining stress This is similar to the effectiveness coefficient,αw,used for stirrup-generated confinement EN 1998-120049:

$$\sigma_{lat} = \frac{1}{2}\left[\alpha_f\left(\rho_{fw,x}+\rho_{fw,y}\right)E_f\cdot\varepsilon_{fu,h}+\alpha_w\left(\rho_{sw,x}+\rho_{sw,y}\right)f_{y,st}\right] \quad (8\text{-}14a)$$
$$=0.5\left(\alpha_f\cdot\rho_{fv}\cdot E_f\cdot\varepsilon_{fu,h}+\alpha_w\cdot\rho_{sv}\cdot f_{y,st}\right)$$

Parameters p₆and p₃are the volumetric ratios of transverse reinforcement(Fig.8-4):

$$\rho_{fv} = \frac{2t_f\left(h+b\right)}{h\cdot b} \qquad \rho_{sv} = \frac{A_{sw,x}\cdot h_o + A_{sw,y}\cdot b_o}{s\cdot b_o\cdot h_o} \quad (8\text{-}14b)$$

8.4.2.2 Confinement effectiveness coefficients αpαw

The confinement effectiveness coefficient is the volume ratio of the encased member that is effectively confined.With reference to Fig.8-4,αwis defined for stirrup confinement according to EN 1998-19 as:

$$\alpha w=\alpha_o\cdot\alpha \qquad \alpha_n = 1-\sum_{i=1}^{n}\frac{b_i^2}{6b_o\cdot h_o} \qquad \alpha_s = \left(1-\frac{s}{2b_o}\right)\left(1-\frac{s}{2h_o}\right) \quad (8\text{–}15)$$

Similarly,the effectiveness of confinement provided by FRP jackets is obtained as the volume ratio of the effectively confined part of the member [see also Eq.(6-58)]:

$$\alpha_f \approx 1-\frac{\left(b-2R\right)^2+\left(h-2R\right)^2}{3b\cdot h} = 1-\frac{\left(b'\right)^2+\left(h'\right)^2}{3b\cdot h} \quad (8\text{–}16)$$

By definition,the effectiveness coefficient cannot exceed 1.In lightly reinforced members that are considered for retrofit with FRP jacketing,the contribution of the stirrups may be neglected with no significant loss of accuracy.The effectiveness coefficient of the FRP jacket, α,decays fast with increasing aspect ratio h/b of the member's dimensions.Further reduction occurs if the FRP jackets are placed in strips and not continuous over the member length (Fig.6-13a)or at an angle with respect to the member's axis not equal to 90°(Fig.6-13b).

-For members with a circular cross section and continuous jacketing(i.e.no strips)α,=1.

-For members with a square cross section and continuous jacketing α≈0.5.

-For cross sections with an aspect ratio>3 the confinement effectiveness is practically negligible(α,≈0),unless transverse anchors are used.However,FRP jacketing in these cases is a very effective means of providing web reinforcement(e.g.in structural walls).

8.4.2.3 Ultimate strain of the FRP jacket

The usable FRP strain ε(design value)is limited in order to protect the retrofit from premature local failures such as:

-(a1)Rupture of the FRP at the corners.This mode of failure occurs mostly due to lateral dilation of concrete under high compression strains in the compression zone of confined members.To delay the occurrence of local rupture due to high compressive pressures the corners ofthe cross section should be chamfered by a radius of R,which should be as large as practically possible.

-(a2)Rupture may also occur due tobuckling of embedded compression reinforcement. The axial compression strain that is allowed to occur in the compression zone of the member at the ultimate limit state shall be limited according to Section 8.6.1.

-(b1) Debonding failure of the FRP in a closed jacket arrangement: The most critical for debonding is the external layer, since the shear strength of the adhesive in interior layers is enhanced by friction due to confinement. The minimum required overlap length of the exterior jacket layer, l_b^{min}, is:

$$l_b^{min} = (\pi/2)\sqrt{E_f \cdot t_o \cdot s_{ao}/\tau_a} \tag{8-17}$$

where t is the shear strength of the adhesive at the stage of plastification and s the slip of the adhesive at shear failure (this data is provided by the adhesive manufacturer).

-(b2) Debonding failure of the FRP in an open (U-type) FRP jacket arrangement (i.e. in case of anchorage on brittle substrate such as the concrete cover): The minimum development length measured from the critical section where εeu will be developed - at the point where the FRP intersects a flexural or shear crack of width w., is:

$$l_b^{min} = (\pi/2)\sqrt{E_f \cdot t_f \cdot w_{cr}/\tau_{b1d}} \tag{8-18}$$

where τ_{b1d} is the design bond strength, equal to fak/Y. Design calculations may be performed for w 。 =0.5 mm. The jacket effective thickness t, is defined in Section 8.4.2.1.

For proportioning of the FRP jacket, the axial tensile strain e shall not exceed the limit:

$$\varepsilon_{fu,h} \le \eta_1 \cdot \eta_2 \cdot \eta_3 \frac{\varepsilon_{fu}}{\gamma_f} \tag{8-19}$$

where y is taken from Table 8-1 depending on the jacketing arrangement: fully wrapped retrofit arrangement refers to closed jackets that fully encase the member; anchorage on brittle substrate refers to open jackets (U-shaped) that do not enclose the member on all sides.

Factor η_1 accounts for the radius of chamfer R at the corners of the member (also known as strain efficiency factor[14,15,16], see Fig. 8-5):

$$\eta_1 = 0.25 + \frac{2(2R + D_b)}{h'} \le 1.0 \tag{8-20}$$

Parameter D_b is the embedded corner bar diameter. Equation (8-20) is valid only for rectangular cross sections (h' is the straight part of the largest cross section side) whereas for circular members η_1=1.

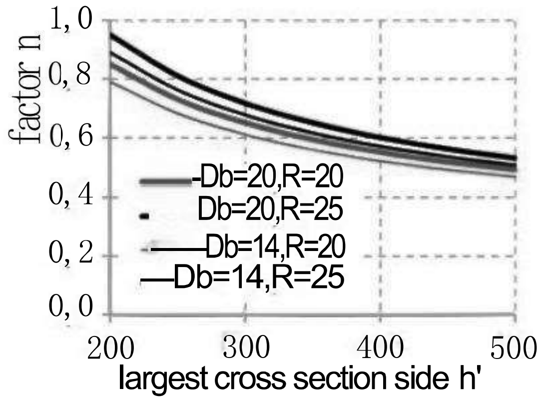


Fig. 8-5 Definition of factor η_1 versus the largest member cross section side for several values of R and D_b (in mm).

Factor η_2 accounts for the overlap length of the exterior wrap layer:

$$\eta_2 = \frac{l_b^{avail}}{l_b^{min}} \le 1.0 \qquad (8\text{-}21)$$

where l_b^{avail} is the available overlap length and l_b^{min} is the minimum required overlap length to ensure rupture of the jacket.

Factor η_3 accounts for the redundancy of the jacket against debonding failure.

-For fully wrapped jackets η_3=1.0

-For U-type arrangements with special details in the ends in order to secure the jacket against debonding(e.g.adhesive anchors,NSM details,etc.)η_3=1.0

For U-type arrangements without special measures against debonding η_3=0.85

-For straight layers with special details in the ends in order to secure the jacket against debonding(e.g.adhesive anchors,NSM details,transverse confining wraps)η_3=0.9

-For straight layers without special measures against debonding η_3=0.6

8.4.2.4 Stress-strain law for FRP-confined concrete

The confined concrete strength fand the corresponding strain at attainment of peak stress,ε,in the compression zone of the encased cross section may be calculated from the classical confinement model of Richart et al.[7] moderated to account for the greater compliance of jackets as compared to conventional stirrups:

$$f_{cc} = f_c + 3\sigma_{lat} \qquad (8\text{-}22a)$$

$$\varepsilon_{cc} = \varepsilon_{c2}\left[1 + 5\left(\frac{f_{cc}}{f_c} - 1\right)\right] \qquad (8\text{-}22b)$$

By substitution of Eq.(8-14a)in Eq.(8-22),and assuming ε_{c2}=0.002(strain at peak stress of unconfined concrete),the following are obtained:

$$f_{cc} = f_c + 3\sigma_{lat} = f_c + 1.5\left(\alpha_f \cdot \rho_{fv} \cdot E_f \cdot \varepsilon_{fu,h} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st}\right) \qquad (8\text{-}23a)$$

$$\varepsilon_{cc} = \varepsilon_{c2}\left(1 + 15\frac{\sigma_{lat}}{f_c}\right) = 0.002 + 0.015\frac{\alpha_f \cdot \rho_{fv} \cdot E_f \cdot \varepsilon_{fu,h} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st}}{f_c} \qquad (8\text{-}23b)$$

The failure strain of confined concrete,$\varepsilon_{cu,c}$,corresponding to a compression strength reduction in excess of 15%is obtained from[1]2,18(Fig.8-6b):

$$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.075\left(\zeta\frac{\alpha_f \cdot \rho_{fv} \cdot E_f \cdot \varepsilon_{fu,h} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st}}{f_c} - 0.1\right) \ge \varepsilon_{cu} \qquad (8\text{-}24)$$

where:for $\varepsilon_{cu,c} \le 0.01$ ζ=1,for $\varepsilon_{cu,c} \ge 0.02$ ζ=0.6 and for intermediate axial strain values ζ varies linearly between the two bounds.This parameter accounts for the reduced jacket effectiveness when a very high confinement is present:at such a very high limit axial compaction of confined concrete accounts for part of the observed axial strain capacity, without engaging the jacket through dilation of the core.Note that material safety factors are not used for concrete characteristic strength in determining the compression stress-strain law.Such a safety factor may affect unfavourably the estimated hierarchy of failure in establishing capacity design principles.It is recommended that a safety factor is applied on the calculated member strength after retrofit to account for uncertainties.

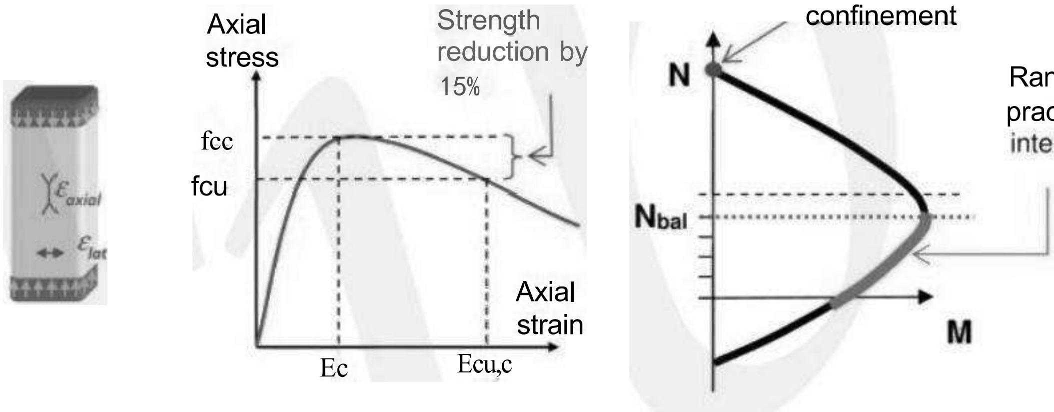
The following stress-strain model,based on correlation of a large database of tests19, is also relevant and might be considered in a final design guideline from among the many confinement models available in the literature(note that the model does not consider the steel stirrups):

$$f_{cc} = f_c\left[1+3.5\left(\alpha_f \cdot \rho_f \cdot f_f / f_c\right)^{0.75}\right] \tag{8-25a}$$

$$\varepsilon_{cc} = 0.002\left[1+5\left(f_{cc}/f_c-1\right)\right] \tag{8-25b}$$

$$\varepsilon_{cu,c} = 0.0035+\left(10/h\right)^2+0.4\alpha_f \cdot \alpha_{eff,j}\min\left(0.5;\rho_f \cdot f_f/f_{cc}\right) \tag{8-25c}$$

where p is the geometric ratio of FRP in the direction of loading;f=E(kE);e,uis the failure strain of the FRP;k=0.6 for CFRP,GFRP or AFRP;h=depth of cross section in the direction of loading(in mm);and aef;=0.5[1-min(0.5;p;f./f)]for CFRP and GFRP, with 0.5 replaced by 0.3 for AFRP.



(a) (b) (c)

Fig.8-6(a)Typical test of FRP-confined reinforced concrete column in compression.(b)Nomenclature for the stress-strain milestone points.(c)Axial load-Moment interaction diagram of typical prismatic element

8.5 Acceptance criteria and safety evaluation

8.5.1 Rotation capacity and displacement ductility of FRP-confined members

Based on ample experimental documentation,RC beams,columns and walls retrofitted with FRP jackets in the critical regions can develop significant rotation capacity and displacement ductility.Rotation capacity refers to the maximum angle that may be sustained between the chord of the member in the displaced position and the normal to the end cross sections(Fig.8-7).

$$\mu_\Delta = 1.3 + 12.4\left(\frac{\sigma_{lat}}{f_c} - 1\right) = \tag{8-46a}$$

$$1.3 + 12.4\left[\frac{0.5\left(\alpha_f \cdot \rho_{fv} \cdot E_f \cdot \varepsilon_{fu,h} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st}\right)}{f_c} - 0.1\right] \geq 1.3$$

The above is simplified by neglecting the contribution of stirrups (if their arrangement is deemed non-conforming to modern standards):

$$\mu_\Delta = 1.3 + 12.4\left[\frac{0.5\left(\alpha_f \cdot \rho_{fv} \cdot E_f \cdot \varepsilon_{fu,h}\right)}{f_c} - 0.1\right] \geq 1.3 \tag{8-46b}$$

where εuhis the ultimate strain of the jacket in the hoop direction (design value), given by Eq.(8-19). Note that predictions given by Eq.(8-46) may be quite conservative.

By recalling here the expressions for the chord rotation at yielding [Eq.(8-27)] and for the plastic component of drift capacity [Eq.(8-32)] by a study[19] the displacement ductility related to the confinement provided by the FRP jacketing is defined by Eq.(8-47); if the displacement ductility demand is known, this equation may be used to extract the required jacket thickness (implicit in the exponent of the numeral factor 25) through iteration:

$$\mu_\Delta = \mu_\theta = \frac{\theta_u}{\theta_y} = \frac{\theta_y + \theta_u^{pl}}{\theta_y} = \tag{8-47}$$

$$1 + \frac{0.0185 \cdot 0.48\left(1 + \frac{\alpha_{sl}}{1.6}\right)(0.25)^{v}\left[\frac{\max(0.01, \omega')}{\max(0.01, \omega)}\right]^{0.3} f_c^{0.2}\left(\frac{L_s}{h}\right)^{0.35} 25^{\left[\frac{\alpha_w \cdot \rho_w \cdot f_{yw}}{f_c} + \left(\frac{\alpha \cdot \rho \cdot f_u}{f_c}\right)_{j,a0}\right]} 1.275^{100\rho_d}}{\frac{1}{3}\phi_y\left(a_v \cdot z + L_s\right) + 0.0014\left(1 + 1.5\frac{h}{L_s}\right) + 0.125\phi_y\frac{D_b \cdot f_{sy}}{\sqrt{f_c}}}$$

In Eq.(8-47), f and fyware the concrete compressive strength (MPa) and the stirrup yield strength (MPa), respectively, directly obtained as mean values from in-situ tests, and from the additional sources of information, appropriately divided by the confidence factors, accounting for the level of knowledge attained.

8.7 Joints

Beam-column joints are regions of very high shear stress demand. The design shear force acting on the beam-column joint during seismic excitation may be estimated from the moment reversal which occurs between the end faces of the joint region, as the slope of the moment diagram over the depth of the beam or column (Fig.8-11a). Joint failure occurs due to inadequate shear reinforcement or by crushing failure of the diagonal compressive strut that forms in the body of the joint (Fig.8-11b). Requirements for retrofit draw from past knowledge about the behaviour and design considerations of conventionally reinforced beam-column joints due to their importance in securing the integrity of the structure: the joint panel lies in the path of the vertical loads (overbearing weight of the structure) and as such, considerations of resilience and integrity of the retrofit necessarily lead to overdesign consistent with capacity-design principles.

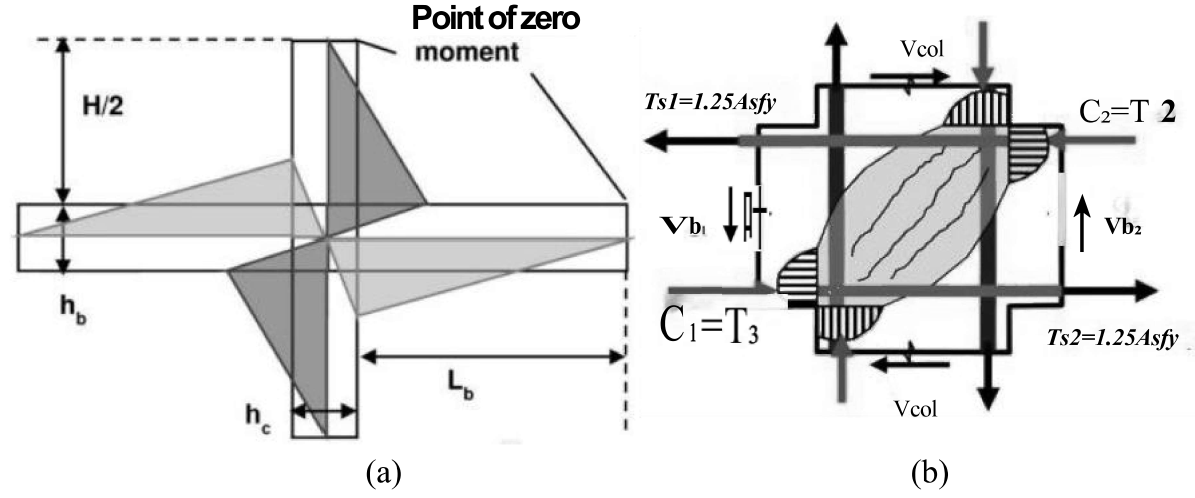


Fig.8-11(a)Calculation of joint shear force V,from the gradient of flexural moments along the column or the beam line in the joint region.(b)Diagonal strut and definition of confinement requirements

As depicted in Fig.8-11a the joint panel is subjected to steep moment gradients as they facilitate reversal of moment from one face of the member to the other.In the ultimate limit state the design force in the joint is so significant that joint strength is thought to be supported primarily by the diagonal compressive strut that forms in the joint(Fig.8-11b) provided that it is confined[29,30].Current codes require that the stirrup arrangement used in the end critical zones of the columns is also extended inside the joint panel in order to secure confinement (EN1998-18);however,the effectiveness of confinement also depends on the number of free faces of the joint (that is,how many sides are unrestrained).It is notable that in reconnaissance reports joint failures are usually reported to occur in the perimeter of the building.In recognition of this fact theACl-ASCE 352 Recommendations[3] limit the allowable shear stress input in an exterior joint to 66%of the value allowed in interior joints;the corresponding limit is at 80%in EN 1998-1[9].

In old construction,joints are generally unconfined or poorly detailed.This renders them susceptible to diagonal tension failure at relatively low levels of shear demand.Past experiments conducted in controlled laboratory conditions as well as analytical studies have demonstrated that RC joints in beam-column connections can be effectively strengthened with a suitable arrangement of externally bonded FRP[32,33,34,35,36,37,38];analytical studies have also been developed to illustrate the mechanics of this strengthening scheme[39,40].These studies support the development of rehabilitation procedures and detailing methods for strengthening of beam-column joints with FRP jacketing.However,a few of the specimens tested are planar assemblies without slab and/or transverse beams.In total,the number of available exterior connection tests that faithfully reproduce the actual three-dimensional features of RC frame joints including the monolithic slab is still considered limited in light of the key role of joints in the overall structural integrity and survival in the event of a serious earthquake,particularly when considering the range of test variables that would need to be documented in a round robin experimental campaign.

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

The emphasis on resilient retrofit designs,in light of the weakness in the method necessarily imparted by the decisive dependency on the engineer's judgement as to the proper arrangement of the FRP jacket so as to effect the desirable confinement,has led to the development of two alternative options in designing FRP-based retrofits of beam-column connections.One neglects this confining contribution in the interest of conservatism and on the assumption that unless designed by specialists,this type of retrofit may prove inferior to expectations as to its effectiveness in providing confinement.This option,termed below as Approach 1,determines the required amount of FRP reinforcement through its function as added shear reinforcement in the joint panel.This generally leads to significant amounts of added reinforcement that would need to be implemented in the form of strips(EBR or NSM).The second alternative considers the benefits of confinement provided by a suitable arrangement of jacket strips in the perimeter and the boundaries of the joint,and accounts for a concrete contribution term to the shear strength of the joint in recognition of the integrity of the encased concrete.This option,termed Approach 2,generally leads to lesser amounts of required FRP reinforcement.

Whether used as shear reinforcement or as a form of confinement,FRP sheets represent an appropriate method of beam-column joint retrofit.In this case,externally bonded FRP reinforcement either confining the joint on all free faces or placed as strips with the fibres running in the direction of principal tensile stresses is needed.To determine the required amount,the jacket thickness t,may be estimated from the two different approaches detailed below.

<u>Approach 1:</u>Consistent with requirement (4)of Section 5.5.3.3 of EN 1998-1[9]on beam-column joints,the integrity of the joints after diagonal cracking may be ensured by reinforcement crossing the diagonal crack paths and designed to support the full amount of the applied joint shear force.Thus,the required jacket thickness is estimated neglecting the contribution of the diagonal strut that forms in the joint on account of the uncertain restraining action of the jacket when placed in the complex 3-D geometry of the connection.In deriving the equations that follow,the FRP fibres are taken oriented in the horizontal and/or vertical direction [in the case of inclined fibres at an angle βwith respect to the beam axis,the result for the required thickness obtained from Eq.(8-48)is further divided by(1+cosβ)sinβ].The required amounts are obtained from the following expression:

-If the FRP fibres are oriented in the horizontal direction,then,t,=t:

$$t_{f,h} = \frac{\gamma_{Rd} \cdot V_{j,h}}{h_b \cdot E_f \cdot \varepsilon_{fd}} \qquad (8\text{-}48a)$$

-If the FRP fibres are oriented in the vertical direction,then,t=t,:

$$t_{f,v} = \frac{\gamma_{Rd} \cdot V_{j,v}}{h_c \cdot E_f \cdot \varepsilon_{fd}} \qquad (8\text{-}48b)$$

with γRequal to 1.5.

In Eq.(8-48)V.and V are the design shear forces in the joint,assumed to act on a horizontal and a vertical plane through the joint,respectively.Parameter ε „is the allowable design value of FRP tensile strain that,for the case considered,shall not be taken higher than 0.4%.An essential requirement is proper anchorage of the FRP strips(e.g.as shown in Section 9.2.3,Fig.9-20).When FRP reinforcement is not properly anchored,FRP strengthening shall not be considered effective.When more than two FRP jacket layers are needed,then the reinforcement shall be placed in the form of NSM strips and shall be encased transversely by properly anchored jacket layers.

For calculating the design values of V.hand V two alternative options are possible; one is based on EN 1998-1[9],whereas the other is based on the assessment procedures by the Greek Code for Assessment and Retrofit13;note that the necessary nomenclature is defined with reference to Fig.8-11b.

EN 1998-1[9](Section 5.5.2.3):

(a)For interior beam-column joints: $V_{,16}=1.25(A_1+A_2),-Vc$

(b)For exterior beam-column joints: V;=1.25A,fy-Vc (8-49)

GREEK CODE FOR ASSESSMENT[13]:

First the sums of yield moments in the beams and in the columns framing into the joint in consideration are calculated.Here,∑M is the sum of yield moments of the beams that frame into the joint and 2M is the sum of yield moments of the columns that frame into the joint.

If 2M<2M,then the horizontal shear force V.his derived from the slope of the column moment diagram as follows:

$$V_{j,h} \approx \Sigma M_{yb}\left(\frac{1}{jd_b}-\frac{1}{H_n}\frac{L_{b,n}}{L_b}\right) \tag{8-50}$$

while the vertical shear force acting in the joint,V,,is obtained from:

$$V_{j,v}=V_{j,h}\frac{h_b}{h_c} \tag{8-51}$$

If ∑M<2My,then the vertical shear force V;is derived by

$$V_{j,v} \approx \Sigma M_{yc}\left(\frac{1}{jd_c}-\frac{1}{L_{b,n}}\frac{H_n}{H}\right)+\frac{1}{2}\left|\left(V_{g+\psi q,b}\right)_l-\left(V_{g+\psi q,b}\right)_r\right| \tag{8-52}$$

while the horizontal shear force V,is obtained from:

$$V_{j,h}=V_{j,v}\frac{h_c}{h_b} \tag{8-53}$$

In the above equations,$jd_b$is the internal lever arm of the beam section and jd is the internal lever arm of columns;(V8+9.,),and $(V_{g+\psi q,b})_r$are the shear forces of the beams to the left(I)and to the right(r)of the joint due to vertical loads that act at the same time with the seismic action.L and L,are the theoretical and clear half span of the beams; H and H are the theoretical and clear storey heights.

Upper limit on beam-column joint demand: The requirement by EN1998-1[9] is enforced, limiting the diagonal compression induced in the joint by the diagonal strut mechanism in the presence of transverse tensile strains. For interior beam-column joints:

$$V_{j,h} \le \eta \cdot f_{cd} \sqrt{1-\frac{v_d}{\eta}} b_j \cdot h_{jc} ;\eta=0.6(1-F./250) \tag{8-54}$$

h is the distance between extreme layers of column reinforcement, b is the effective joint width and v is the normalised axial load ratio exactly above the joint. Coefficient η accounts for the reduction in strength of the diagonal compression strut forming in the joint, due to diagonal tension cracking.

For exterior beam-column joints: V; should be less than 80% of the above limit value.

<u>Approach 2:</u> This enables determination of the required jacket thickness with fibres oriented in multiple directions (multi-axial fabrics with fibres at 0°, 90°, ±45°)38. This approach is based on the use of the principal tensile stress derived combining the joint shear stress $v_{j,h}$=V./b:h and the axial stress $f_a = N/b_c \cdot h_c$. The horizontal shear force acting in the joint, $V_{j,h}$ is derived from Eq.(8-50) or Eq.(8-53) and N is the axial load acting on the top column.

The principal tensile stress to be used for determining the required FRP amount (FRP area, A_f) is computed from:

$$p_{t,f} = -\frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{j,h}^2} - k\sqrt{f_{cm}} \tag{8-55}$$

where: k is a numerical coefficient representing the original joint shear capacity and it is equal to 0.30 for beam-column joints with deformed bars and 0.20 for beam-column joints with smooth bars; f_{cm} is the mean compressive strength of concrete.

In order to calculate the unknown t, two parameters should be calculated: the required FRP area, A, and the design FRP strain, ed

The FRP area is defined as follows:

-Uniaxial fabric-fibres oriented in the horizontal (β=0°) or vertical (β=90°) direction

$$A_f = n \cdot t_f \cdot h_b \cdot \sin\theta \quad \text{for} \quad \beta=0°$$
$$A_f = n \cdot t_f \cdot h_c \cdot \cos\theta \quad \text{for} \quad \beta=90° \tag{8-56a}$$

-Bidirectional fabric-fibres oriented in the horizontal and vertical direction (β=0° and β=90°)

$$A_f = n_f \cdot t_f \cdot h_c \cos\theta(1+\tan^2\theta) \tag{8-56b}$$

-Quadriaxial fabric-fibres oriented in the horizontal, vertical and ±45° direction (β=0°, β=90° and β=±45°)

$$A_f = n_f t_f \cdot h_c \cdot \cos\theta(1+\tan\theta+2\tan^2\theta) \tag{8-56c}$$

where β is the inclination of fibres with respect to the beam axis, n is the number of joint panel sides strengthened in shear with FRP (1 or 2 sides, Fig. 8-12), and θ is the inclination of concrete compressive strut with respect to the beam axis, θ=arctan(h_b/h).

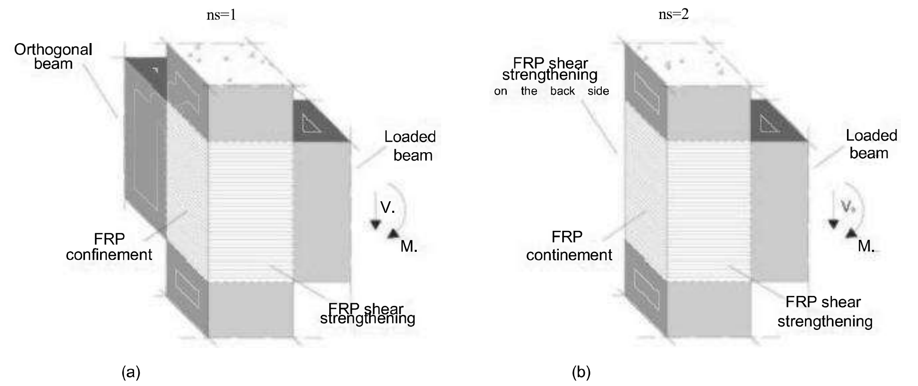


Fig.8-12 Joint panel sides strengthened in shear with FRP:(a)one side(ng=1)and(b)two sides(n,=2)

The design FRP strain ed is defined according to Eq.(8-57)and cannot exceed the ultimate FRP strain ε_{fu}:

$$\varepsilon_{fd} = 34\left(\frac{f_{cm}^{2/3}}{A_f \cdot E_f}\right)^{0.6} \tag{8–57}$$

When the FRP strengthening is applied on a repaired substrate,0.8εshould be used.

Based on the demand given in Eq.(8-55),the total FRP thickness t(thickness of n plies of FRP reinforcement)may be estimated from Eq.(8-58):

$$p_{t,f} = \frac{A_f \cdot E_f \cdot \varepsilon_{fd}}{b_c\left(h_c / \sin\theta\right)} \tag{8–58}$$

Special details at the ends of the FRP strengthening need to be provided in order to secure the jacket against debonding (e.g.adhesive anchors,NSM details,transverse confining wraps).When the FRP reinforcement is not properly anchored,FRP strengthening shall not be considered effective.

In the case of discontinuous FRP reinforcement(FRP strips),A,may be estimated as follows:

$$A_f = \sum_{i=1}^{n} A_{f,i} \cdot \sin\left(\theta + \beta_i\right) \tag{8–59}$$

where A=n;t;b and b is the width of the FRP sheet derived as a function of the fibres' inclination as follows:

$$b_f = \frac{\left(w_f \cdot n_{str}\right)^2 \cos\beta}{h_b} \quad \text{for } \beta < \theta \tag{8-60a}$$

$$b_f = \frac{\left(w_f \cdot n_{str}\right)^2 \sin\beta}{h_c} \quad \text{for } \beta \geq \theta \tag{8-60b}$$

where w,is the strip width and n,is the number of strips in the joint panel(Fig.8-13)

In the design procedure the joint principal compressive stress,pe,cannot exceed the ultimate compressive strength of the joint:

$$p_c = \frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{j,h}^2} \leq 0.5 f_{cm} \tag{8–61}$$

9.Detailing

9.1 General

The quantity of bonded FRP reinforcement is calculated according to the models described in the previous chapters.Detailing rules give practical information on the location,arrangement and limitations for the FRP reinforcement required by considerations such as minimum ductility,functional requirements,adequate anchorage,applicability of calculation models,practical durability measures,environmental conditions,etc.

Compared to other aspects of the FRP strengthening technique,requirements for detailing are much less supported by available test results.Nevertheless,detailing rules are important and if not applied may lead to premature failure of the strengthened structure.The detailing rules in this chapter are recommended,and should be considered compulsory when directly associated with the design provisions of Chapters 6-8.

9.2 Detailing for strengthening with EBR FRP

9.2.1 Beams

9.2.1.1 Flexural strengthening

Flexural strengthening is provided by axially oriented strips or cured in-situ sheets/fabrics bonded to the top or bottom faces ofthe member or even to the sides.In the anchorage zones(including those of strips or sheets over supports of continuous beams or slabs) no additional transverse reinforcement is required if adequate anchorage is provided by verifications developed according to provisions of Chapter 6.Alternatively,if the anchorage force is not evaluated according to the provisions of Chapter 6,the following rules (shown in Fig.9-1)could be used:

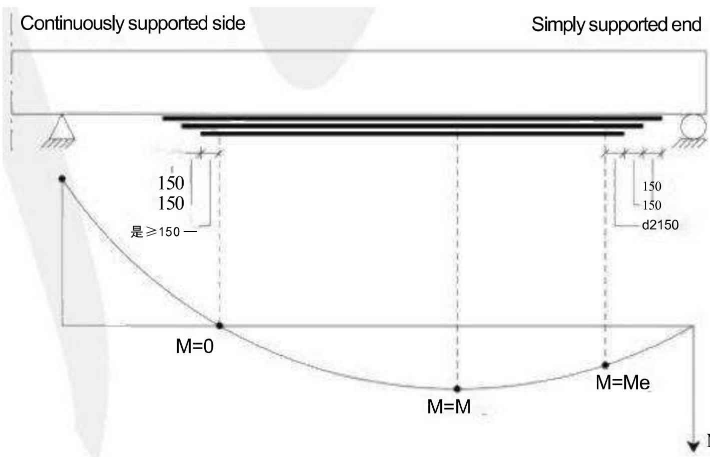


Fig.9-1 Graphical representation for allowable termination points of a three-ply FRP laminate(based on ACI 440.2R-08).Dimensions in mm;d is the effective depth of the flexural reinforcement

This chapter was mainly authored by Ceroni.

-For simply supported beams, the plies should extend a distance d (effective depth of the beam) past the point along the span corresponding to the cracking moment M under ultimate loads.

-For continuous beams, a single-ply FRP laminate should be terminated d/2 or 150 mm minimum beyond the inflection point (point of zero moment resulting from factored loads).

If several layers are to be applied, it is recommended to apply the one next to the other rather than the one onto the other. In this latter case, more than 3 layers of pultruded strips or 5 layers of cured in-situ fabrics are not recommended to apply unless proved by experimental evidence. In the case of multiple-ply laminates, the termination points of the plies should be tapered; suggested dimensions are depicted in Fig. 9-1[1]. By applying several layers of prestressed strengthening strips, reduction of prestressing due to the successive release of prestressing forces should be considered[2].

The following general recommendations apply for longitudinal strips[3]:

-Maximum spacing (centre distance) amax between strips (to avoid too large unreinforced sections):

- amax≤0.2E (E=span length; for cantileversl=28 wheret.=length of cantilever)
- amax≤4h (h=total depth)

-Permissible radii of bends should be given in the product description for the strips. Permissible radii of fabrics need not be specified. However, it is recommended that sharp edges of the section are mechanically rounded before application. In this case, a minimum radius of 30 mm is recommended.

-Crossing of strips is allowed, with bonding in the crossing area and proper levelling following unevenness limitation rules.

-Lap joints should be avoided; they are not necessary, because FRP can be delivered into the required length. Nevertheless, if needed, lap joints should be made in the direction of the fibres with an overlap that will ensure tensile fracture of the FRP prior to debonding at the lap joint.

For flexural strengthening the most common anchorage system is given by U-shaped straps (Fig. 9-2) or by fibre sheet or laminate glued transversally to the strengthening direction (Fig. 9-3). Generally, the first option is preferred. The second option may be viable when the FRP-to-concrete width ratio (b/b, Fig. 9-3) is considerably less than 1, because the transversal sheet/laminate can extend to the width of concrete covered by the FRP strengthening[4]. This enlargement of the bonded width could be properly taken into account in the theoretical formulations suggested in Section 5.3.2.

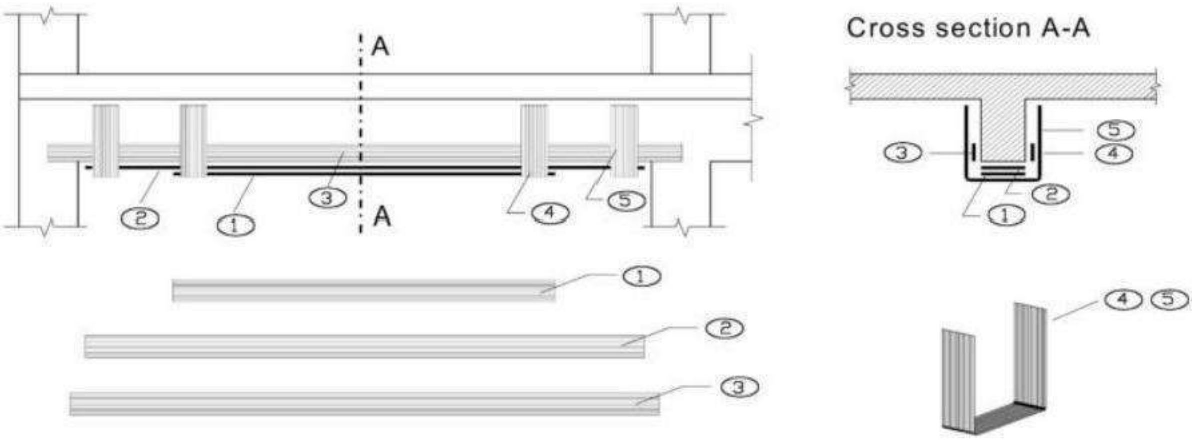


Fig.9-2 U-shaped FRP straps

When the b,/b ratio approaches to 1,a simple transversal strip(Fig.9-3)becomes unhelpful and U-shaped fibre straps(Fig.9-2)result more efficient in increasing strength and ductility.These external FRP straps enclose the longitudinal strips at their ends and are not considered to be the part of the shear reinforcement but keep the longitudinal strips in their position and to prevent peeling-off.It is not necessary to anchor these straps in the compression zone.

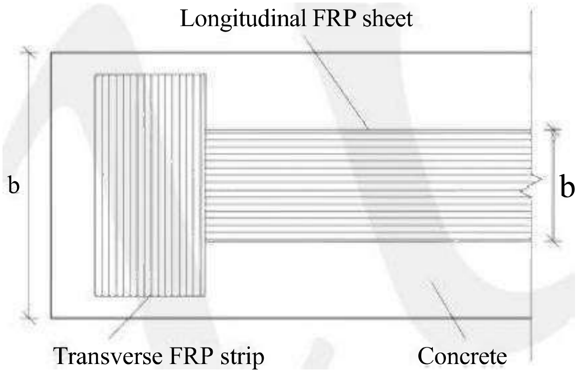


Fig.9-3 Transverse FRP strip as end anchoring device

Instead of U-shaped straps,alternative solutions such as bolts (possibly combined with end tabs)(Fig.9-4a)or FRP rods[5](Fig.9-4b)may be applied.Unless the FRP is of a special type,made of multidirectional fibres[6],the use of bolts through its thickness is not permitted,as it may lead to premature interlaminar shear,splitting or local crushing of the FRP.In any case,bolts should be supplied with large washers and tightened up to a specified torque to prevent crushing.End anchorage through bearing(Fig.9-4c)is an effective solution,but generally not practical.

Another option for end anchorage involves the use of U-shaped or L-shaped steel devices7,8,especially in the case of strengthening slabs,where transverse wrapping is not possible (e.g.Fig.9-5).Such devices require a minimum cover in the order of 20 mm.

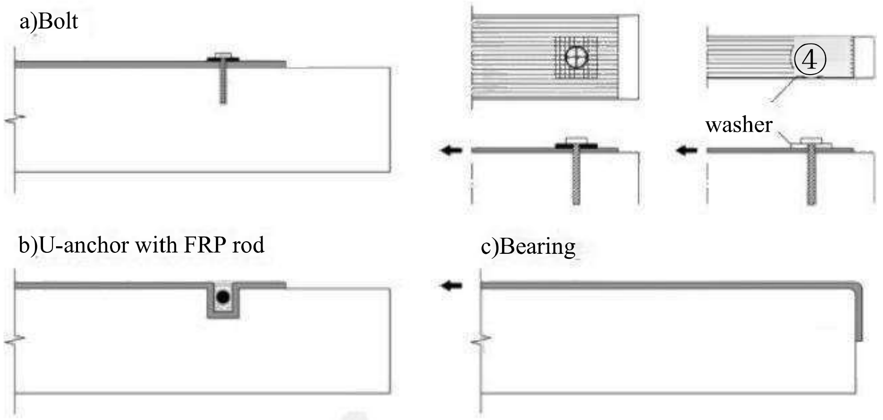


Fig.9-4 Alternative end anchorage solutions for flexural reinforcement:(a)bolts,(b)rods inside grooves,(c)anchorage through bearing

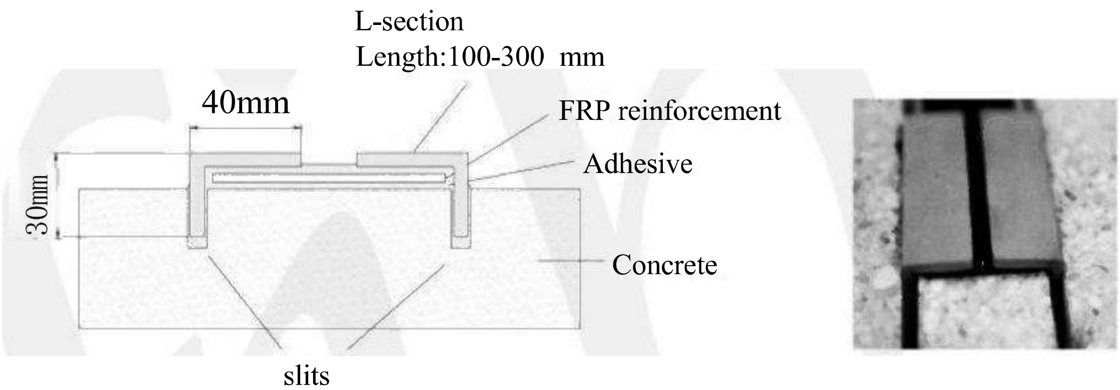


Fig.9-5 Anchorage system for CFRP strips8

Fibre fan anchor systems (see Fig.9-6 and 9-7),also referred to as spike anchors,are generally effective in bonding sheets or fabrics onto concrete substrates.A hole is drilled in the concrete either perpendicular(Fig.9-6)or-even better-parallel(Fig.9-7)to the FRP.A fibre tow(Fig.9-7a)is forced into a pre-drilled hole(Fig.9-7b)and the ends are splayed outwards on longitudinal fibres and bonded with epoxy resin (Fig. 9-7c).

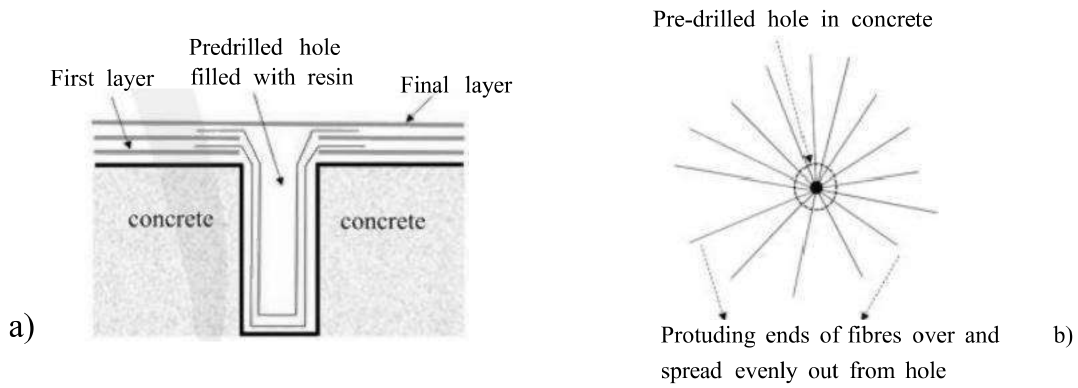
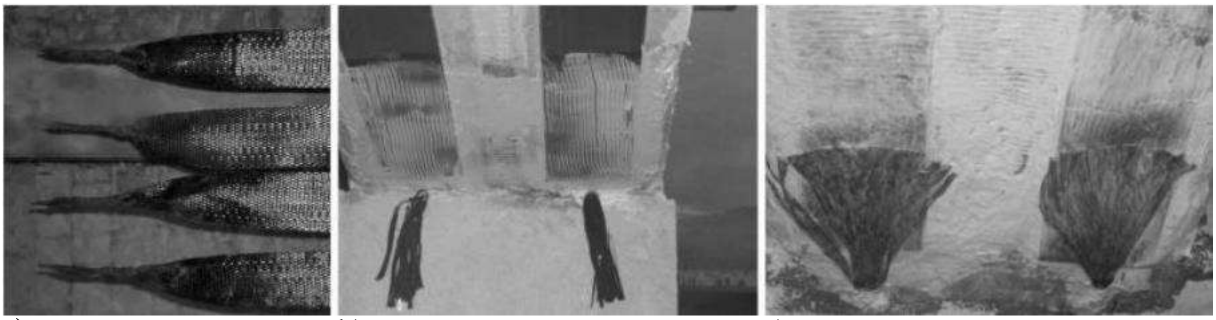


Fig.9-6 Fibre fan anchor system(spike anchor):(a)cross section,(b)top view



a) b) c)

Fig.9-7(a)Carbon fibre fan anchor;(b)execution of holes;(c)splaying of fibres outwards the hole

Experimental studies[9] on the effective embedment depth of fan-shaped anchors evidenced that the capacity of 14-20 mm diameter anchors does not increase for depth longer than 100 mm in 10-20 MPa concrete elements. Tests with values of anchor depth greater than 100 mm[10] evidenced a good efficiency of this system (larger than 60% of the tensile capacity of fibres) depending on some design parameters (size, number and spacing of anchors, offset height and angle, type of FRP material). Tensile tests on 6-12 mm diameter carbon fibre anchors[1] showed high activation of the anchors (60-75% of their tensile capacity) used to connect CFRP to concrete.

The design of the fan anchors can be done so that their total weight w corresponding to a length l is higher than the weight w, of fibres in an equal length sheet or fabric, as expressed by Eq.(9-1). The cross sectional area A (and from this the anchor nominal diameter), is then calculated from the weight using Eq.(9-2).

$$w_a = Y \cdot w \tag{9-1}$$

$$A_a = \frac{w_a}{\rho l_a} \tag{9-2}$$

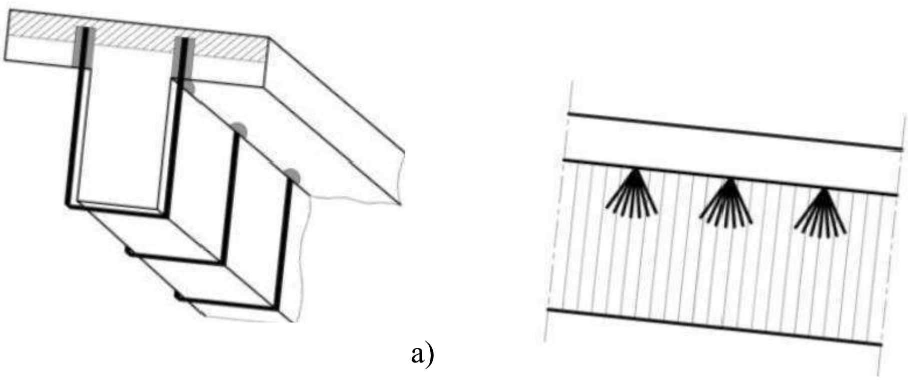
where γ is an effectiveness coefficient, in the order of 1.5[12] and p is the anchor fibre material density. Indications on the length / and effectiveness coefficient are as follows:

- Anchors have to be inserted at least 50 mm into the core of the concrete and should have a total depth at least equal to 120 mm.
- The cross-sectional area of an anchor has to be at least 50% (y=1.5) greater than the cross-sectional area of the longitudinal sheet; splitting the anchor into smaller anchors at a spacing of about 40-50 mm is recommended.

The effect of fan shape on the effectiveness of anchors is related to the fan angle[13], which should not exceed 30°, in order to minimise stress concentrations14. In general for end anchors of flexural strengthening, the effectiveness of fan shaped anchor systems is at least as high as that of U-shaped straps1°, however the technique is more labour intensive.

9. 2. 1. 2 Shear strengthening

As already discussed in Chapter 6, shear strengthening is achieved through the use of closed or open jackets (Fig.6-18). Open jacket systems, which are typically U-shaped (Fig.6-18b), are less effective than closed systems; but their effectiveness increases by anchoring the fibres through the slabs, as shown for instance in Fig.9-8.

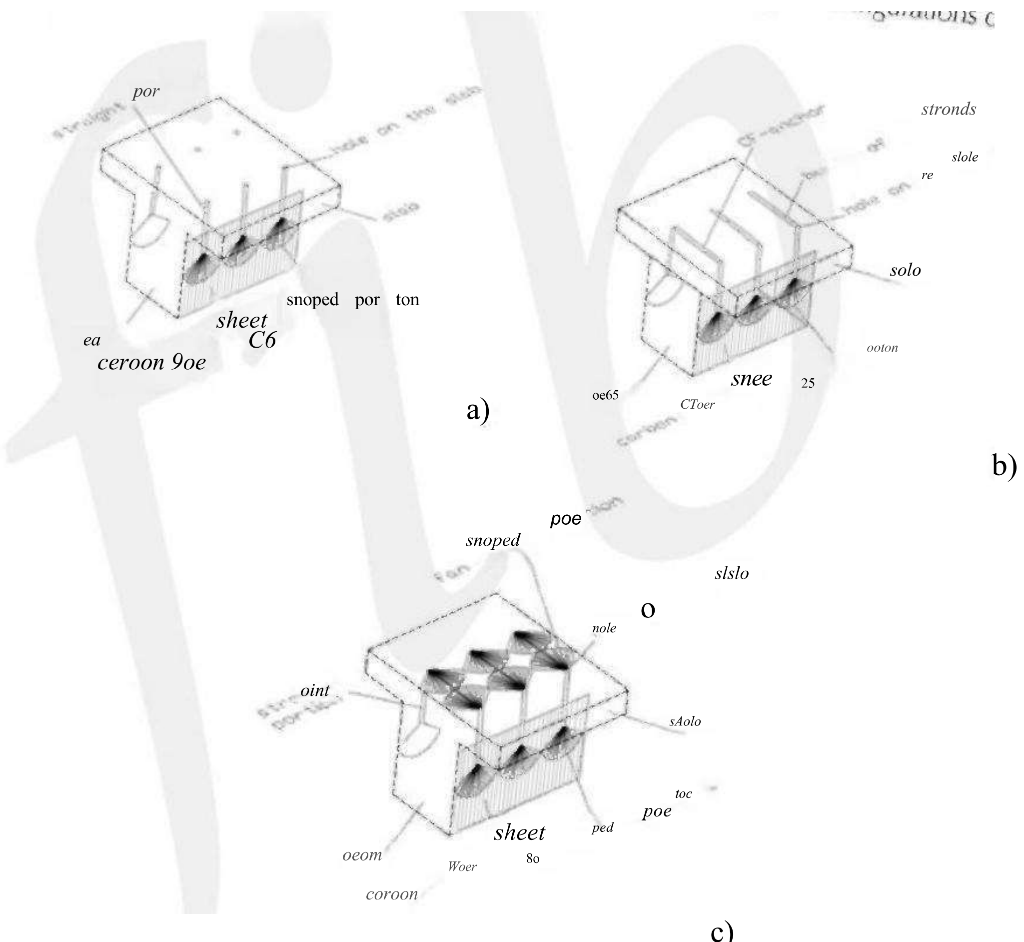


a)

Fig.9-8 Anchorage of U-shaped FRPin the compression zone ofT-beams using:(a)epoxy resin,(b)spike anc)hors

b

acspedfbres spike anchors may be applied in a varietyofcf *no*



a)

b)

c)

Fig.9-9 Fan shapedfibre anchorsfor shear strengthening15,16

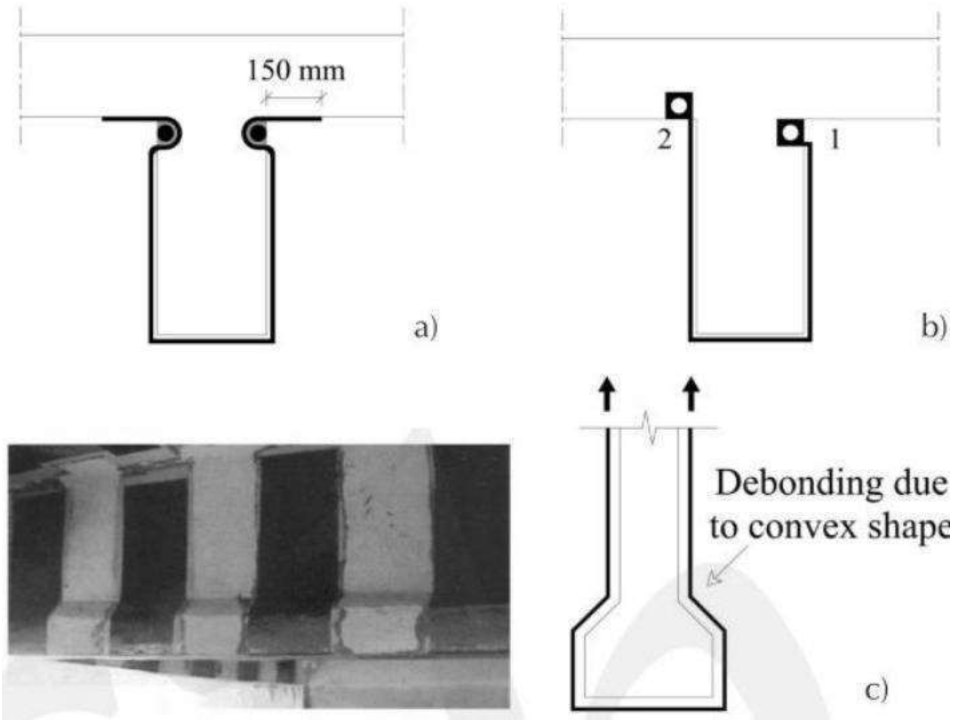


Fig.9-10(a)NSM rods as anchors for U-shaped FRP shear strengthening of an RC beam (b)Possible configurations,with 1 being better than 2(c)Extra anchorage needed for I-shape

Near surface mounted rods inside epoxy-filled grooves may also be applied,as illustrated in Fig.9-10a.The rods inside the grooves could be non-metallic(e.g.FRP);if shear strengthening is provided with CFRP sheets and the rods are made of steel,the use of excessive resin inside the groove should ensure the desirable non-contact between carbon and steel (due to the potential of galvanic corrosion).Configuration 1 in Fig.9-10b is preferable than 2,as stresses are transferred through compression and not through direct tension under the slab.

Special detailing attention is especially needed in the case of shear strengthening of I-shaped cross sections,where direct debonding of the FRP in tension is initiated at the convex divergence point between the web and the bottom flange(Fig.9-10c).At this point,extra anchorage should be provided,similar to the one presented in Fig.9-13c.

9.2.2 Columns and walls

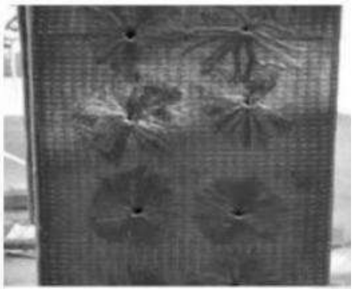
Compressed members can be effectively confined by externally bonded reinforcement with horizontally or spirally running fibres (see Section 6.2.3).Flexural reinforcement can also be provided with axially oriented externally bonded reinforcement.

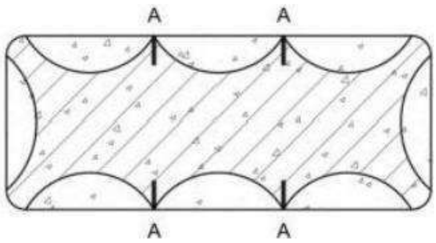
9.2.2.1 Confinement

The number of superimposed layers is obtained by the analysis described in Chapter 6 or 8 for static or seismic loading,respectively.

Rounding of the corners in columns should be done at the maximum possible radius (typically determined by the concrete cover).Minimum permissible radii at corners of rectangular cross sections are in the order of 20 mm for carbon or glass fibres and 10 mm for aramid fibres.

Concerning the application of FRP on rectangular columns or pier walls with large aspect ratio, the FRP does not actually confine the internal concrete structure if just applied to the surface. In order to achieve effective confinement, the FRP jacket should be constrained on both sides along the length through the use of spike anchors (or dowels or bolts) that anchor the jacket to the pre-existing structure[17], thereby creating shorter distances which are confined between anchors (Fig.9-11).


a)



b)

Fig.9-11 Effective confinement of RC column with large aspect ratio using through the thickness spike anchors: (a) photo, (b) effectively confined concrete in the cross section of the anchors

For confinement of columns where walls physically prevent complete wrapping of the elements, application of fan fibres15,16 can provide continuity to the reinforcement (see Fig.9-12).

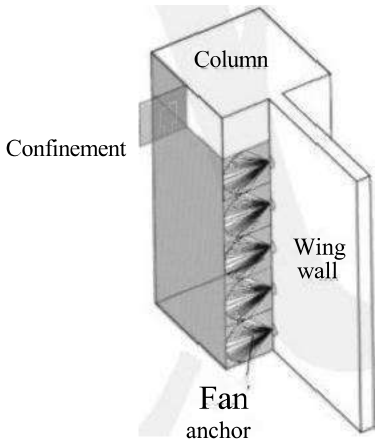


Fig.9-12 Fibre fan anchors for confinement of columns

Spike anchors provide a low cost solution to anchor the jacket to the existing structure and have been tested with very good results for the attachment of FRP jackets at the re-entrant corners of L-shaped cross section columns (Fig.9-1318). Tests evidenced that the partial depth anchors (Fig.9-13c,d) result in high effectiveness in terms of deformation capacity and strength increase, while the low extra benefit provided by full depth anchors (Fig.9-13e,f) is not justified by the high difficulty of installation.

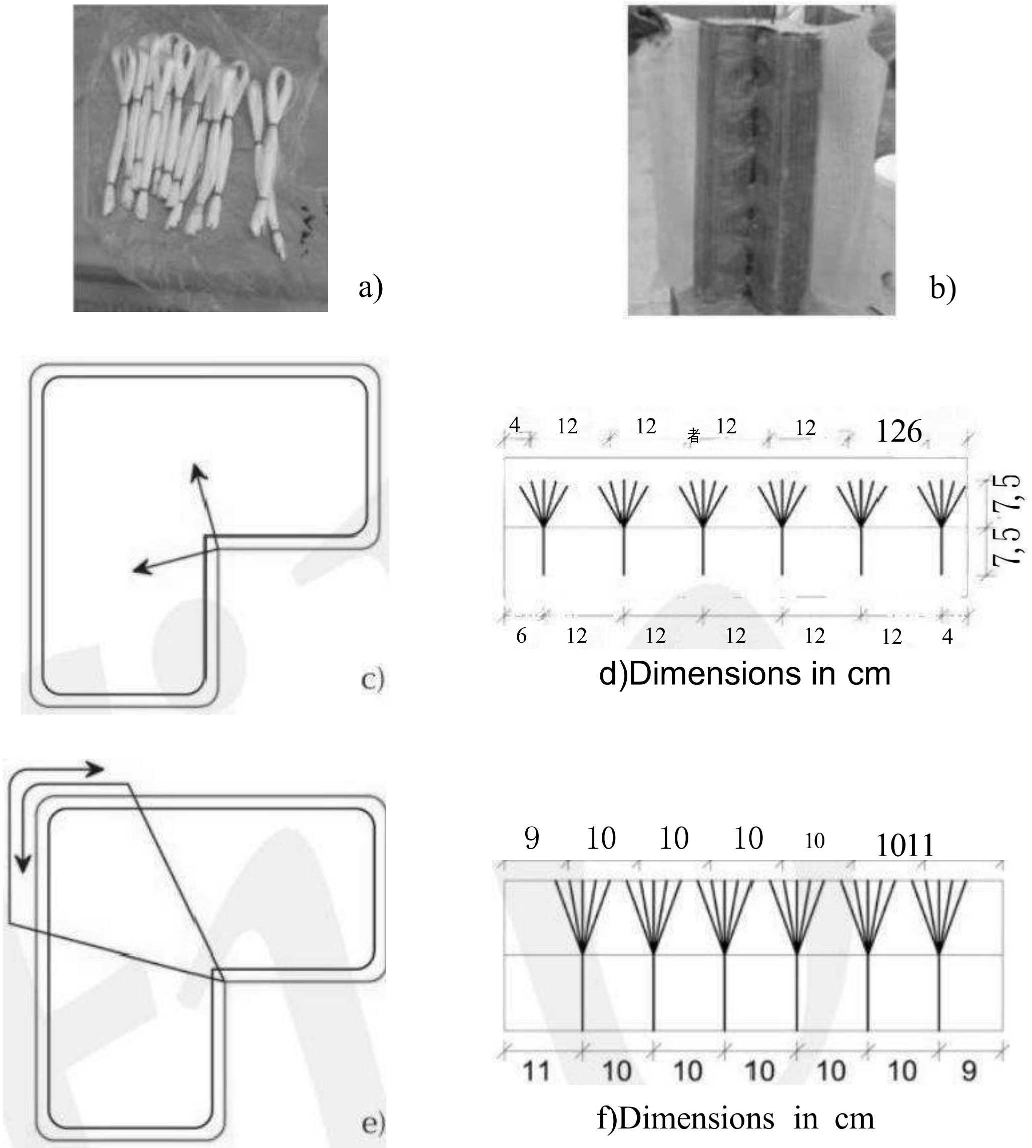


Fig.9-13 Spike anchors foreffective confinement of columns with re-entrant corners18:(a)spike anchors;(b)spike anchors applied at re-entrant corner;(c)strengthening configuration with partial depth anchors;(d)spreading of partial depth anchors;(e)strengthening configuration with full depth anchors;(f)spreading of full depth anchors

Overlapping of the jacket's ends in rectangular cross sections should be done on long sides(Fig.9-14a)in a way such that fracture of the FRP would occur prior to debonding. Typical minimum lap lengths are in the order of 200 mm for carbon fibre sheets with a nominal thickness about 0.12-0.14 mm.

When jacketing is applied at column ends to obtain a ductility increase,a 10 mm gap is recommended to allow for unrestrained rotation of the end cross section as well as to prevent damage of the FRP in compression(Fig.9-14b).

Fullwrapping with several strips of FRP along the height should be done with the lap joints in different sides (Fig.9-14c).

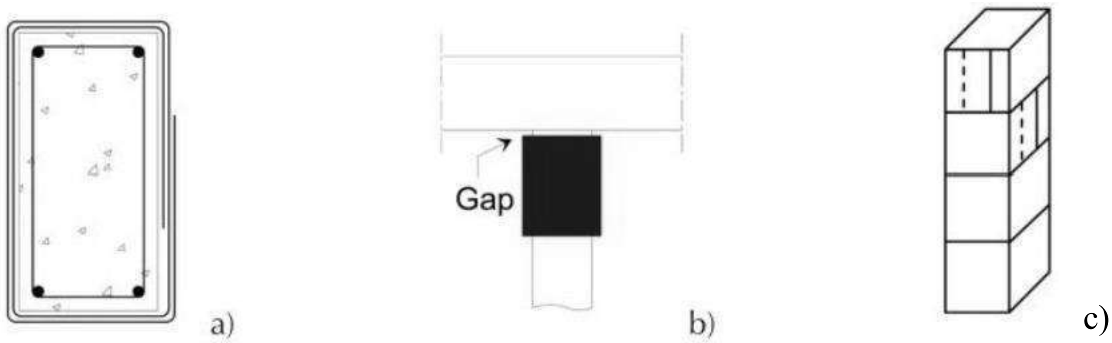


Fig.9-14 Details for confining of columns:(a)overlapping of jacket's end;(b)gap at column end;(c)lap joints in different sides

When jackets are provided to prevent lap-splice failures(e.g.at the bottom of columns),the FRP should extend at a height equal to at least the lap splice length plus 150 mm (Fig.9-15).

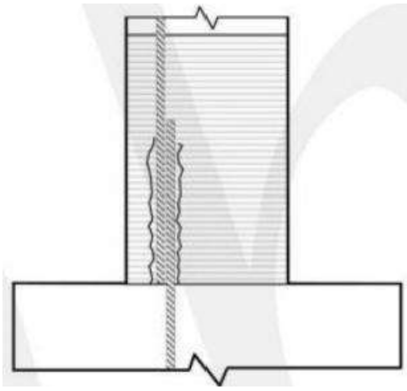


Fig.9-15 Local jacket to prevent lap-splice failure

9.2.2.2 Flexural reinforcement

Anchorage of flexural reinforcement of columns/walls into the beam-column/wall or footing-column/wall joints should be provided.A possible solution based on the use of spike anchors[12,19] is illustrated in Fig.9-16.

For flexural strengthening in seismic areas longitudinal FRP should be combined with transverse jacketing,which will delay buckling of the FRP reinforcement due to load reversals.

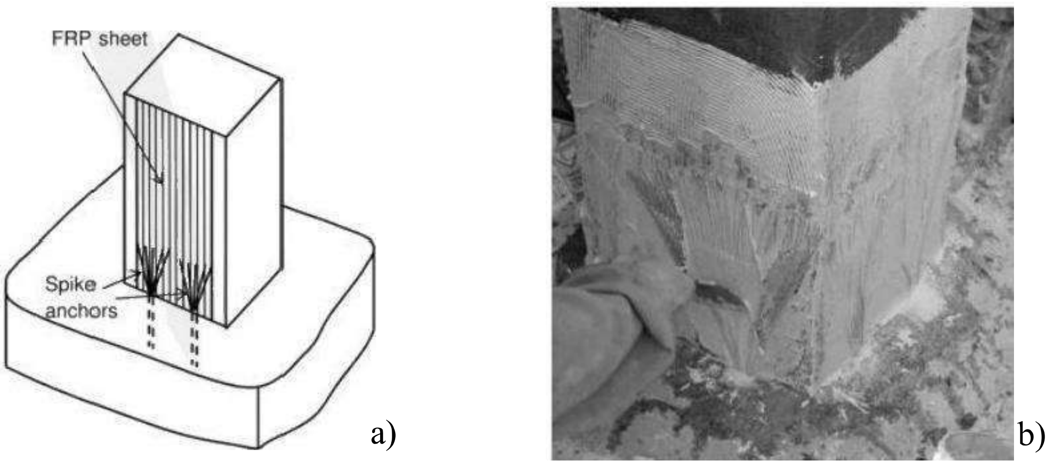


Fig.9-16 Use of spike anchors in flexural strengthening of columns with FRP sheets:(a)concept,(b)photo

Anchorage at the footing zone of columns and walls has also been provided by U-shaped devices made by fibres or steel/composite laminates glued or bolted (Fig.9-17) Fixing systems using bolts drilled in the footing web of columns or walls and crossing the reinforcement splayed on the web(Fig.9-18)have been experimented by one study[20]. However,such solutions shouldbe adopted with caution,as premature rupture of the fibres at sharp corners is quite likely to reduce the effectiveness of the strengthening scheme.

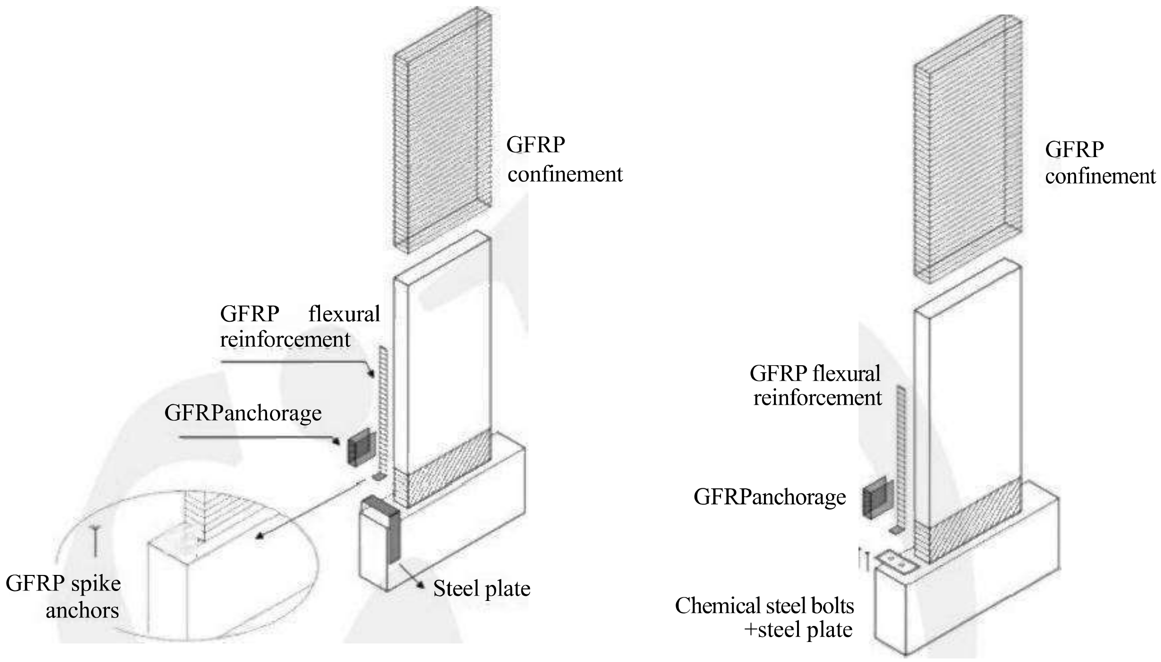


Fig.9-17 Anchorage systems at footing zone of shear walls[21]

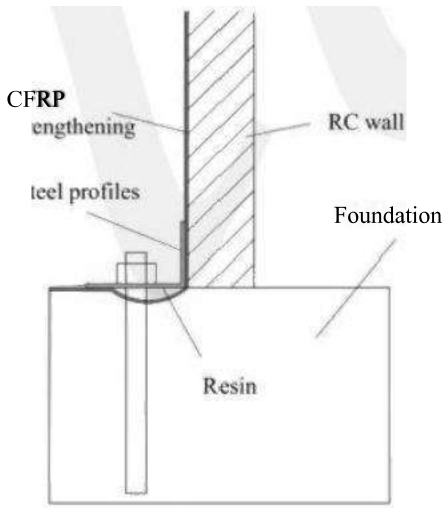


Fig.9-18 Anchorage system at footing zone[20]

9.2.2.3 Shear reinforcement

Minimum permissible radii at corners of rectangular cross sections are in the order of 20 mm for carbon or glass fibres and 10 mm for aramid fibres.

As in the case of confinement,full wrapping with several strips of FRP along the height should be done with the lap joints in different sides (Fig.9-14c).

Shear strengthening of columns between partial height infill walls should be done along the full column height, not just in the free part (Fig. 9-19).

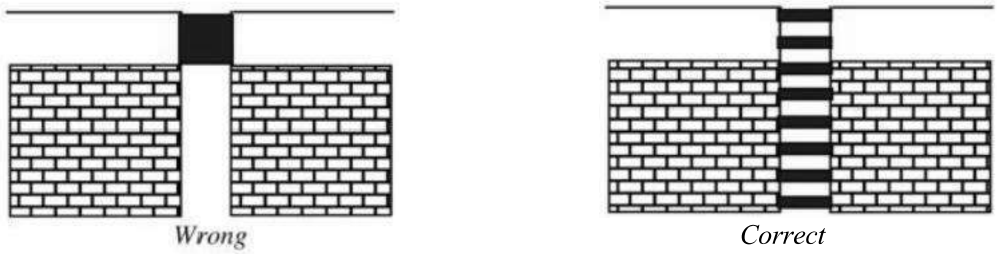


Fig. 9-19 Shear strengthening of column between partial height infill walls

9.2.3 Joints

FRP sheets or fabrics in beam-column joints should be extended beyond the joint area and anchored e.g. through transverse wrapping (Fig. 9-20a), possibly combined with bolts (Fig. 9-20b).

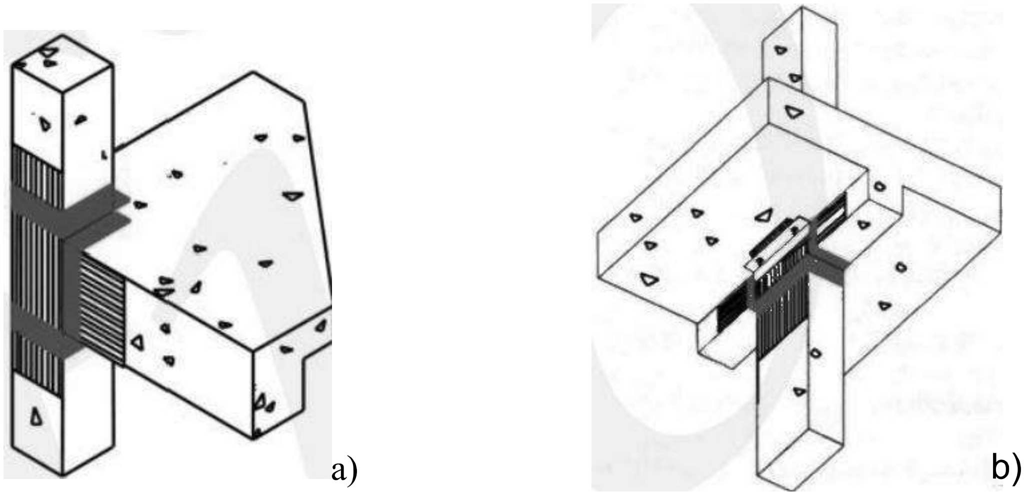


Fig. 9-20 Typical configurations for shear strengthening of beam-column joints and anchorage outside the joint. (a) Exterior joint, (b) interior joint[22].

9.2.4 Protection of the FRP

Some form of finishing may be required for aesthetic purposes. In terms of fire protection, possible occurrence of accidental damage and protection against ultra violet radiation and humidity, if exposed to the environment, a finishing layer can be crucial to the long-term integrity of the strengthened structure (see also Section 4.5). Different types of finishing layers can be provided such as painting, shotcrete, fire-resistant mortars or fire protection panels. These finishing layers should be applied according to the specifications given by the manufacturer. Compatibility between the externally bonded reinforcement and the finishing layer should be proved. If finishing layers or toppings involve heating, this should be shown not to damage the bond integrity.

9.3 Detailing for strengthening with NSM FRP

9.3.1 General

In the NSM strengthening technique the FRP reinforcement is embedded in grooves cut onto the surface of the member to be strengthened.Groove dimensions,groove distance from the edge of the member and distance between adjacent grooves are important details to avoid premature debonding failure of the strengthened member.

Most detailing rules,e.g.those on groove dimensions and edge distance,are valid for any type of strengthening system;such rules are described next.

9.3.2 Beams

9.3.2.1 Flexural strengthening

The available research shows that CFRP elements of rectangular cross section(flat strips) are more effective than elements of round or square cross section[23]for flexural strengthening in accordance with the NSM technique.

Bond tests and analytical models have shown that the pull-out load increases with the increase of the width of the strip[24,25]and have also verified that strips installed as deep as possible into the grooves have provided the highest pull-out load and energy dissipation. Both conclusions indicate that the grooves should have the maximum possible depth (limited by the thickness of the concrete cover)and the strips should have the largest possible width.

For instance,for the same FRP strengthening ratio,the C2 configuration in Fig.9-21 is preferable over C1,as decreasing the distance between strips may result in end concrete cover separation due to the interaction between the concrete fracture surfaces of consecutive FRP strips.Moreover,between C2a and C2b configurations,the latter provides the best bond performance,due to the high level of confinement that the surrounding concrete applies to the strip.

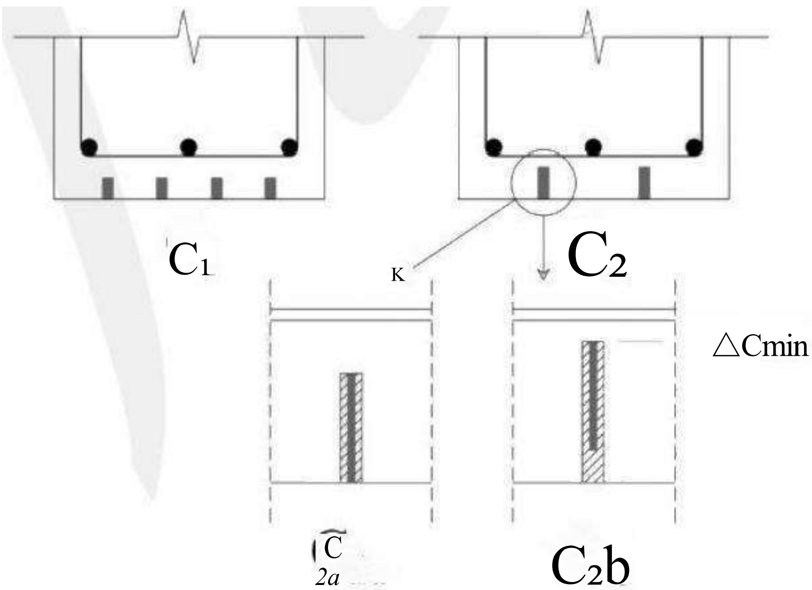


Fig.9-21 Flexural strengthening configurations with NSM strips(dimensions in mm)

As a result of concrete casting conditions,the concrete under longitudinal rebars may have a concentration of voids and defects.Therefore,the effectiveness of NSM FRP has a tendency to decrease as the number of longitudinal rebars increases.The reduced concrete quality under rebars as well as interactions between rebars and NSM FRP call for placement of the NSM reinforcement preferably between rebars(C2 in Fig.9-22)and not directly below them(C1 in Fig.9-22).

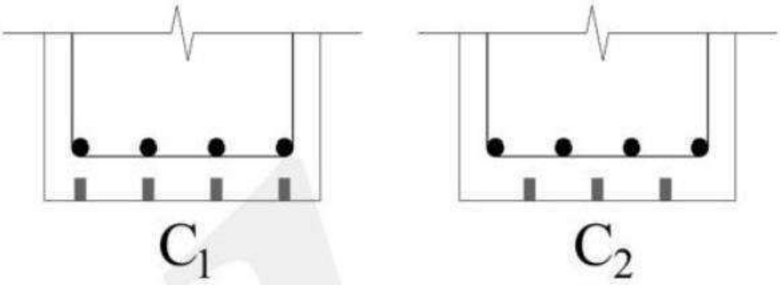


Fig.9-22 Configuration C2 is more favourable than C1 in terms of bond conditions

9.3.2.2 Shear strengthening

The NSM FRP technique can also be used to increase the shear strengthening of RC beams,by applying the external reinforcement in grooves on the lateral faces of the beams(see Section 5.4.3).Amongst the possible inclinations of the FRP elements,those in the range between 45°to 60°degrees with respect to the member axis are the ones that provide the highest shear strengthening effectiveness26.

The depth of the grooves where the FRP elements are installed shouldbe as high as possible in order to provide higher confinement to the FRP elements and to mobilise a larger resisting fracture surface from the surrounding concrete[2]7.In this perspective,the configuration illustrated in Fig.9-23 can be adopted,provided that beams are sufficiently deep.

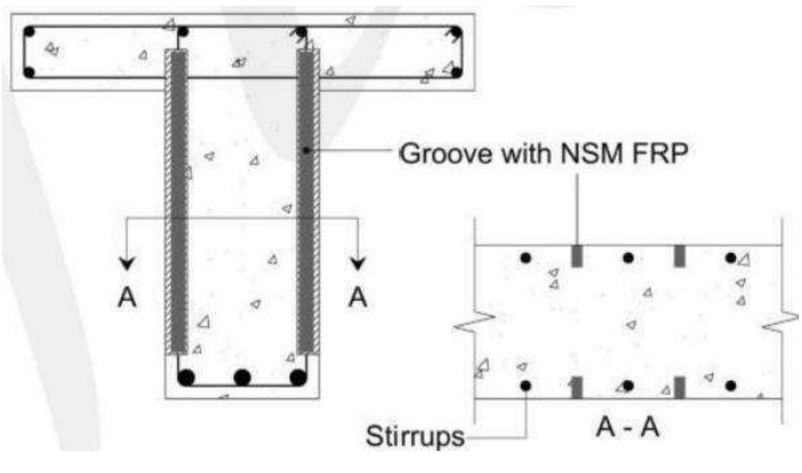


Fig.9-23 NSM configuration for beams of relatively deep cross section

The NSM reinforcement used for shear strengthening of T-beams can be anchored in epoxy-filled holes drilled in the flange at the location of the web side grooves.Test results28 have shown that this anchorage measure is highly effective in preventing debonding of the NSM reinforcement from the beam sides, thereby enhancing the capacity of the strengthened beam.

The contribution of the anchored NSM reinforcement to the shear capacity can be approximately computed with the model presented in Chapter 6, considering that the length of the NSM reinforcement is larger when this is anchored through the beam flange.

9.3.2.3 Groove dimensions

The groove dimensions have to meet two main requirements. First, the groove depth must be smaller than the existing minimum concrete cover to avoid damage to the internal steel reinforcement. Secondly, the NSM reinforcement must be fully embedded in the groove over its full length and, in the case of NSM round bars, a sufficient cover thickness must be provided to the bar for proper bond.

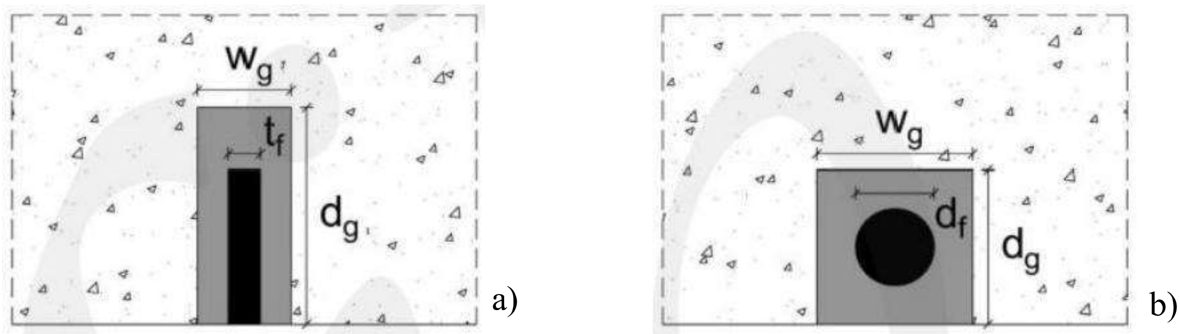


Fig. 9-24 Geometrical parameters of NSM strengthening: (a) strips; (b) round bars

Based on the first requirement, the groove depth should comply with the following limit (DIBT 2010)29:

$$d_g \leq c - \Delta c \tag{9-3}$$

where:

- d is the depth of the groove
- -c is the existing minimum concrete cover
- -△c is the additional safety value calculated as: △c=△Caw+△cc+△cmember where:
  - ·△Cadepends on the saw use-at least 1 mm
  - ·△cut is an additional safety value for the depth-at least 2 mm
  - ·△cmembedepends on the type of member: 0mm for slabs, 2 mm for beams

Based on the second requirement, the following limit should be applied for NSM strips (Fig. 9-24a):

$$t_f + 1.5 \leq w_g \leq t_f + 3.0 \tag{9-4}$$

where wg is the width of the groove (mm) and t, is the thickness of the strip (mm).

Moreover, based on the second requirement, the following limits should apply for NSM round bars (Fig. 9-24b):

$$w_g \geq d_f + 2.0 \tag{9-5}$$

$$d_g \geq \alpha \cdot d_f \tag{9-6}$$

where d,is the bar diameter(mm)and a is a coefficient depending on the splitting tendency of the bar. In the absence of specific data, the following values can be used[30]: α=1.5 for bars with moderate surface deformation (e.g. smooth, sand-blasted, sand-coated) and α=2.0 for bars with pronounced surface deformation (e.g. ribbed).

9.3.2.4 Groove distance from edge of the member

The net distance a between the groove and the concrete edge should exceed a minimum value depending on the maximum aggregate size and the width of the groove (Fig. 9-25a):

$$a_e \geq \max\left(2w_g,\ d_a\right) \tag{9-7}$$

where w is the width of the groove andd is the maximum aggregate size of the concrete.

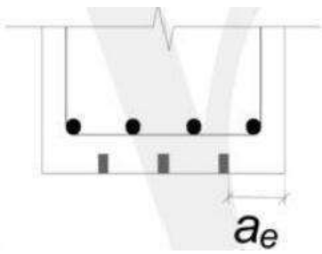


Fig. 9-25 Distance a between the groove and the concrete edge

If NSM reinforcements are applied on both sides of the edge (i.e. in the case of flexural strengthening on both tension face and side face of a concrete beam) the net edge distance on each side should comply with the following limit:

$$a_e \geq \max\left(4w_g,\ d_a\right) \tag{9-8}$$

9.3.2.5 Distance between adjacent grooves

For the net clearance, ag, between adjacent grooves, the following limitations are recommended:

$$a_g \geq \max\left(4w_g,\ d_a\right) \tag{9-9}$$

and additionally for flexural strengthening:

ag≤(0.21。,4h) (9-10)

wherel is the distance between points of zero moment(for cantileverst。=28)and h is the total depth of the member.

9.3.2.6 Confinement of the NSM with transverse FRP

When round bars are used as NSM reinforcement,confinement of the NSM joint by means of an externally bonded FRP laminate with the fibres oriented transversely to the axis of the bar may be used to increase its bond strength.This effect was proved by a limited number of test results[31]where increases in bond strength of about 25%were measured.The presence of the EBR reinforcement significantly increases the transverse stiffness of the cover of the NSM bar by providing it with a transverse external restraint. This benefits both the risk of cover splitting and the frictional resistances of the joint.

To achieve its full strength,transverse FRP should extend past the axis of the NSM bar by at least one effective bond length on each side of the bar.However,the tensile stress induced in the transverse FRP as a result of the confining action may be significantly smaller than its debonding strength,hence shorter lengths may be sufficient.The increase in bond strength provided by confinement and the length needed for the transverse FRP should be quantified by appropriate testing.

9.3.3 Columns and walls

The flexural strengthening of columns can be achieved also with NSM FRP bars that can be anchored in foundation blocks or in beam-column joints.In applications where NSM FRP may be subjected to compression,e.g.in seismic retrofitting,longitudinal FRP in the form of either sheets or NSM bars should be combined with transverse jacketing(Fig.9-26),which willdelay buckling of the FRP reinforcement in the highly compressed regions32,33.

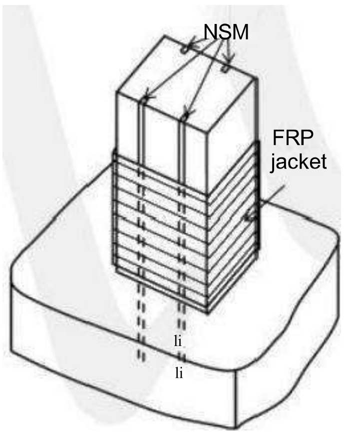


Fig.9-26 Transverse wrapping of NSM FRP to delay buckling in compression zones[32]

The guidelines above concerning the groove dimensions,the groove distance from edge of the member and the distance between adjacent grooves are valid also in the case of strengthening of columns and walls.