ABSTRACT
This work focuses on actual research development in the field of FRP strengthening of RC buildings. The main outcomes on these activities have been analyzed to provide possible recommendations towards a future update of EC8 part 3. The main strategies and driving principles for the seismic retrofit of existing structures have been discussed focusing on local and global interventions. In this framework, FRP confinement has been also analyzed, focusing in details on rectangular columns with high aspect ratio.

KEYWORDS
Confinement, FRP, Guideline, RC Buildings, Seismic Retrofit

1 INTRODUCTION
The most common strategies adopted in the field of seismic retrofit of existing structures are the restriction or change of use of the building, partial demolition and/or mass reduction, removal or lessening of existing irregularities and discontinuities, addition of new lateral load resistance systems, local or global modification of elements and systems.

In particular, the local intervention methods are aimed at increasing the deformation capacity of deficient components so that they will not reach their specified limit state as the building responds at the design level. Common approaches mainly include steel jacketing and externally bonded Fiber Reinforced Polymer (FRP). On the other hand, global intervention methods involve a global modification of the structural system; such modification is designed so that the design demands (often denoted by target displacement) on the existing structural and non-structural components are less than structural capacities. Common approaches mainly include: Reinforced Concrete (RC) jacketing, addition of walls, steel bracing and base isolation.

The above overview of the rehabilitation strategies shows that the structural performances of an existing building can be enhanced in different ways by acting on ductility, stiffness or strength (separately or, in many cases, at the same time); in each case, a preliminary assessment of the existing structure performances and the evaluation of the analysis results are necessary to select the rehabilitation method that meets the required performance targets.

Nevertheless, numerous factors influence the selection of the most appropriate technique and therefore no general rules can be applied. The present paper focuses on the potential of FRP for seismic strengthening of RC buildings highlighting the criteria for selecting the type of
intervention and discussing the outcomes of some related research activities lately performed by the authors.

2 FRP STRENGTHENING IN SEISMIC ZONES

EC8 – Part 3 offers the possibility, as an alternative to more traditional strengthening techniques such as RC and steel jacketing, to use composite materials for seismic retrofit of under-designed RC structures. However, EC8 – Part 3 only indicates that FRP can be used to increase shear strength of members, provide ductility to concrete and prevent lap splice failure. The authors believe that preliminary insights should be given about the overall target of the FRP intervention on the building; a possible strategy outline is proposed herein.

First, it could be important to point out that stiffness irregularities cannot be solved by applying FRP. Strength irregularities can be modified by strengthening a selected number of elements, however, attention should be paid that the global ductility is not reduced.

From the seismic standpoint, FRP strengthening could be regarded as a selective intervention technique that could allow:

a) increasing the flexural capacity of deficient members, with and without axial load, through the application of composites with the fibers placed parallel to the element axis;

b) increasing the shear strength through the application of composites with the fibers placed transversely to the element axis;

c) increasing the ductility (or the chord rotation capacity) of critical zones of beams and columns through FRP wrapping (confinement);

d) improving the efficiency of lap splice zones, through FRP wrapping;

e) preventing buckling of longitudinal rebars under compression through FRP wrapping;

f) increasing the tensile strength of the panels of partially confined beam-column joints through the application of composites with the fibers placed along the principal tensile stresses.

The driving principles of the FRP intervention strategies should be:

a) all potential brittle collapse mechanisms should be eliminated: failures such as shear, loss lap splice, bar buckling and joint shear should be prevented

b) the global deformation capacity of the structure should be enhanced, either by: b1) increasing the ductility of the potential plastic hinge zones without changing their position, or, b2) relocating the potential plastic hinge zones by applying capacity design criteria. In this latter case, the columns should be strengthened in flexure with the aim of transforming the framed structure into a highly dissipating mechanism with strong columns and weak beams.

For case a), when eliminating potential brittle failure mechanisms, the relative strengthening modalities are quite straightforward. The most common case is potential shear failure, for which a strengthening of the shear mechanism should be sought. More peculiar cases are those of longitudinal bars lap splices and buckling. In the former case, due to either bond degradation in splices or insufficient splice length, the relevant regions of potential plastic hinge formation should be adequately confined through an FRP wrapping; in the latter case of bar buckling, the strengthening intervention should consist in confining the potential plastic hinge zones where the existing transverse reinforcement cannot prevent the bars post-elastic buckling.

For case b), when all possible brittle and storey mechanisms have been prevented, it is necessary to assess to which extent the structure could exploit its ductility. This can be done, for example, through a nonlinear pushover analysis, now adopted and codified in the most modern seismic codes. Usually, it is requested to check if the structure can actually ensure a
given ductility, expressed by a pre-selected behavior factor, or, which is the same, if it is able to attain a given target displacement. Such analysis allows identifying the elements whose local collapse, due to ductility exhaustion, prevents the structure from exploiting its global ductility and from reaching the target displacement. Thus the required global deformation capacity can be obtained either by b1) or b2) strategies. In the former case, the deformation capacity of elements that collapse before the global target displacement is attained have to be increased. The deformation capacity of beams and columns can be measured by the chord rotation $\theta$, that is, the rotation of the chord connecting the element end section with the contraflexure section (shear span). Generally, the plastic deformation capacity is controlled by the compressive behavior of concrete. An intervention of FRP-confinement on such elements (usually columns) increases the ultimate compressive strain of concrete, thus determining a ductility increase of the element.

In the latter case, the overall resisting mechanism should be changed in order to distribute the ductility request over a larger number of elements. This can be achieved by relocating all potential plastic hinges by applying the capacity design criteria. The application of the capacity design criteria implies the elimination of all potential plastic hinges in columns. In “weak column-strong beam” situations, typical of frame structures designed for gravity loads only, the columns’ cross sections are under-designed both in terms of geometry and reinforcement. In such case, it is necessary to increase their flexural strength with the objective of changing the structure into a “strong column-weak beam” situation. It should be noted that, this strategy implies an increase of shear demand on columns due to the flexural capacity increase. It is therefore necessary to perform the required shear verifications, and to eventually increase the shear strength in order to avoid brittle failure modes. Moreover, attention must be paid to the foundation systems as the increased seismic strength capacity leads to an overturning moments increase.

Within the outlined strategies, FRP confinement is a key technique to increase the seismic capacity of RC members. Existing analytical models for predicting the stress–strain behavior of FRP-confined concrete are mostly derived for cylindrical plain concrete columns. Square- and rectangular-section columns were found to experience less increase in strength and ductility than their circular counterparts. This is because the distribution of lateral confining pressure in circular sections is uniform, in contrast to square and rectangular sections, in which the confining pressure varies from a maximum at the corners and diagonals, to a minimum in between. In particular in the case of wall-like columns (ratio between sides higher than about 3), the effectiveness of FRP jackets is even more reduced (Prota et al. 2006). To determine the effective lateral confining pressure, some researchers proposed to transform the rectangular section into an equivalent circular section (e.g. circumscribed, inscribed, or with an equivalent cross-sectional area). A more refined iterative approach has been proposed by Lignola et al 2009a, based on solid mechanics. Confinement models based on regression analyses are very sensitive to the value adopted for the ultimate FRP strain: in fact, FRP ultimate tensile strain determined experimentally according to flat coupon tests is not reached at the rupture of the FRP jacket in confined concrete columns compression tests (reasons for this have been provided by many authors, a summary is in Lignola et al 2008a). The ratio between the two strain values is termed “efficiency factor”, $\beta$. If the effective lateral confining pressure is inserted in a confinement model whatever is the material of the confining device, the scattering between theoretical and experimental results can be drastically reduced and a single expression can be formulated to predict benefits provided by confinement, e.g. for concrete, independently from the materials used as confinement device. It is highlighted that also more refined iterative confinement models may need a stop criterion given by the effective failure of the FRP jacket.
In the case of rectangular cross-sections with high aspect ratio (structural walls), the failure is strongly affected by the occurrence of premature mechanisms (compressed bars buckling and unrestrained concrete cover spalling), while nowadays slender structural wall members are usually designed without special prescriptions [Lignola 2006].

3 RESEARCH DEVELOPMENTS

The present section discusses two specific aspects of FRP seismic rehabilitation. First, the confinement of RC members is analyzed and significant research outcomes on the behavior of rectangular cross-sections with high aspect ratio are dealt with. Then, the strategy based on increasing the global displacement capacity without relocalizing plastic hinges is discussed; the design procedure is outlined and its validation by comparison to the experimental results on a real scale structure is reported.

3.1 Confinement

The analysis of the behavior of hollow RC piers, peculiar of bridge constructions to maximize structural efficiency of the strength-mass and stiffness-mass ratios, allowed the confinement of circular and non-circular and also of slender structural wall members, peculiar of buildings, to be studied in details. A refined numerical iterative procedure and a detailed nonlinear confinement model was provided for the analysis of hollow RC columns (Lignola et al 2008b). Even though, to provide a direct, practical tool, oriented to the profession more than a nonlinear refined iterative analysis, it was evaluated the opportunity to simplify the analysis, considering the effect of confinement on the walls composing the hollow cross-section. A preliminary Finite Element Method (F.E.M.) analysis has been conducted (Lignola et al 2009b) in the elastic range to evaluate the stress field generated by external wrapping on confined wall members. The arch-shaped paths of the confining stresses rapidly changes in a straight distributed confinement stress field moving away from the corner.

The results of the previous works suggest that a reliable numerical procedure to predict structural wall behavior under combination of flexure/shear and compression should include appropriate models for compressed bars buckling, concrete cover spalling and, of course, confined concrete behavior. If compressed bars buckling and concrete cover spalling are neglected inaccurate ductility predictions may be given. In these cases confinement models may be successfully used to predict essentially the strength of the column if the evaluation is limited to the occurrence of buckling of the compressed steel reinforcement bars.

Usually confinement does not change the actual failure mode for walls, but it is able to delay bars buckling, restraining also concrete cover spalling, and to let compressive concrete strains attaining larger values, thus resulting in higher load carrying capacity of the member and in significant ductility enhancement. The strength increase in confined concrete due to FRP wrapping turns into load carrying capacity increases mainly in the elements loaded with small eccentricity (it is clear that close to pure bending load the effect of concrete strength enhancement is not relevant because failure moves to tension side and, at lower levels of axial load, i.e. at higher eccentricities, also the influence of reinforcement buckling on the element behavior is less significant).

A confinement model, recently proposed, is based on solid mechanics in plane strain conditions and able to predict the fundamentals of the behavior of solid and hollow circular (Lignola et al 2008b, 2009c) and solid square (Lignola et al 2009a) members confined with FRP. A secant approach is used to account for the nonlinear behavior of concrete. The key innovative aspect of the proposed model is the evaluation of the contribution of confining
stress field neither equal in the two transverse directions $x$ and $y$, nor uniform along those
directions. The effect of confinement is evaluated in each point of the cross-section explicitly
considering a plasticity model for concrete under triaxial compression. The model traces the
different confinement effectiveness and lateral stress field inside the cross-section and it
allows to evaluate, at each load step, the multiaxial state of stress, and eventually the failure,
of the concrete or the external reinforcement: i.e. effective FRP strain at failure, Lignola et al
(2008a) and Zinno et al. (2009). The lateral-to-axial strain relationship provides the essential
linkage between the response of the concrete column and the response of the FRP jacket in a
passive-confinement model. The ultimate strength surface (Figure 1), $\rho = r(\theta, \xi) f'_{c}$ with failure
parabolic meridians $r$ (Elwi and Murray 1979) is formulated in the Haigh–Westergaard stress
space defined by the cylindrical coordinates of hydrostatic length ($\xi$), deviatoric length ($\rho$) and
Lode angle ($\theta$). In the ultimate surface equation the only unknown is the confined concrete
strength $f_{cc}$ and it can be iteratively evaluated. It is noted that the cited model is the basis for
the equation reported, for instance, in the ACI 440.2R (2008) code or Mander et al. (1988), to
evaluate the cylindrical triaxial confined concrete strength $f_{cc}$ given a uniform confining
pressure $f'_{l}$:

$$
\frac{f_{cc}}{f_{c}} = 2.25 \sqrt{1 + 7.9 \frac{f'_{l}}{f_{c}} - 2 \left(\frac{f'_{l}}{f_{c}}\right)^2} - 1.25
$$

(1)

Figure 1 Ultimate strength surface (a); in the Rendulic plane (b); in the deviatoric plane (c).

Again, even though a refined nonlinear confinement model was provided for the analysis of
circular and noncircular RC columns (e.g. Lignola et al 2009a, 2009c), to provide a direct,
practical tool, oriented to the profession, it was provided also a simplified confinement model
for wall-like cross-sections (arch-shaped path of confining stresses was seen to rapidly change
in a straight field moving away from the corner (Lignola et al 2009b)). According to this
alternative simplified approach, that gives rather accurate results despite the heavily reduced
computational effort (no iterations are needed), the confining stress field is only parallel to the
longer side of the cross-section, thus neglecting the confinement in the shorter direction and,
confining pressure can be assumed equal to:

$$
f'_{l} = 2 \frac{E_{frp} \varepsilon_{frp}}{h} \quad \text{assuming cross-section height } h < \text{ base } b
$$

(2)

Assuming zero stress for the minimum principal stress, confining pressure $f'_{l}$ equal to the
intermediate principal stress, $f_{cc}$ as the maximum principal stress the following approximated
equation is derived by the failure surface:

$$
\frac{f_{cc}}{f_{c}} = 1 + 1.42 \left(\frac{f'_{l}}{f_{c}}\right) - 1.40 \left(\frac{f'_{l}}{f_{c}}\right)^2 + 0.30 \left(\frac{f'_{l}}{f_{c}}\right)^3
$$

(3)
where \( f'/f' < 1.3 \). Eq. (3) can be also used to evaluate the stress-strain relationship for confined concrete in slender walls according to the procedure proposed by Spoelstra and Monti (1999), relying on an iterative procedure through which the stress-strain curve crosses a family of curves at constant confinement pressure, at each point induced by the FRP jacket subjected to the corresponding lateral expansion. It is highlighted that in the case of wall confinement the response of concrete may show an high load carrying capacity loss (e.g. more than 20%) before FRP failure and therefore numerical simulation can be concluded due to the high capacity loss rather than due to the failure of the confining material.

Experimental campaign conducted on wall-like columns (Prota et al. 2006) confirmed that significant strength increases can be achieved by FRP wrapping: the number of plies does not play a major role on the axial strength while it gives improvements in terms of axial ductility. The failure of these walls determines the bulging of the FRP laminates occurring at fiber strains far below the ultimate values provided by the manufacturers.

A theoretical model has been proposed in Lignola et al (2008a) suggesting an upper bound of the efficiency factor \( \beta \) (because it neglects stress localization and premature failures). To avoid an iterative procedure the bond between concrete and FRP is also neglected providing a direct closed form solution (assuming the three-dimensional Tsai-Wu failure criterion):

\[
\beta = \left(1 - \nu_{LT}^2 + \nu_{LT} t \frac{R}{L} + \nu_{LT} t \frac{1}{R}\right)^{1/2} \left[1 + \left(\frac{f_a}{f_f} t \frac{R}{L}\right)^2 + \left(\frac{f_a}{f_f} \left(\frac{v_{LT} - v_{LT} t \frac{R}{L}}{f^L/f_f} - \frac{v_{LT} - v_{LT} t \frac{1}{R}}{f^L/f_f} + \frac{v_{LT} - v_{LT} t \frac{1}{R}}{f^L/f_f} f^L/f_f^2\right)\right)^2\right]^{1/2} \tag{4}
\]

The sensitivity of the involved parameters has been discussed in Lignola et al (2009d), where it was shown that the main parameter driving coefficient \( \beta \) is the FRP composite relative strength \((f_a/f_f)\) and GFRP presents the highest dependence on the analyzed parameters. In this sense the proposed Eq. (4) can be simplified assuming typical values for Poisson’s ratios \((\nu_{TL} \text{ and } \nu_{LT})\). There is a research need to collect and publish in future confinement experimental works also those FRP mechanical (orthotropic) properties.

To better limit the range of variability of the effective FRP strain in confinement, a second model was proposed (Zinno et al. 2009) to analyze the effect of the stresses concentration at the free edge of the FRP jacket. Interlaminar stresses can cause premature failure of the FRP wrapping due to separation or delamination, thus limiting the confinement capacity of the FRP wrapping. This second model provides directly the effective FRP strain depending on the maximum interlaminar shear, \( \tau_{max} \), or on the normal tensile interlaminar peel stress, \( \sigma_{nmax} \), capacity:

\[
\varepsilon_{FRP} = \min \left\{ -\tau_{max} / t_u^{f_f} , \frac{\sigma_{nmax}}{t_u^{f_f}} \right\} \cdot \frac{1 + n \left(1 - e^{2\gamma_{EL}}\right)}{E t_n \gamma_{EL} \left(1 + e^{2\gamma_{EL}}\right)} \tag{5}
\]

where the parameters are described in detail in the original paper and their typical values are provided.

### 3.2 Seismic retrofit without relocalization of plastic hinges

In the case of structures designed for gravity loads only, the overall deformation capacity is usually governed by the limited rotation capacity in the plastic hinge at columns ends (inadequate cross-sectional dimensions and amount of longitudinal steel reinforcement). A seismic upgrade intervention targeted at increasing the overall structure deformation capacity can be pursued by FRP columns confinement. Indeed, columns wrapping allows enhancing the ultimate concrete compressive strain; this corresponds to an increase of curvature ductility
that, assuming a plastic hinge length not significantly affected by the upgrade intervention, determines a proportional increase of the plastic hinge rotation capacity. Because confinement using composite materials at columns ends induces, for intervals which are typical of normal stress levels, a considerable increase in terms of sections ductility, but does not lead to a significant increase in strength, such kind of retrofit does not modify the strength hierarchy of the structure.

The outlined seismic strengthening strategy effectiveness was experimentally investigated within the European research project SPEAR (Seismic PErformance Assessment and Rehabilitation). Such project involved a series of pseudo-dynamic bi-directional tests carried out on a three-storey RC structure with an irregular layout at the ELSA laboratory of Joint Research Centre (JRC) in Ispra (Italy). The structure under examination was designed and built with the aim of creating a structural prototype featuring all the main problems normally affecting existing structures: plan irregularity, dimensions of structural elements and reinforcement designed by considering only gravity loads, smooth reinforcement bars, poor local detailing, insufficient confinement in the structural elements and weak beam column joints. The structure was subjected to pseudo-dynamic tests, both in its original configuration and retrofitted by using GFRP. The structure in its original configuration was subjected to experimental tests with maximum peak ground acceleration (PGA) of 0.20g. Since both theoretical and experimental results showed that the ‘as-built’ structure was unable to withstand a larger seismic action, a retrofit intervention by using FRP laminates was designed. Once the design of the GFRP retrofit was provided, the structure was subjected to a new series of two tests with the same input accelerogram selected for the ‘as built’ specimen but scaled to a PGA value of 0.20g and 0.30g, respectively. The design of the rehabilitation was based on deficiencies underlined by both the test on the ‘as-built’ structure and the theoretical results provided by the post-test assessment (nonlinear static pushover analysis). They indicated that a retrofit intervention was necessary in order to increase the structural seismic capacity; in particular, the theoretical results showed that the target design PGA level of 0.30g could have been sustained by the structure if its displacement capacity was increased by a factor of 48% (Di Ludovico et al., 2008a). In order to pursue this objective, the retrofit design strategy focused on two main aspects. First it was decided to increase the global deformation capacity of the structure and thus its dissipating global performance; such objective was pursued by confining column ends with two plies of GFRP laminates. In particular, the amount of FRP plies to be installed to provide the required ductility increase of plastic hinges at columns ends was determined based on the following steps: 1) maximum theoretical ratio between ultimate chord rotation demand and capacity, $\gamma = \frac{\theta_{u,demand}}{\theta_{u,capacity}}$ was determined; 2) the target rotation capacity was computed as $\gamma \theta_{u,capacity}$ and thus the corresponding design cross-section ultimate curvature, $\phi_{u,target}$ was evaluated; 3) the concrete ultimate strain, $\varepsilon_{cu,target}$, to achieve such curvature was computed based on cross-section analysis; 4) the amount of FRP plies ensuring the attainment of $\varepsilon_{cu,target}$ was evaluated. Moreover, the second design key aspect was to allow the structure to fully exploit the increased deformation capacity by avoiding brittle collapse modes. To achieve this goal corner beam column joint panels were strengthened by using two plies of quadri-axial GFRP laminates as well as a wall-type column for its entire length with two plies of the same quadri-axial GFRP laminates used for the above joints (see Figure 2). The assessment of structural global performance, before and after the strengthening intervention, was performed by nonlinear static pushover analysis in longitudinal direction (positive and negative X-direction, PX and NX, respectively) and in transverse direction (positive and negative Y-direction, PY and NY). In Figure 3, the theoretical base shear-top displacement curves for the ‘as built’ and FRP retrofitted structure are depicted with reference to direction NX (where the maximum
capacity-demand gap was recorded for the ‘as-built structure at the significant damage limit state LSSD). Figure 3b clearly shows that the FRP retrofit is able to greatly increase the global deformation capacity of the structure, slightly affecting its strength. The comparison between the seismic structural capacity and both elastic and inelastic demand is reported in Figure 4 for direction NX by using the Capacity Spectrum Approach (CSA) (Fajfar, 2000). Figure 4 clearly shows that column confinement provides the structure with significantly enhanced ductility, allowing it to achieve the theoretical inelastic demand by only modifying

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**Figure 2** – Column confinement and shear strength of corner joints (a); shear strength of wall-type column and retrofitted structure overview (b) [Di Ludovico et al. 2008a]

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**Figure 3** – Theoretical base shear – top displacement curves for ‘as-built’ (a) and FRP retrofitted structure (b), [Di Ludovico et al. 2008a]

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**Figure 4** – Theoretical seismic performance comparison at 0.3g PGA between ‘as-built’ (a) and FRP retrofitted structure (b), [Di Ludovico et al. 2008a]
the plastic branch of the capacity curve. After that columns and joints were wrapped with GFRP, the retrofitted structure was able to withstand the higher (0.30g PGA) level of excitation without exhibiting significant damage. After tests, FRP was removed and it was shown that the RC core was neither cracked nor damaged. The comparison between the experimental results provided by the structure in the ‘as built’ and GFRP retrofitted configurations highlighted the effectiveness of the FRP technique in improving global performance of under-designed RC structures in terms of ductility and energy dissipation capacity without significantly affect its strength (Di Ludovico et al. 2008b).

4 CONCLUSIVE REMARKS AND RECOMMENDED GUIDELINE UPDATE

The following recommendations can be made in order to update the existing EC8 – Part 3 provisions:

1) It is suggested to include an introductory section to the list of the three potential interventions using FRP. This section could provide principles about the main strategies that can be pursued when facing the retrofitting of RC framed structures. This could help the engineer to set the target of the intervention prior to designing specific FRP strengthening. This would also imply the addition of a section about shear strengthening of joints and flexural strengthening of columns.

2) It seems important to standardize the calibration process of confinement models by using the efficiency factor $\beta$, because the average absolute error of confinement models for circular cross-sections shows a remarkable decrease when the effective strain is considered. In particular, with respect to Eq. A.34 of EC8 – Part 3, it is recommended to provide more information about how to determine the adopted FRP jacket ultimate strain, $\varepsilon_{ju}$. This strain could be recommended to be the minimum among the following values:
   a) ultimate strain of the FRP jacket determined by means of flat coupon tests;
   b) strain value ensuring integrity of concrete with respect to mechanisms contributing to shear capacity of the member;
   c) strain of the jacket corresponding to the attainment of tridimensional Tsai Wu failure criterion. According to the discussion presented above, this strain can be obtained by multiplying the ultimate strain of FRP by the efficiency factor $\beta$;
   d) strain value corresponding to inter-laminar failure due to stress concentration at free edge of the jacket overlap, as discussed in the previous sections.

3) It is highlighted that simple geometrical considerations show that the confinement effectiveness factor (see Eq. A.36 in EC8 – Part 3) tends to have no physical meaning if the parabolas overlap, which occurs if $h<(b-2R)/2$ (assuming that $h<b$). With reference to these cross-sections, a recommendation could be added to compute the effective lateral pressure and the confined concrete strength according to Eq. (3) reported above.

4) It is also recommended to address the issue of FRP strengthening limits. When dealing with existing structures located in seismic zones, the engineer could find deficiencies due to either gravity loads or seismic loads. With this respect, the guideline should provide provisions about maximum strength increases depending on the type of actions.

5 ACKNOWLEDGEMENTS

The analysis of the test results was developed within the activities of Rete dei Laboratori Universitari di Ingegneria Sismica—ReLUIS (Research Line 8) funded by the Italian
Department of Civil Protection—Executive Project 2005-2008. The SPEAR project was funded by the European Community under the “Competitive and sustainable Growth” Programme (1998-2002) and was co-ordinated by Dr. Paolo Negro and Prof. Michael Fardis. The retrofit of the structure was supported by MAPEI S.p.a.

6 REFERENCES


