Chapter 3.2

MECHANICAL BEHAVIOUR OF MASONRY STRUCTURES STRENGTHENED WITH DIFFERENT IMPROVEMENT TECHNIQUES

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Abstract: The paper describes the main repair and strengthening techniques proposed to preserve historic masonry buildings subject to severe loading conditions. Based on experimental and modeling studies and on real cases applications, design and feasibility problems, execution and efficiency aspects of the interventions are discussed.

Key words: historic masonry; seismic improvement; strengthening and repair; injections; bed joint reinforcement technique; compatibility; FRP.

1. INTRODUCTION

A large number of materials and application techniques are available, and new ones are continuously being developed to strengthen and repair historic masonry buildings. The more recent developments mainly focus on finding appropriate solutions for satisfying at the same time conservation requirements, structural safety prescriptions and the specific, very variable structural problems of every single historic building. As far as this last issue is involved it must be pointed out that the same material and the same technique can give completely different results in terms of efficiency and type of structural behaviour. In such a content, the main factors involved are: the type of masonry (e.g. regular brick or rubble stone), the structural component (wall or vault), the combination of structural solutions (e.g. well connected or loose walls, floors made of masonry vaults or timber beams) and of actions (particularly in seismic or not seismic situations).

S.K. Kourkoulis (ed.), Fracture and Failure of Natural Building Stones, 137–156.

As a result, continuous effort is required to assess the real feasibility and efficiency of each technique in different application conditions and to give simple but reliable design rules.

The present paper briefly describes the researches that have been conducted at the University of Padova (based both on in situ and laboratory tests) on most of these topics regarding both traditional and innovative materials and solutions, under the basic assumption that all can give useful contributions if appropriately designed and applied.

Both stone and brick masonry structures are considered, with particular emphasis on the most severe loading conditions which they can be subjected to during their life (e.g. seismic actions, mainly for stone buildings and creep phenomena for heavy brick constructions). Case studies are also presented showing the major steps of the design process and the practical application of the research results.

2. STRENGTHENING OF MASONRY BUILDINGS IN SEISMIC AREAS

Stone masonry buildings are very common in many Mediterranean and North-European historical centres, often in seismic areas, and represent a typology particularly sensitive to horizontal load effects. In fact, they are often characterized by the use of poor materials and the construction of an irregular morphology (often revealing multi-leaf sections), affected by the presence of voids; moreover, they frequently show a general scarce collaboration among structural components (walls and floors or roof), which can led to particularly brittle collapse mechanisms (detachment of the layers and out-of-plane expulsions of material) (Giuffrè, 1993; Doglioni et al., 1994; Giuffrè and Carocci, 1999). Under these conditions, stone masonry constructions are particularly vulnerable under seismic loads, which basically represent the most severe actions for them.

From a structural point of view, it is well known that the proper behaviour under seismic loads is performed by constructions able to activate the so-called “box-behaviour”. It concerns the presence of effective connections between bearing walls, between walls and horizontal components (floors and roof), and the contribution of “sufficiently” stiff floors or roof (Tomazevic, 1999).

At a global level, the main structural problems of masonry buildings under seismic actions are due to:

- the lack of connections among structural components (wall to wall, walls to floors and roof),
- the presence of horizontal structures (floors and roof) having scarce in-plane stiffness,

- the irregular morphology, due to continuous modification, stratification and superposition that have occurred during time.

At the local level, additional vulnerability is due to the peculiarity of the multi-leaf stone walls and, in particular, mainly to:

- the weakness of the internal core,
- the lack of interlocking among the layers.

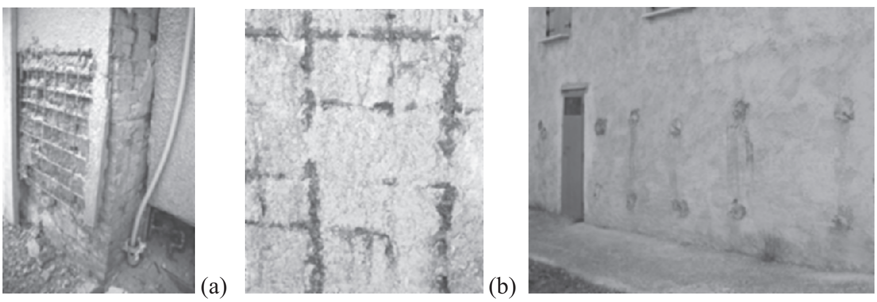
The limits in the knowledge of the mechanical behaviour of existing masonry structures under seismic actions were clearly evidenced by the effects of the last seismic events that occurred in Italy, in particular the 1997 Umbria-Marche earthquake, which drew many experts from several countries. Despite the not particularly high magnitude (5.6 R), a severe damage scenario was observed: brittle out-of-plane mechanisms of collapses (partial or global overturning of façades or at the corners), local expulsions and losses of material were largely detected in the buildings (Figure 1) (Binda et al., 1999; Penazzi et al., 2001). A severe damage was also detected in buildings retrofitted with common upgrading interventions (as substitutions of timber floors and roofs with reinforced concrete slabs and hollow tiles, jacketing, etc…), systematiccally applied after a previous earthquake incident in that area in 1979 (Figure 2). Such effects were due to the "hybrid" behaviour activated from the combination of new and old structures, unpredictable by the common available assessment procedures, based on the "a priori" assumption of the effecttive activation of the "box" behaviour. Moreover, many defects in the application of the consolidation techniques on walls (nets not sufficiently connected for the jacketing, unsuccessful injections of cement grouts not supported by proper investigations, etc.) were detected (Figure 3).

(a) (b) (c)

Figure 1. Out-of-plane mechanisms detected in the unretrofitted buildings: (a) overturning and collapse of the façade due to the lack of connection between floors and wall and among walls; (b) external layer expulsion due to the lack of connection of layers in the thickness of the wall; (c) collapse of the corner due to the excessive nearness of the openings.

(a) (b) (c)

Figure 2. Brittle mechanisms detected in the upgraded buildings: (a) collapse of the upper floor of a building due to the substitution of the timber roof with a heavier r.c. and hollow tiles mixed one, supported by poor masonry walls; (b) rear of the same building where a large overturning of the façade occurred, due to the high percentage of openings, combined with the presence of a heavy r.c. floor supported by a single layer of the poorly connected double-layer masonry; (c) detail of the eccentric loading on the wall.



(a) (b) (c)

Figure 3. Effects of uncorrect design and execution of consolidation techniques in seismic zone: (a) loss of effectiveness of a wall strengthened with jacketing due to the scarce connection of the steel net through the thickness and the lack of overlapping of the reinforcement at the corners; (b) corrosion of the steel net; (c) leackages of grouts due to the uninjectability of a wall.

After this experience the need of suitable procedures for seismic assessment of masonry building and even whole centers was clear. Based on the extensive in situ observations a specific survey form and a matrix of the collapse mechanisms detectable in various types of buildings (single, row or complex aggregates) were set up (Binda et al. 2004). In this context, computational procedures based on the macro-modelling approach were calibrated and updated, in order to be able to interpret the real behavior of old buildings (Valluzzi et al., 2004; 2005) and to perform simulation of possible intervention. Finally, the rehabilitation of traditional techniques was mainly recognized as the most cautious way of intervention in order to avoid possible incompatibility with original material and structures.

2.1 Improvement of the structural global behavior

First, techniques able to improve the box behavior of the buildings should be preferred, as the local strengthening of the masonry is practically ineffective (or even worsening) when global out-of-plane collapse can be activated (Penazzi et al., 2001). Tying is one of the most effective interventions improving the collaboration among walls and avoid the trigger of out-of-plane mechanisms, even maintaining lighter timber floors and roof (Tomazevic, 1999) (Figure 4). In comparison with heavier substitutions with r.c. horizontal structures and the connected concrete tie-beams, or reinforced injections, metal ties are also easy to be removed and allow improving the seismic response without altering the original structural function (Figure 5).

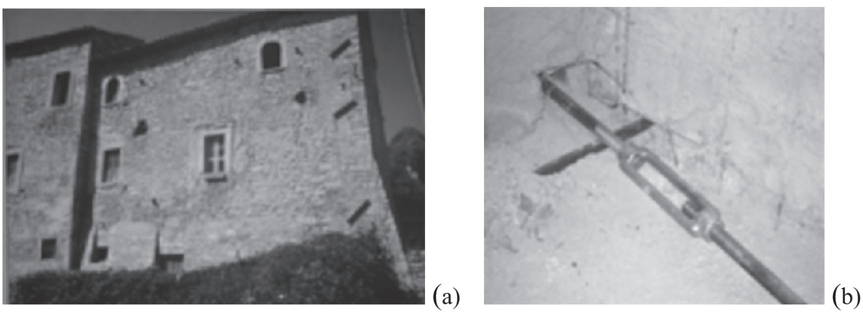


Figure 4. Application of metal ties in old masonry buildings: (a) the presence of traditional ties and their regular distribution garanteed the optimal behavior during the 1997 Umbria-Marche earthquake of this building in its original condition; (b) application of a modern metal tie in an existing building.

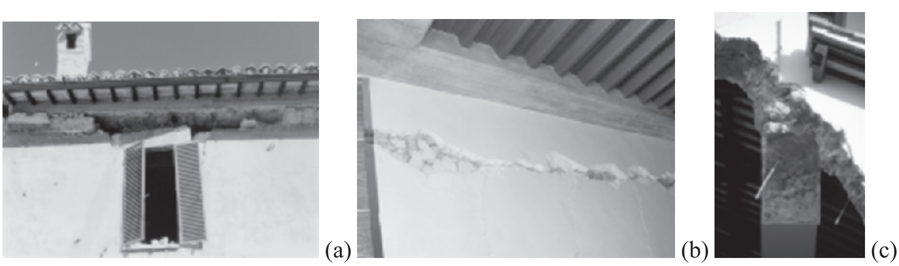
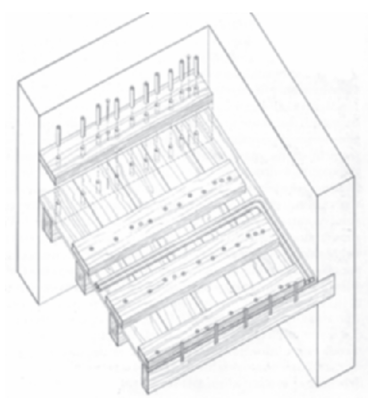


Figure 5. Effects of concrete tie-beam and reinforced injections in stone masonry buildings: the contiguity of two different materials (original masonry and r.c.) provokes a damage in the continuity of the wall and espose the wall to brittle out-of-plane collapses (a) (b); reinforced injections ineffective to restrain the main façade of a church (c).

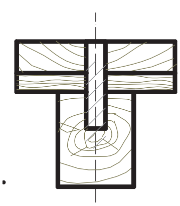
As for innovative materials, FRP (Fiber Reinforced Polymer), either in the form of laminates or rods, are recently considered also in restoration problems of monuments, due to their good mechanical properties and their ability to wrap structural components and even whole buildings (used as confinement). Nevertheless, regular surfaces (e.g. as in brick rather then stone masonry) are needed for the correct exploitation of the techniques, and durability aspects (resins are used at the interface with the substrate) should be carefully taken into account for real application on historic buildings or monuments. Mechanical performances related to the application of such materials will be shortly described in the following.

To contribute to the correct distribution of horizontal loads to the shear walls of the building, the in-plane stiffness of existing timber floors can be rehabilitated by using homogeneous materials together with the original ones. A strengthening technique has been recently experimentally set up at the University of Padova; it is performed by the placing of planks above the main beams of the floor and of one or more layers of boardings, positioned in different directions above the original extrados, connected by hardwood dowels, which has been proved to be of high efficiency with minimum obtrusiveness (Modena et al., 1998; 2004). The intervention is executed from the extrados of the floor, thus preserving possible precious intradoses, and enable the placing of systems (electrical, water, etc.) without impair walls at their base (Figure 6).

After stiffening, the resulting beam has a T-shaped section, whose web (original beam) and flange (new plank) are wooden made and a deformable connection between the flange and the web. The existing boarding, having a thickness of about 2-2.5 cm, which separates web and flange, is usually not considered as part of the compound section. The simplified design method for mechanically jointed beams with deformable fasteners was developed by



(a)



$$(EJ)_0 = E_1 J_1 + E_2 J_2$$
$$(\mathrm{EA})_\mathrm{S} = \mathrm{E}_1\mathrm{A}_1 + \mathrm{E}_2\mathrm{A}_2$$
$$(\mathrm{EA})_\mathrm{P} = \mathrm{E}_1\mathrm{A}_1 \cdot \mathrm{E}_2\mathrm{A}_2$$

$$(EJ)_{eff} = (EJ)_0 + \gamma \frac{(EA)_P}{(EA)_S} \cdot d^2$$

$$\gamma = \left(1 + \pi^2 \cdot \frac{(EA)_P}{(EA)_S} \cdot \frac{s_i}{K_i \cdot l^2}\right)^{-1}$$

(b)

Figure 6. Drawing of the stiffening intervention by using dry hardwood pins to connect planks and/or boardings to the original timber floors (a) and equations for calculation of the compound T-section (b).

by Möhler (1956) and assumed by the Eurocode 5 (2003). The compound section is characterized by the effective bending stiffness $(EJ)_{eff}$ as in the equations of Figure 6, where subscripts 1 and 2 refer to the stiffening plank and the existing beam respectively, d is the distance between the centroids of plank and beam, γ is the coefficient of connection efficiency ($0<\gamma<1$, 0 and 1 refer to infinitely deformable and rigid connection, respectively), l is the beam length, s_i is the dowels' spacing and K_i is the value of instantaneous slip modulus. This last parameter is conventionally estimated for the serviceability limit state (K_s) and for the ultimate limit state (K_u) from the experimental load-displacement curve obtained by push-out tests (UNI-EN 26891) (Figure 7).

Several experimental campaigns were carried out (1997-2004), testing various types of connection (wood or steel dowels, screws or rivets, also in combination among them), having different length, and without or in presence of different types of glue (vinyl, melamine, etc.) (Modena et al, 1997; 2004). Results showed that the best performance is given by dry wooden dowels having medium diameters (22-26 mm) and length of 130 mm, to which screws and a very thin layer of vinyl glue can give further support during the assembly phase (Figure 8). It is worth to remark that dry connections are more easily removable and therefore they represent the most suitable solution to the conservation criteria.

After experimental validation, the proposed strengthening technique was applied in the framework of some conservation work on the fifteenth-century complex Cà Duodo (Monselice, Padova, Italy), a relevant example of Venetian late gothic style (Modena et al., 1998). The building has three floors, having a relatively valuable intrados with tempera paintings, thus only the extrados was basically accessible (Figure 9). The strengthening intervention was carried out on all floors (first and second), for a total surface of 553 m^2. The 240 red pine beams presented a slender section (about 11x24 cm) and a

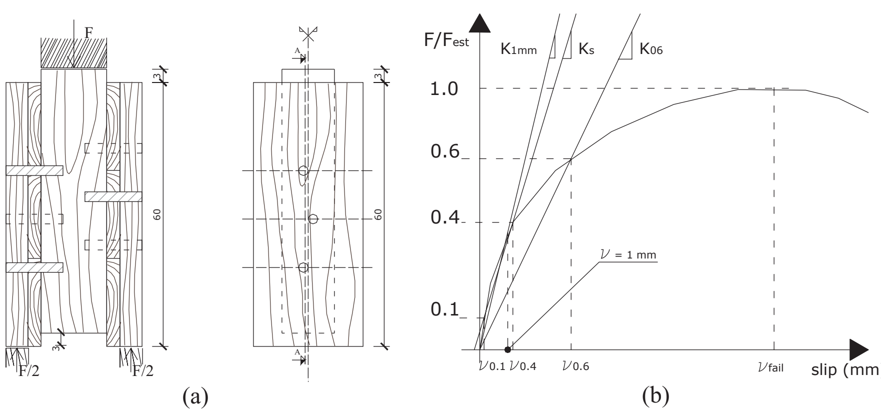
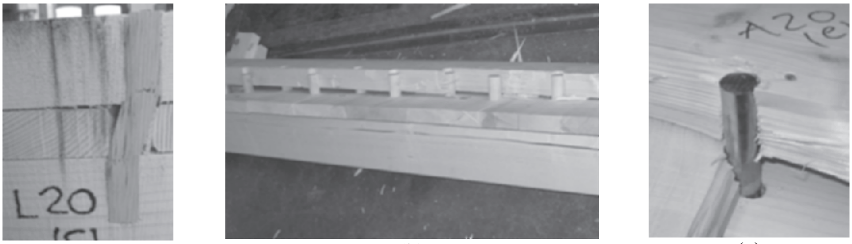
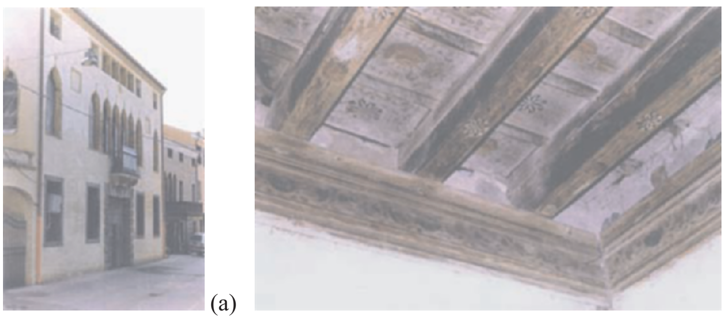


Figure 7. Push-out tests on compound sections: samples (a) and experimental curves for the definition of the slip moduli (b).



(a) (b) (c)

Figure 8. Results on application of various connections: rupture of dowel due to too small diameter (20 mm) (a); optimized diameter (22-26 mm) allow a combined shear-flexural behavior, avoiding both rupture of pin and damage of plank and beam, as checked after a bendng test (b); greatest wooden diameter (29 mm) or metal pins (even for small diameters, e.g. 12 mm) reveal to be too stiff and provoke damage of both existing and reinforcing materials (c).



(a) (b)

Figure 9. View of palazzo Ca' Duodo in Monselice (Padova) (a) and of the intrados of the floors (b).

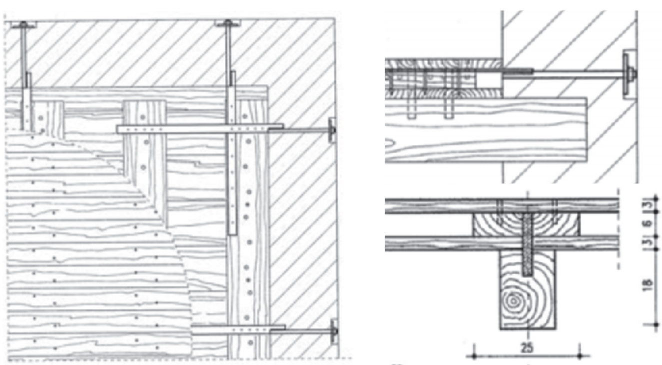


Figure 10. Sketches of the intervention executed in Ca' Cuodo: Drawing of the stiffening intervention by using dry hardwood pins to connect planks and/or boardings to the original timber floors, and details of the floor-to-walls connection (quotes in cm).

length of an average of 480 cm, spaced of about 56 cm. Preliminary investigations confirmed that the wood was in good condition, also in proximity to the bearing points on the walls.

The connections between beams and planks were made by hard-wood (beech-tree) dowels with diameter of 22 mm coupled with screws to facilitate the assembly phase. The longest beams (560 cm of span) were strengthened with 8x25 cm planks and 30 beech pins spaced from 11 to 30 cm, whereas for the shortest ones (430 cm of span) 5x25 cm planks 24 dowels were used (see details in Figure 10).

The stiffening planks were sawn before being placed on the boarding and their intrados were leveled in order to adhere perfectly to the existing surface. Planks were preliminarily fixed to the existing boarding with screws, in order to facilitate the following intervention phases; in case of no propping, they can also improve the adhesion between the straight planks and the often permanently deflected beams of existing floor. The position of the dowels is staggered of about half diameter from the longitudinal beam axis and they are not aligned in order to avoid longitudinal splitting. The pre-bored holes are about 1-2 mm smaller than the diameter of the pins and are cleaned with compressed air before forcing the dowels using a simple hammer (Figures 11(a,b)). It is also possible to use planks shorter than the floor span, avoiding their insertion into the load bearing walls. Such solution, still being statically admissible, allows placing all the technical installations in the floor thickness (Figure 11c) avoiding the opening of dangerous horizontal chases in the walls. The placement of installation and/or thermal and sound insulating panels in the empty space between adjacent planks can complete the intervention, if required.

2.2 Strengthening techniques for walls

Once the global behavior has been improved and the possibility of brittle out-of-plane effects has been eliminated or minimized, it is possible to increase the mechanical strength of shear walls, in order to raise the ultimate load. Among several solutions, injections are particularly able to consolidate the internal core of rubble multi-leaf stone masonry walls and improve the connection among the leaves (Valluzzi et al., 2004), with more mechanical

(a) (b) (c)

Figure 11. The phases of the intervention: fixing of the dowels on the strengthening planks (a) and detail of the technical installations placed between two planks (b).

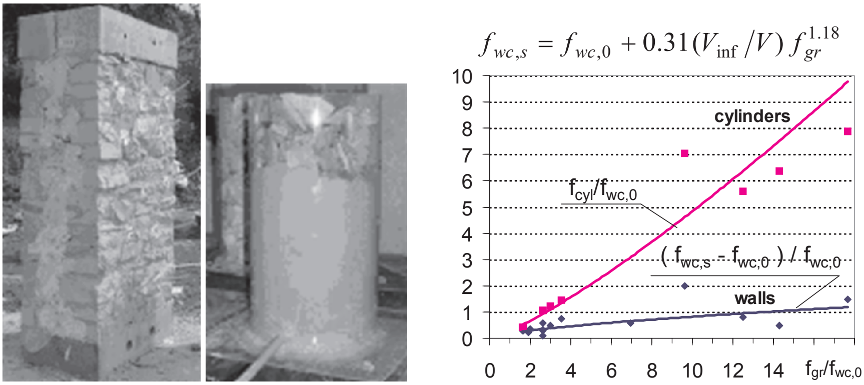
compatibility than jacketing (Modena et al., 1998a). Most specific problems related to the injections are connected to the uncertainties of the masonry (features and composition of the section, size and distribution of voids, absorption of the mortar, etc.), especially in the case of rubble stone multi-leaf walls. Therefore, the correct application of the intervention requires: ((i) the investigation of the wall section in order to check its injectability, (ii) the selection of grouts with appropriate properties and adequate compatibility (chemical, physical and mechanical) with the original masonry, (iii) the proper procedure of injection itself (choice of the number and distribution of the injection holes, injection pressure, times of execution), (iv) the control of the effectiveness of the consolidation by using specific investigation techniques before, during and after the intervention (Berra et al., 1992; Binda et al., 1993; Valluzzi et al., 2003; da Porto et al., 2003). A comprehensive research conducted at the University of Padova demonstrated that low-strength grouts (e.g. natural hydraulic-lime based) can provide a full compatibility (chemical, physical and mechanical) with the original materials; moreover, they allow making more reliable analytical formulations for the prediction of the mechanical strength of the injected walls (Valluzzi et al., 2004) (Figure 12).

A significant result was that the use of high strength grouts (compared to the original strength of the walls) can influence the strength of the injected infill, but it is not able to increase with the same contribution the final strength of the wall (see Figure 12, where symbols are as follows: $f_{wc,s}$ and $f_{wc,0}$ are the compressive strength of the original and of the injected wall respectively, f_{gr} and f_{cyl} are the compressive strength of the grout and of the injected cylinders, which is assumed comparable to the strength of the consolidated infill; V_{inf} is the volume of the core). For real cases, prediction of strength can be approximated by assuming $f_{wc,0}$ from a double flat jack test and by simply measuring dimensions of walls and the compressive strength of the grout (taken as reference strength for the injection) by a common laboratory test.

Finally, to verify the efficiency of the techniques, the use of non destructive methods, as sonic pulse velocity and radar tests, can be successful, especially if in combination with inspections and endoscopies. An example of application of the sonic tomography on a vertical section of the curtain walls of Cittadella (Padova, Italy) before and after injections is given in Figure 13.

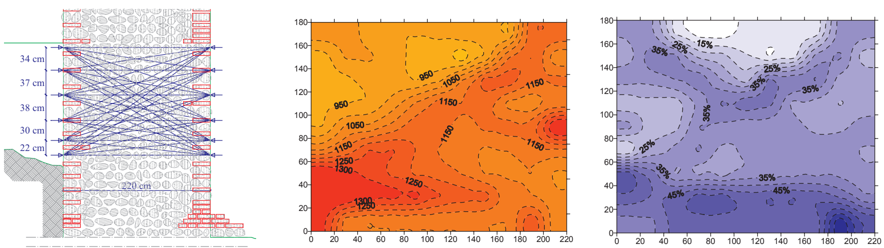
To improve the strength of the walls, deteriorated mortars can be replaced by selected proper materials with the deep repointing technique (involving at least 6-7 cm of the thickness) as it is shown in Figure 14. Combination of different techniques may be also adopted aiming to increase the performance of walls suffering various structural problems (Valluzzi et al. 2004).

Application of FRP laminates can improve the shear strength of the walls or consolidate arches and vaults (Figure 15) (Valluzzi et al., 2002; 2001). It requires regular surfaces (in order to reduce thickness of substrate at the interface) thus it is more suitable for brick walls. Some experimental results carried out at the University of Padova demonstrated the high effectiveness of



(a) (b)

Figure 12. Experimental wall ready to be injected and preliminary check of injectability (a); comparison between cylinders and wall increment of compressive strength, and analytical model for strength prediction after injection (b). The relation is reliable for low-strength grouts, e.g. for ratio $f_{gr}/f_{wc,0}$ not exceeding 4.



(a) (b) (c)

Figure 13. Results of the sonic tomography: (a) drawing of the wall section, map of velocity after injections (b) and their variation compared to the original condition (c) (darker colours corresponds to higher velocities).

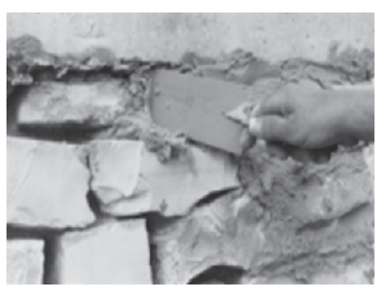
 (a)  (b)

Figure 14. Deep repointing executed in an experimental wall: mechanical excavation of the joints (a) and phase of filling of the joints with new mortar (b) (Valluzzi et al. 2004).

medium-strength (and less expensive) products (e.g. glass fibers, instead of carbon). In particular, the use of three types of fibers (carbon, glass and polyvinyl-alcohol, labelled as CFRP, GFRP and PVA in the following) applied in different configurations (at one side or both sides) and geometry (as a squared grid or diagonally) on the faces of a total of 24 walls was considered (Valluzzi et al. 2002). Failure of the plain panels was due to brittle splitting (ultimate load of 101 kN) (Figure 15a). Splitting failure with a clear diagonal crack was also obtained for all single-side reinforced panels, whereas the ultimate load was in many cases lower than the reference one; their average value are: 91 kN for CFRP, 96 kN for GFRP and 101 kN for PVA for the grid arrangement, 116 kN for CFRP and 112 kN for GFRP for the diagonal one. The samples exhibited a clear bending deformation during the loading phases along the unstrengthened diagonal, due to the eccentricity of the reinforcement (Figures 15(b,c)). Among the on-side reinforced specimens, the diagonal strengthening configuration always revealed a higher effectiveness in comparison with the corresponding squared grid set up.

In the case of double-side strengthening the failure mechanism changed to the loss of collaboration between the reinforcement and the substrate, due to the detachment (peeling) of the superficial layer of the masonry (see Figures 15 (d,e)), particularly serious for the CFRP-reinforced panels, or the rupture of the FRP strips along the diagonal itself. Despite the lower strength, the advantages of grid reinforcement distribution was confirmed by the more spread crack patterns (see Figure 15f), whereas a clear splitting crack appeared in all the diagonally reinforced panels. The ultimate strength increase was anyway noticeable in almost all cases: 104 kN for CFRP, 115 kN for GFRP and 148 kN for PVA for the grid arrangement, 146 kN for CFRP and 175 kN for GFRP for the diagonal one.

The abovementioned behavior is also in agreement with numerical simulations, as shown in Figure 16. GFRP strengthening applied on both sides gave the best behavior, both in terms of strength and failure mode: the crack spreading provided sufficient signals of crisis far before collapse, indicating a less brittle global behavior. It is worth to remark that the use of those innovative materials should be particularly cautious in historical heritage, as incompatibility and durability problems can not be disregarded in such cases. Experimental validations are therefore fundamental in order to prevent possible alterations or damage.

3. STRENGTHENING OF MASSIVE MASONRY STRUCTURES SUBJECTED TO CREEP

Brick-masonry load bearing elements of heavy historic structures as towers, heavily loaded pillars, and large masonry walls (i.e. curtains), frequently exhibit very typical mechanical deterioration phenomena like:

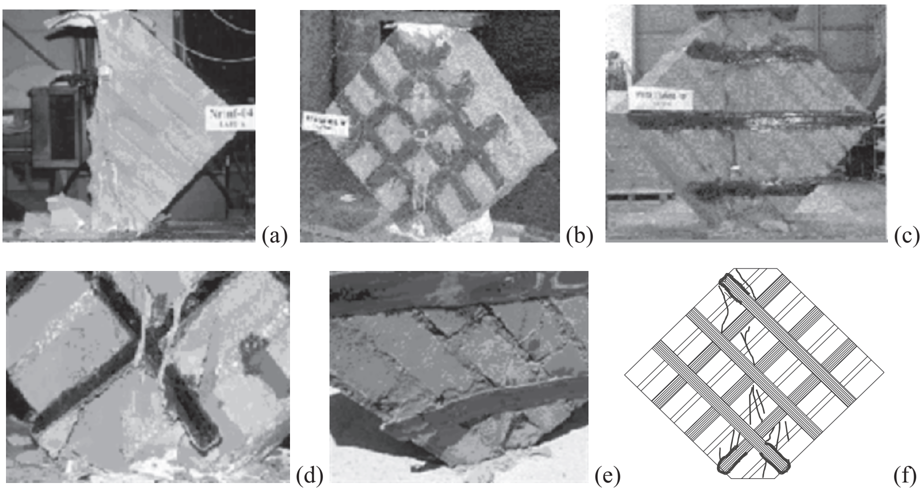


Figure 15. Mode of failure of the panels under diagonal loads: (a) reference panel (unstrengthened); (b) (c) one-side strengthened panels: (b) grid pattern, (c) diagonal pattern; double-side strengthened panels: detail of the peeling in the anchoring zone on grid (d) and diagonal (e) pattern; (f) crack distribution on grid pattern.

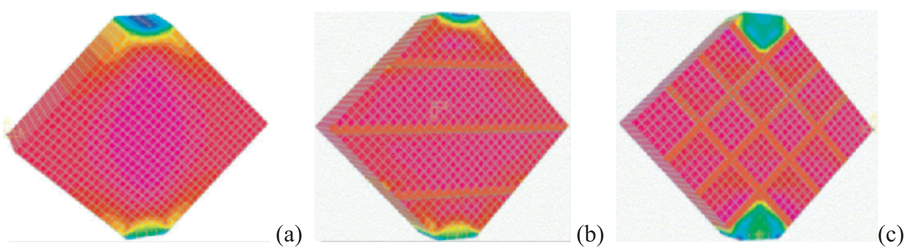


Figure 16. FEM simulation of distribution of stresses in the tested panels: (a) unreinforced sample (highest level of splitting stresses are concentrated in the core); (b) diagonal pattern reinforcement (tensile stresses spread from the core); (c) grid pattern reinforcement (stress peaks are shifted to anchoring areas).

- formation of vertical or sub-vertical, thin but very diffused cracks,
- b) more or less local detachment of the outer leaf in multiple leaf walls (Binda and Anzani, 1993; Anzani et al., 2000).

Such a particular crack pattern is often not attributable to common causes of damage like seismic events, foundation settlements, instantaneous increase of external loads (e.g. for added storey or building changing) or to chemical, physical and mechanical degradation of the basic materials. On the contrary, it is due to the prevalent effect of the dead load and the related time dependent phenomena, often combined with cyclic loads, like wind actions, thermal and hygroscopic excursions, or bell ring oscillations (in bell-towers) (Binda et al., 1991; 1992). This damage, generally disregarded in those structures, in comparison with other more evident critical conditions (large cracks, out-of-plumb, relevant deformations, etc.), can induce sudden and unexpected brittle collapse, as observed in several cases (e.g. the Civic Tower of Pavia, the Bell Tower of San Marco in Venice and the Cathedral of Noto in Italy; the Bell Tower of St. Magdalena in Goch in Germany) (Figure 17) (Binda et al., 1992; 2001), even for stress values lower (40% to 60%) than the strength of the masonry under short term static loads.

In the last years, several experimental campaigns jointly performed by the University of Padova and the Polytechnic of Milan, demonstrated that the creep damage can be effectively counteracted by minimal reinforcement of the superficial layer of the masonry, properly inserted into the bed joints. The "bed joint reinforcement" technique is executed by removing the mortar in the bed joint for few centimeters (about 5-7), by placing the reinforcement (small bars or plates) and by the final filling of the cut with a proper embedding material (Figure 18) (Binda et al., 1999a; Valluzzi et al., 2004a). Both-sides application is more effective, but also in case of single-face accessibility or large-section walls, the insertion of transverse pins in holes sealed with grouts are recommended. Moreover, the use of more compatible mortars (hydraulic lime-based) as embedding material can prevent incompatibility with original materials. The technique does not require particular skills and

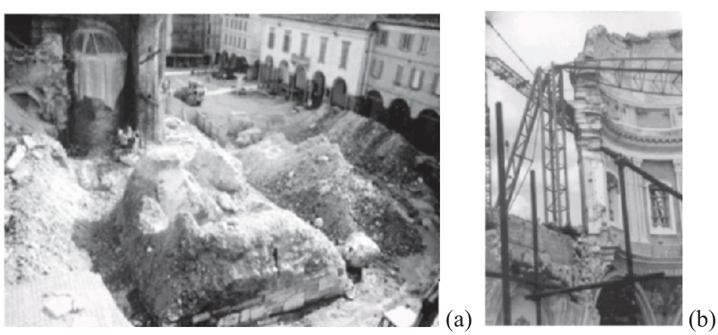


Figure 17. Collapse of the Civic Tower of Pavia (1989) (a) and of the Cathedral of Noto (Siracusa, 1996) (b), Italy.

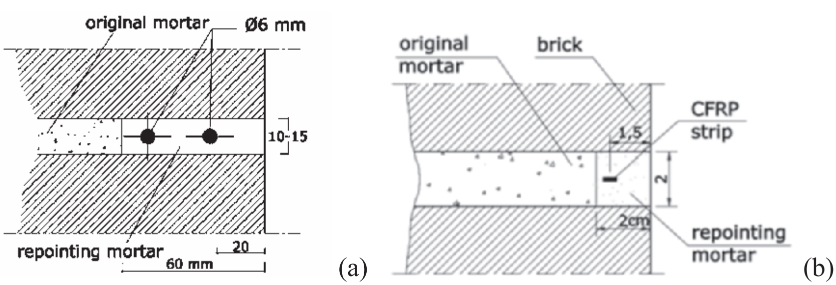
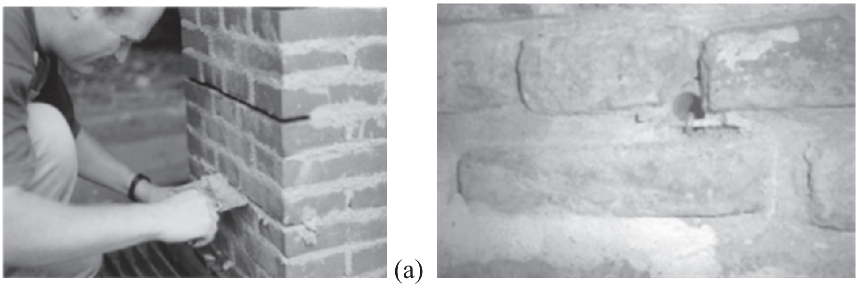


Figure 18. Sketch of the bed reinforcement technique with steel bars (a) and thin FRP strips (b).



(a) (b)

Figure 19. Bed joint reinforcement technique: (a) detail of embedding phase in strengthened joint; (b) arrangement of bar and transverse tie into an excavated joint before final sealing of hole.

skills and tools during application; some care is required in some operative phases (cutting of the bed joint, cleaning, repointing) but it can be performed quite easily and quickly (Figure 19).

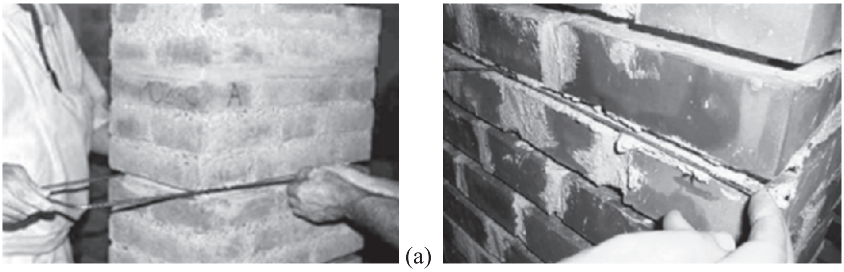
First experimental investigations (Modena et al., 2002; Valluzzi et al., 2004a) dealing with applications of low diameter (5 mm or 6 mm) stainless steel reinforced bars, showed the efficiency of the intervention in reducing the transverse dilation due to high compression stresses, as the bars bear the tensile stresses otherwise directed to the bricks. Therefore, even if a signify-cant increment in strength is not detectable, the technique is able to act directly on the limitation of the cracks development, thus improving the performance and the safety of the structure.

Further investigations were focused on identifying the possible application of FRP materials, aimed at checking, by laboratory tests, both advantages and disadvantages in their use, before any possible in-situ application. Monotonic, cyclic and creep simulating tests were performed first on panels strengthened with circular section carbon bars (CFRP) (5 mm in diameter) (Valluzzi et al., 2003) and then, recently, with thin rectangular section CFRP strips (1.5x5 mm) (Figure 21) (Saisi et al., 2004; Valluzzi et al., 2005a).

The advantages of the use of carbon fibers instead of steel reinforcement are mainly related to their complete corrosion immunity, but many aspects still need to be deeply investigated. Despite their high strength, FRP are very brittle and inductile to bending and folding (e.g. for anchorage); they are sensitive to high temperatures and constitutive laws able to describe the interface behaviour among reinforcement-mortar-brick are not comprehensively known in masonry yet.

Thin strips have more advantages in comparison with circular bars, due to their higher flexibility to the unevenness of the joints and the better behaviour against splitting failures, which lead to possible more superficial and less obtrusive interventions (see Figures 18 and 20).

Figure 21 shows the vertical and the horizontal deformation vs. stress diagrams of the creep tests on experimental panels. It can be noticed that the reinforcement has not great influence on the strength and the vertical deform-



(a) (b)

Figure 20. Example of panels strengthened with different reinforcements: (a) steel bars; (b) thin CFRP strips.

ations, whereas a great role is attributed to the control of the horizontal deformations. In particular, the reinforcement was able to reach the tertiary creep conditions at deformations around the 70% of the original case; moreover, in such ultimate phase, the prisms reinforced by the technique D, showed also an increment of about the 25% in the strength.

It is worth to say that, despite a number of researches and efforts devoted to FRP materials, until now real applications have been only performed with stainless steel bars in combination with lime-based repointing mortars, as they can guarantee effectiveness and durability, even in exceptional conditions not easy to predict.

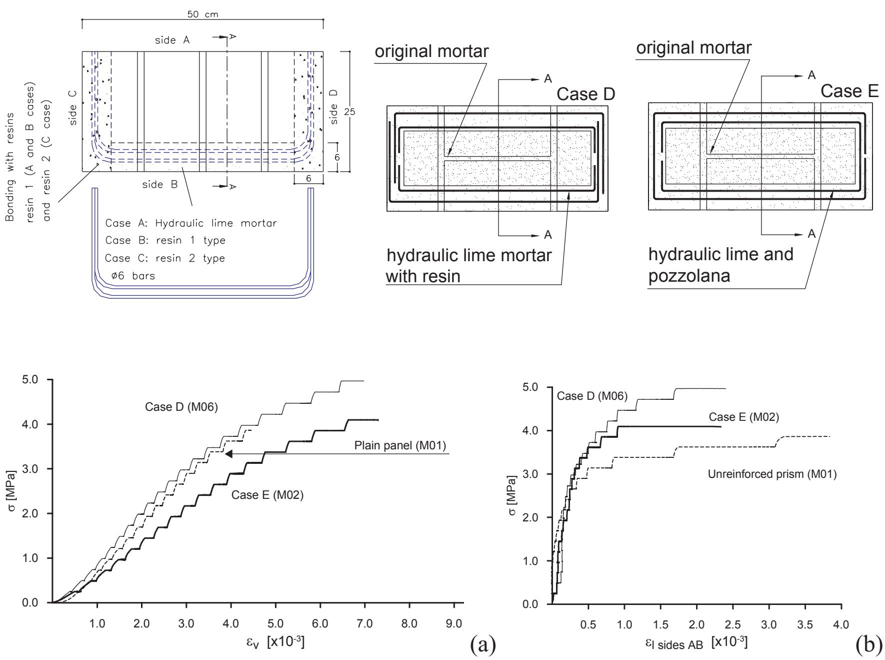


Figure 21. Draws of the laboratory samples (sections) and creep test results: vertical (a) and horizontal (b) deformations of the panels.

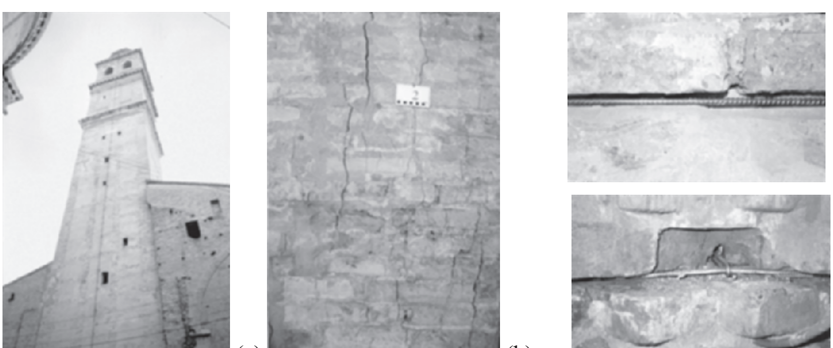
3.1 Interventions on case studies

The experimental validation of the performance of the bed reinforcement technique allowed proposing it for the consolidation of several historic constructions and monuments at risk in Italy, in order to counteract the dilation under high compressive stresses by improving the material toughness. It is worth to remark that, in real cases, more damage typologies can coexist, thus the proper selection and combination of different intervention techniques should be considered. In many cases, both injections and partial and limited rebuilding (the traditional "scuci-cuci") are used, to reduce the stress concentration and to replace the most damaged resistant parts, respectively. These techniques act locally by improving the mechanical behaviour of the material, which contributes to the rehabilitation of the proper load bearing capacity of the structure. In the following, some representative case studies of towers and structural components of churches in Italy are depicted (see Figures 22-24).

As suggested by the laboratory experience, both in repair and strengthening conditions, the bed joints reinforcement technique is easily and quickly performed; moreover, aesthetic aspects of the façade can be improved or maintained, depending on the specific conservation requirements.

4. CONCLUSIONS

Different studies on the application of repair and strengthening techniques on historic masonry structures in severe loading conditions have been presented. In seismic areas, the rehabilitation of masonry building has to be performed first at global level, by favouring the collaboration of the structural components (walls, floors and roof). Tying and the use of compatible tech-



(a) (b) (c)

Figure 22. St. Giustina bell tower (Padova): general view (a), typical creep crack (b), application of the technique (c).

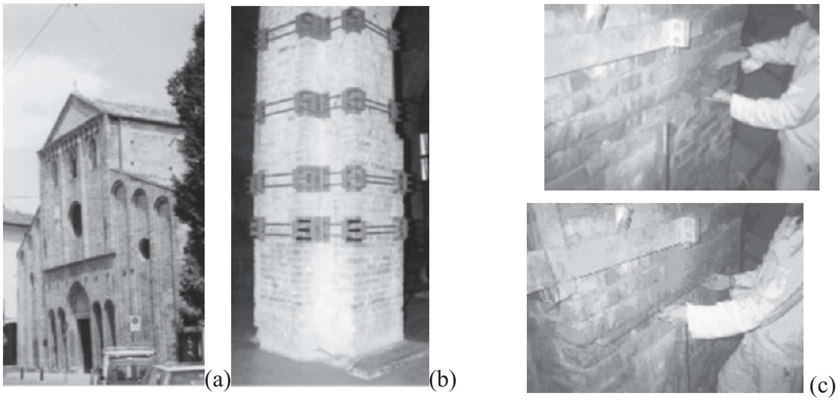


Figure 23. St. Sofia church pillars (Padova): general view of the church (a), provisional measures on a cracked pillar (b), application of the technique (c).

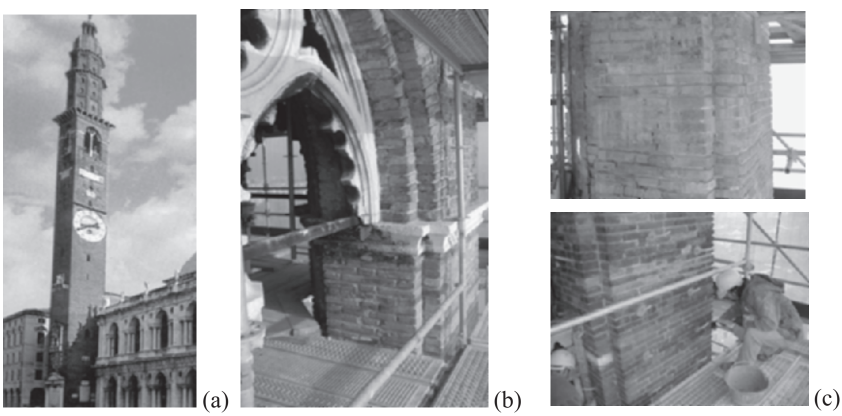


Figure 24. Civic tower of Vicenza: general view (a), cracked pillar of belfry (b), joints strengthened by structural repointing and combination with injections (c).

niques for the stiffening of floors and roof can represent a solution of minimum intervention, in order to preserve as much as possible the historical identity of cultural heritage. Strengthening of walls by injections and repointing requires a proper design of grouts, with particular attention to compatible binders. In particular, for grout injections, injectability on site and laboratory tests are recommended. The importance of the design and of the site control of the intervention has been pointed out for all the mentioned techniques, as well as the necessity of using appropriate preliminary diagnostic procedures for the effectiveness of the intervention.

The progressive damaging of masonry structures subjected to high long term compressive loads can be effectively counteracted by the insertion of small diameter bars into the bed mortar joints. Laboratory experiments simulating both monotonic and creep loads showed a significant reduction in the horizontal dilation and a consequent minor diffusion of the cracks.

FRP materials should be considered with caution in such ambit, but have a great potential in improving the mechanical strength of masonry, provided that experimental validation is available before real applications.

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