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Steel solutions for the seismic retrofit and upgrade of existing constructions

Constructive and performance analysis of the retrofit systems for vertical masonry elements

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Summary

This document is part of the project STEELRETRO, Task 2.2. The aim of the task was to collect, and briefly describe the most common earthquake rehabilitation techniques used for masonry buildings, with special focus on vertical structural elements. In the document, methods are classified in: traditional techniques and techniques based on Fiber Reinforced Polymers (or FRP's). A separate section is dedicated to a brief presentation of the possibilities provided by advanced techniques.
The presentation format is: to briefly introduce each rehabilitation method, to highlight the advantages and disadvantages, and to present the possible performance improvements. The main performance improvements are: increase of strength, ductility and/or energy dissipation. Other, method specific, improvement mechanisms are also described.
At the end a hierarchical classification of the methodologies is attempted in terms of structural performance improvement and economic advantages.

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Preface

The report is part of the project STEELRETRO, Task2.2. The global aim of the STEELRETRO project is to *“set up steel solutions for the seismic retrofit of existing buildings, furnishing design and construction methodologies, tools for dimensioning of elements and connections as well as for cost estimation”*.

Specifically, Task 2.2 aims at supplying a *“construction and performance analysis of the retrofit or upgrading systems (not necessarily in steel) for existing masonry and reinforced concrete buildings actually adopted in the European Counties in seismic areas in order to evaluate structural performance actually achieved in common practice”*.

The aim of this document is to supply data on the *“construction and performance analysis of the retrofit or upgrading systems for vertical masonry elements”* adopted in seismic areas.

Espoo, 31.1.2011

Ludovic Fulop, Merja Sippola

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1 Introduction

Old masonry buildings represent the overwhelming part of the building heritage of Europe. Even if some of these buildings are maintained only for their historical importance, the large majority is still in use and fulfills everyday functions as dwelling or office buildings. Therefore, preserving these buildings has an important practical dimension, together with the natural interest of preserving the historical heritage.

Besides slow degradation, and together with it, one of the major concerns in case of masonry buildings is seismic vulnerability, which has been revealed in even moderate earthquakes in Europe in the recent past [1]. This poor performance is primarily attributed to the frequent disregard of simple design rules [1], but more fundamental causes can not also be overlooked. These are:

- The degradation of the performances of the masonry in time. Degradation of masonry is very slow compared to other construction materials, especially if the building is protected from water. But, masonry buildings are usually of considerable age, and even at this slow rate, the degradation is significant. Furthermore, most of the masonry buildings experienced damages, or were neglected, during periods of economic hardship.

- Due to the long period of service many of the masonry buildings were subsequently modified by their owners, in order to fulfill changing occupancy requirements. These modifications were rarely made based on any design, the structural safety of the building being taken as granted. Most of the times modifications are not documented, and are reveled only during major renovations.

- Masonry buildings were not designed for earthquake requirements. Except for a few isolated cases (e.g. Pombalino buildings in Lisbon), at the time of construction such requirements did not exist, and knowledge of earthquake design was completly lacking.

- Masonry, especially Un-Reinforced Masonry (URM), has inherent physical properties which makes the building vulnerable to earthquake loading. Large mass, reduced ductility and tensile-strength are among the disadvantages of masonry.

2 Goal

The main goal of this report is to present a summary of the masonry rehabilitation techniques applied in earthquake regions. The document tries to give an overview of the possible rehabilitation methods, their fields of application, their performance and economical feasibility. A hierarchical classification of the techniques is attempted, based on these parameters.

3 Limitations

This report does not intend to be a complete overview of the research field in each rehabilitation technique. A large number of publications have been reviewed for each technique but, in order to limit the length and complexity of this document, only a synthesis of the main tendencies, and conclusions of each field are reported. Major dissenting opinions are also mentioned when applicable.

4 Masonry typologies

Masonry is one of the most ancient ways to build. The blocks, the mortar, the construction techniques were always adapted by local masons, depending on the historical age and place. Locally available materials, local experience, had a major influence on the local masonry structures. Even if at the end of the last century some standardization can be observed, most of the older masonry buildings are very characteristic to their location. The most common masonry wall typologies are:

- Single leaf walls;
- Cavity walls with rubble filled core;
- Bonded brickwork walls;
- Stone masonry walls;
- Walls made of lightweight Concrete Masonry Units (CMU);
- Concrete block walls;

More special masonry wall configurations:

- Pombalino walls - Timber and masonry combination developed after the 1755 Lisbon earthquake
- Versions of Roman masonry walls clad in the most varied configurations (e.g. opus reticulatum, opus testaceum, opus mixtum, opus craticium, opus quadratum, etc).
- Tuff masonry constructions – Typical to earthquake locations in Southern Italy, Turkey, Armenia etc. [2]

The blocks commonly used in masonry walls can be: (a) adobe, (b) solid clay brick, (c) cellular clay brick, (d) hollow clay brick, (e) perforated clay bricks, (f) hollow concrete blocks, (g) cellular concrete blocks, (h) autoclaved cellular concrete blocks, (j) stone blocks (with blocks in different processing state), etc.

The mortar used in masonry construction is as varied as the blocks themselves. In new constructions the most common mortars are: Portland Cement (PC) mortar, calcium mortar, or mixed calcium and cement mortars. But most of the walls can be constructed without the use of mortar altogether. Mechanical fixings between the construction blocks, usually made of metal, are also not uncommon. These fixings are present in masonry from the earliest times to the present.

5 Rehabilitation of masonry buildings

Due to large variety of the masonry typologies, rehabilitation techniques are also numerous. Some of the techniques were developed for a single typology of masonry; some can be applied for many configurations. The aim of rehabilitation work can be:

- ***Reparation*** – Usually involves only cosmetic repair of the wall, or of the finishing. Does not improve the structural performance of the masonry, further it can even be disadvantageous, because visible faults (e.g. cracks) are hidden, giving the impression of no problems.

- ***Restoration*** – Implies the remediation of existing damage to a masonry element. The aim is to restore the previous state of the masonry in terms of structural performance (i.e. strength and stiffness). Restoration usually occurs after some loading condition damaged the masonry, but it is judged that the behaviors of the wall was satisfactory in the given, usually exceptional, condition. The condition which causes the damage can be: neglect, water penetration, accident, explosion, earthquake etc.

- ***Strengthening*** – It is undertaken when it is believed that a masonry element would not perform well under loads that are expected (e.g. earthquakes). Strengthening is often performed in combination with restoration after damage occurred, for instance after damaging earthquakes.

If walls have to be rehabilitated, the decision has to be taken as to what the aim of the work is. Cosmetic reparations do not improve structural performance, while restoration and strengthening does. Furthermore, reparations can hide information concerning the degree of structural degradation.

Any rehabilitation work has to start from the assessment of the entire structure. Improving the performance of vertical load bearing masonry elements may make little sense if the structure is not well configured, 3D interaction of elements is not ensured, or the uniform distribution of earthquake loads is not provided by effective diaphragm action of the floors.

The Italian experience of recent earthquakes pointed to a characterization of failure modes of masonry in two distinct categories [1]:

- ***First stage failure mechanisms*** are usually local, caused by out of plane failure of walls, insufficient anchoring, tying or deficient diaphragm. This first mode may, or may not lead to the development of global collapse mechanisms. These failure modes can be studied on sub-models, without the need for complete modeling of the building.

- ***The second stage failure mechanisms*** involve the whole structure, and they usually develop when diaphragm action assures the transmitting of forces to the masonry shear walls. Full structural modeling (e.g. elastic spectral, non-linear pushover, time-history analysis etc.) of the building is required for the assessment of these failure mechanisms.

In new masonry buildings, by respecting the constructive rules of the design codes, it is supposed that first stage failure mechanisms are avoided. Therefore, design checks refer to mainly second stage modes.

The following questions should be in the focus of the initial structural assessment:

- What is the basic type of the structure?
- What are the main paths of stress flow in the structure caused by the inertial forces generated by an earthquake?
- What is the condition of the structure and its components?
- Will the structure maintain its stability and integrity as a ‘3D, well-anchored multi-material frame’ subject to these loads?
- What are the weakest points in this structural chain
- Is there hazardous architectural design – un-symmetry of the building, lack of global stability, weak floors, wrongly spaced or too large or too many openings, weak terraces, too large difference in stiffness of adjacent parts etc?
- Is there hazardous engineering design - weak materials, bad or lacking connections between elements (walls and floors, walls and roof, two adjacent walls, columns/walls and foundation, wall leaves), too weak pears of spandrels etc. The principle must be to ensure the integrity of the 3D frame of the whole building by strengthening the weakest parts and adding/strengthening connections. Only after this evaluation of the whole structure and identification of the weakest links it is time to look for methods for strengthening these components (walls, columns, joints, foundations, floors, roof, terraces, chimneys etc).

The target of rehabilitation work, in case of seismic rehabilitation, is usually to ensure that the structure would withstand a given earthquake level. In principle, this target can be achieved by improving response parameters like:

- Increasing strength, and sometimes ductility, of connections between building elements in order to ensure effective 3D interaction;
- Increase floor diaphragm strength and stiffness in order to ensure uniform transmission of horizontal forces to vertical elements.

- Increasing the load bearing capacity (strength) of the vertical structural elements;
- Increasing ductility;
- Improving energy dissipation capacity;
- Increasing damping;
- Modifying the stiffness, and hence changing the period of vibration, in order to reduce the earthquake's input.

Usually, by any intervention, a number of the above parameters will be affected. It is very improbable that, for example, the stiffness of a component can be changed without affecting the strength too much, or vice versa.

Masonry rehabilitation techniques for vertical elements, most often, have the main goal of increasing the strength of elements or connections. However, it is important to remember that the intervention should improve, or at least not affect, the ductility and energy dissipation of the structure. For instance, by strengthening a masonry element, not only the strength will increase, but the entire failure mechanism of the element and/or of the entire structure may change. If the failure mode changes from a ductile one to a fragile one, the designer has to think if the gain of strength justifies the intervention at all.

6 Principles of strengthening vertical masonry components

6.1 Strengthening of masonry walls

6.1.1 Mechanisms of wall failure

In this stage it is very important for the designer to have a clear picture concerning the expected failure mode of the masonry element. Increasing the strength implies strengthening the locations/parts which trigger this failure mode. However, by inserting strengthening elements to prohibit a certain failure mode, the designer forces the wall to fail in a different mode. Not only the strength of the wall increases, but the mode of failure may also change.

There are two main concerns with masonry walls in earthquake:

- ***Out-of plane failure*** due to excessive horizontal forces. This scenario is most often seen when the diaphragms of the building are not effective and the wall is acting like a standing cantilever with no connection at the top. Long, tall and slender walls are especially vulnerable. Even in massive, historical buildings the self-weight of the wall can generate enough inertial force, in case of earthquake, so that the wall panel fails out of plane. The most important counter measure for avoiding out-of-plane failure is to ensure effective diaphragm action at the floor levels.

- ***In-plane failure*** of masonry walls occurs when the diaphragms are effective in transmitting the horizontal forces to the walls placed parallel to the horizontal force. In this case, the walls may fail in-plane. As in-plane failure only occurs in the well configured buildings, with good 3D interaction between elements, in this case, the strengthening of the walls, to resist horizontal forces, has to be the main focus of the rehabilitation.

Because out-of plane failure is clearly related to bad 3D configuration, the rehabilitation techniques of masonry against in-plane failure are the main focus of this document. This does in no way suggests that out-of plane failure is not important! It is just to say that, most of the time out-of plane weakness of masonry can be corrected by improving 3D configuration rather than strengthening the masonry wall itself.

Furthermore, even for in-plane failure, strengthening solely the wall can have no effect. The design has to consider the fixing of wall to its surroundings, especially anchoring to the foundation and fixing to the floor. For a well anchored Un-Reinforced Masonry (URM) wall three main in-plane failure modes exist:

- ***Sliding Shear Failure*** (Figure 1.a) usually occurs in one of the lowest courses of the wall. It is typical failure mode for walls with aspect ratios of 1:1 or lower, and small vertical loads. It is also a non-ductile failure mode, so not very advantageous in case of earthquake.

- ***Diagonal Shear failure*** (Figure 1.b) is the most common failure mode of URM. It occurs in walls with aspect ratio 1:1 or larger, in the presence of significant vertical loads. The failure happens due to the action of the principle tensile stresses in the inclined plane. Cracks can develop in the mortar, or cracking can involve the masonry blocks, depending on the strength ratio of the mortar and blocks. It is a non-ductile failure mode.

- ***Bending failure or Rocking*** (Figure 1.c) occurs in slender wall segments with aspect ratio of about H/L=2:1. In the early stage it consists of the opening of a course at the bottom of the wall, due to tensile stresses. As the opening advances, the compressive stresses increase on the opposite end of the wall, and the failure of the wall is usually due to crushing of the masonry in the compressed region. This failure mode usually involves large inelastic deformations, without the decrease of the load bearing capacity of the wall, and is advantageous in case of earthquake loads.

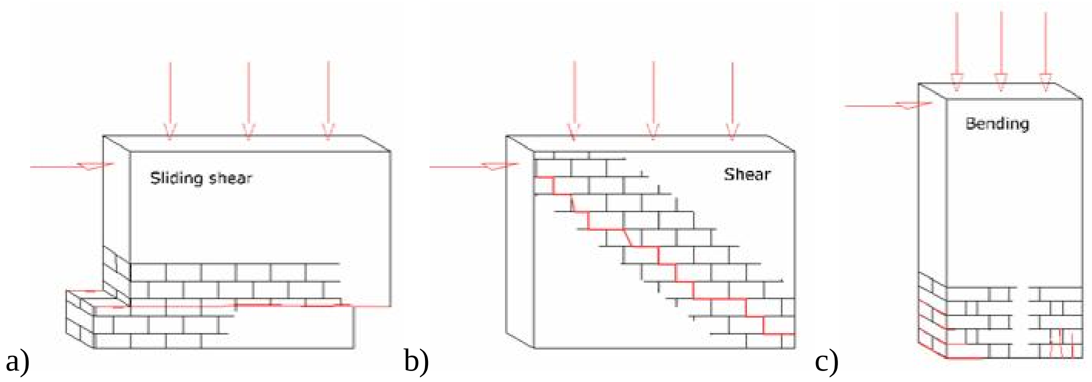


Figure 1. Failure modes of masonry walls [3]

6.1.2 Goals of in-plane wall strengthening

As it is described above, failure of URM in earthquake can be driven by different failure modes. The rehabilitation techniques strive to increase the strength of the masonry for all or some of these failure modes. For the rehabilitation techniques reviewed in this document the goal, in terms targeted failure mechanism, is described. A summary table (Table 5) is also presented noting which rehabilitation technique is effective against which failure mode.

For instance, the presence of cracks greatly reduces the shear and tensile strength between the two parts of masonry. Even minor cracks greatly affect the strength. Two main methodologies aim to restore/improve the bond between the cracked parts; *injection techniques* (e.g. using epoxy resign) try to restore the bond between the two surfaces directly in order to provide shear and tensile strength; while *bandaging techniques* (e.g. by FRP mesh) try to circumvent the original/damaged stress path, and supply an alternative path for stress transmission. But fundamentally, both methods aim for the improvement of shear and tensile strength over the cracks.

Injection techniques can improve the strength in sliding shear and shear. They are not so effective for bending because the bonding can not significantly increase the tensile strength of the masonry.

Bandaging techniques can improve the shear and sliding-shear strength, but they can also be effective in bending because FRP fibers can act in the tension zone of the wall and even the compressive strength of the masonry can be improved via confinement. Confined masonry not only has superior compressive strength, compared to unconfined masonry, but some ductility is also provided by the confinement.

If the aspect ratio of the wall is small (i.e. the height of the wall is small compared to the length), the wall will always fail in shear, and rocking behavior cannot be obtained. Then the goal of the strengthening is to increase shear capacity and add ductility and energy dissipation. To avoid collapse of the structure the cracking should be distributed to as large area as possible. Usually, failure involving a significant part of the wall is superior to a localized failure in two ways (Figure 2):

- When failure is distributed to almost the entire wall, the wall mobilized all its strength in resisting the force. In case of a localized failure, one part/component of the load resisting system is weak while the other parts still have some reserve strength which can not be activated.

- An evenly distributed failure usually places less demand, e.g. in terms of plastic deformation, on a particular region in the wall. If the deformation demand is more evenly distributed, the wall can withstand more overall deformation, and supply more ductility and energy dissipation.

Therefore, failure modes involving large parts of the wall are desirable in earthquake loading scenarios.

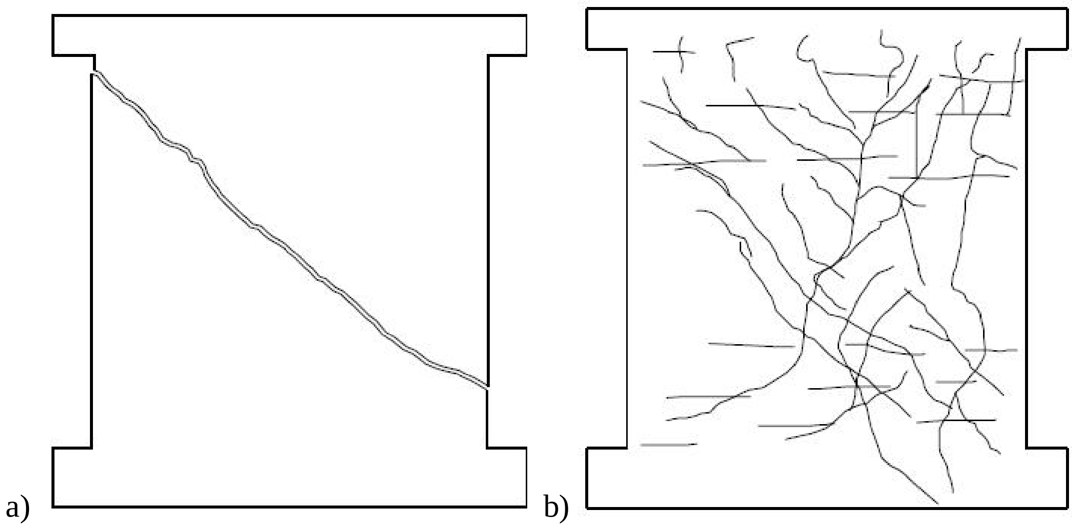


Figure 2. (a) Brittle-shear failure and (b) ductile shear failure of wall panel [7]

If the aspect ratio is large (i.e. the wall is very high compared to its length), rocking failure cannot be avoided. So the strengthening should concentrate on strengthening the toes against crushing, the uplifted side against tensile stresses, plus some shear strengthening in the middle, if the masonry has low tensile capacity and the aspect ratio is close to moderate. Rocking is an advantageous failure mode, as it provides large deformation capacity and self-centering (i.e. at the end of the shaking remnant displacement is 0). However, strengthening the uplifted side against tensile stresses (e.g. with FRP overlay) has little sense if uplift from the foundation is not cared for with proper anchoring. Also, the state of the foundation is crucial here, in order to be able to transmit the required anchor forces.

In the area between the extremes (aspect ratio 1:1) there is no obvious agreement, in the research community, whether the goal of the strengthening should be to move the failure mode towards rocking and strengthen the toes, or to increase both the shear capacity and the in plane bending capacity and try to distribute the cracking to as large area as possible. It was suggested [4] that strengthening should be so that shear cracking is delayed, allowing for the development of horizontal flexural cracks. This can be achieved, for example, by placement of the reinforcing fibers (e.g. FRP) horizontally.

For walls failing in flexure, the limiting element concerning strength is usually the crushing of the toe on the compressed side of the wall. Crushing strength can be locally increased by inserting steel plates between horizontal brick layers, or by other local confinement of the masonry. In the study of Priestley *et al.* [4] the shear strength of a five-storey reinforced masonry building tested to failure was restored by full CFRP overlays of the two lower stories plus reconstruction of the crushed wall toes with polymer concrete. The inelastic deformation capacity was doubled compared to the as-built structure.

7 Masonry strengthening techniques

7.1 Walls - Traditional strengthening

7.1.1 Surface treatment

7.1.1.1 Ferrocement

Ferrocement consists of a closely spaced, fine rod mesh (see Figure 3). The mesh can be made of metallic material, but also of other fibers. The mesh is fixed to the surface of the wall, and covered in high strength cement-mortar (10-50mm), achieving a reinforcement ratio of 3-8% [5]. Prefabricated Ferrocement walls are used for constructing entire buildings, but also for strengthening existing masonry.

Ferrocement improves the performance of the masonry wall by providing tensile strength to bridge over developing cracks and by confining the masonry. Both in-plane and out of plane strength of the wall are increased. Experimental investigation by Abrams & Lynch, shows that in plane strength can be increased by 1.5 times, using Ferrocement strengthening [5].

a)  b) 

Figure 3. (a) Ferrocement mesh [5] and (b) example of rehabilitation [6]

7.1.1.2 Shotcrete

Shotcrete is essentially an overlay of concrete sprayed on the surface of the masonry wall, over a mesh of reinforcing bars Figure 4. The procedure of spraying is less costly than jacketing and pouring concrete, and therefore it is preferred in rehabilitation work. The thickness of the resulting cover can be adapted to the strength requirement, but the working thickness is usually more than 60mm. The reinforcing mesh is usually made of welded bars designed to achieve effective crack control.

One of the difficulties of strengthening with shotcrete is achieving effective bonding between the existing masonry and the newly placed shotcrete layer. Steel dowels fixed with cement grout or epoxy to pre-drilled holes in the wall were often used with the intent to improve bonding [6]. However, experiments by Khan [12] have shown that dowels did not improve the composite panel's response.

It was reported that a common opinion, among practitioners, is that bonding would substantially be improved by painting the masonry surface with epoxy before the shotcrete is applied [5]. Some suggested that wetting of the masonry wall would in improve bonding, but it has been reported that such procedure does not affect the cracking or ultimate load [5].

a) 

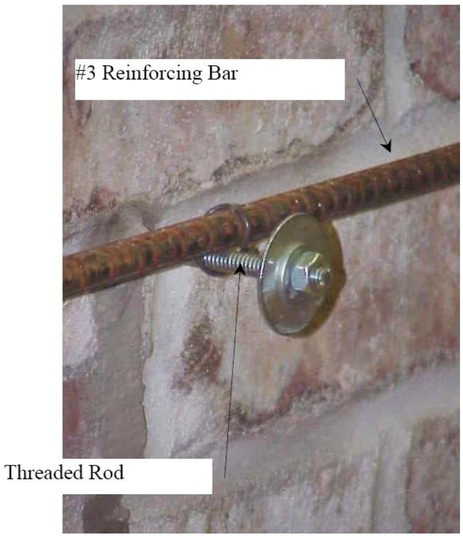
b) 

Figure 4. (a) Spraying shotcrete on masonry wall [5], (b) dowel to improve bonding [6]

Very substantial increase of the racking strength can be achieved by shotcreteing. Because, in essence, a new load bearing structure is formed on the surface of the masonry wall, this increase can be several times the initial strength of the wall. The factor of increase of 3 to 25 is reported in [5]. Based on experiment on masonry piers it was reported that "the shotcrete rehabilitated specimen behaved as a reinforced concrete pier with no evidence of composite action with the masonry" [6], proving that this method is about effectively building a new RC wall on the surface of the masonry. This approach is also reflected by the evaluation methods of the strength, which neglects completely the strength contribution of the masonry wall.

Besides the possibility to spectacularly increase the strength, the method also provides large deformation capacity and energy dissipation to the wall due to the yielding of the reinforcing bars [5], [6].

7.1.1.3 Reinforced plaster

Reinforced plaster is normal cement plaster, applied over a high-strength steel mesh [5]. By applying reinforced plaster, both the in-plane and out of plane strength of the masonry can be increased. It was reported [5], that an increase of strength from 1.25 to 3 times can be achieved, depending on the strength of the steel mesh and on the quality, and thickness, of the plaster.

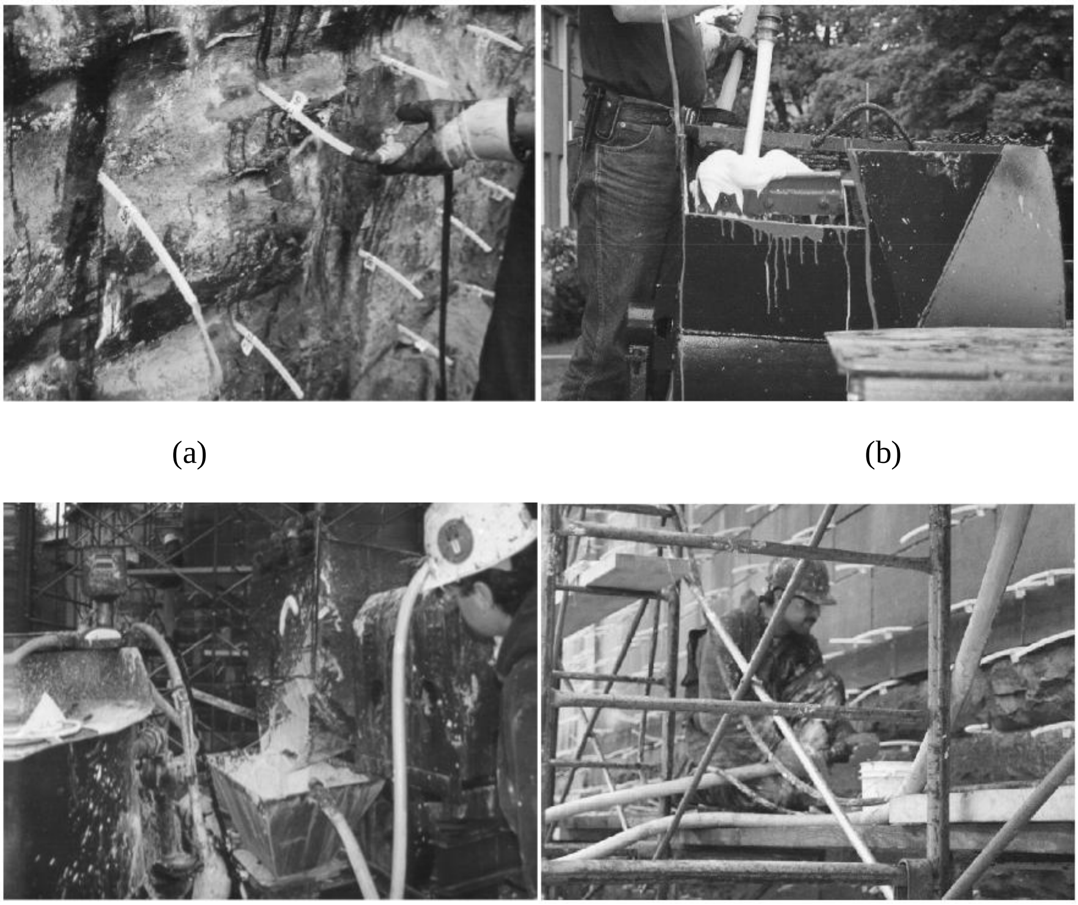
7.1.2 Grout injection

Grout injection is a classical method of rehabilitation, which has the advantage of not changing the architectural aspect of the building. Most of the time, the method is used for reestablishing the bond in the cracks of the wall. Therefore, the method aims to restore the original state of the wall.

In double layer masonry walls, both brick and stone, the filling of the internal cavity with cement based grout can significantly increase the strength of the wall, by ensuring composite action between the layers of the wall [5]. Similarly, the rubble core of cavity walls can often be strengthened by injection [10]. Usually the rubble core is seriously destroyed by water infiltration, and sufficient amount of cavities exist for efficient consolidation of the walls.

The work phases of the grouting operation are (some steps presented in Figure 5):

- Choosing the injection ports and sealing off the crack around them;
- Washing the surfaces of the cracks with water by injecting water in the ports. This process of soaking the wall is started with the lower ports, and it advances upwards. Besides the aim to improve the adherence of grout to the masonry, during the water injection, the active tubes can also be identified;
- Injection of the grout, with pressure adapted to the application; starting with lower ports and advancing upwards. Larger cracks are filled in a first step; repeating the procedure a second time for smaller cracks.



(a) (b)

(c) (d)

Figure 5. (a) Flushing the core with water; (b) Pumping foam into mortar mixer; (c) Transferring grout to pump hopper; (d) Injecting grout into inner core rubble [10]

The composition and consistency of the grout depends on the application. Epoxy resin is used for fine cracks of up to 2mm, while cement based grout is recommended for larger cracks and voids. It is important that the physical properties and the chemical composition of the grout should match the properties of the masonry [5].

By cement grout injection the strength of the wall can be recovered to 80-100% of the original (i.e. un-cracked) strength. An increase of up to 40% is also possible. When using epoxy resin an increase of strength (2-4 times) can also be achieved [5]. In both cases, the change of stiffness is insignificant (10-20%) and should not affect the applicability of the method. In case of filling the cavity walls with cement based grout, the strength gain can be as significant as 25-40 times [5].

7.1.3 External reinforcement

7.1.3.1 Diagonal steel strips

Diagonal steel strips can be provided on the face of the wall as external reinforcement of the URM (Figure 6.a). In fact steel strips are only one choice, as steel tubes, or FRP laminates can be similarly used.

Providing diagonal bracing in effect transforms the structure into some kind of X-braced frame, once the masonry is cracked. When the primary load bearing masonry is destroyed, the wall will resist the lateral load by developing tensile stresses in one brace and one vertical strip; and compression in the masonry at the opposite end of the wall [11]. The bracing and vertical strips can improve the strength of the wall by a factor of 4.5 [5], [11]. The vertical strips also improve the out of plane strength of the wall. The failure of the wall subjected to shear occurs due to the crushing of the toe of the wall in the compressed region. At larger drift values (1-1.5%) the buckling of the vertical and the bracing strips was observed [11]. However, such buckling does not affect the strength of the wall, because the role of the strips is to resist tension.

An alternative solution for the load transmission is that parts of the wall form piers, which resist the lateral loads by rocking (Figure 6.b). In this case all steel strips can be subjected to tension.

a) b)

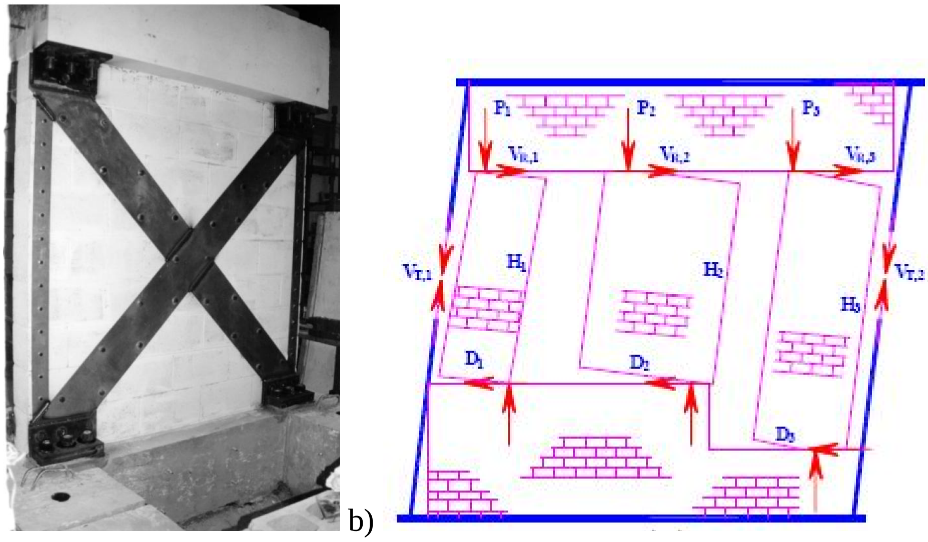


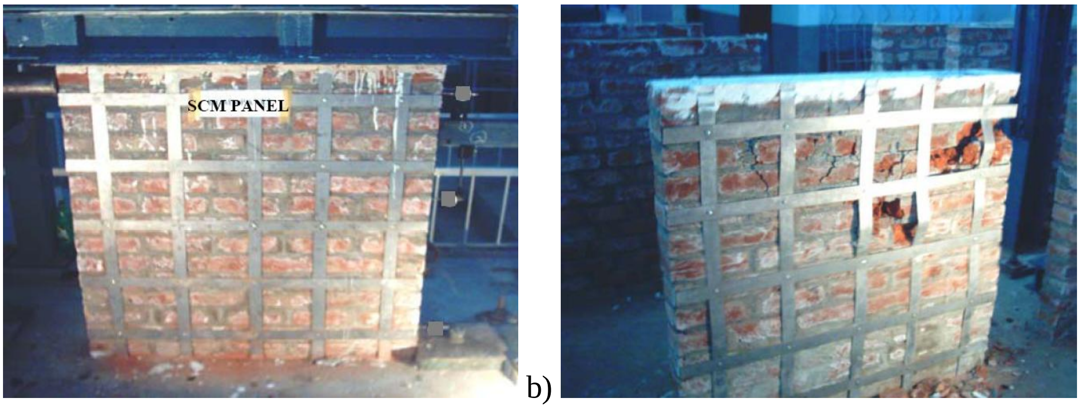
Figure 6. (a) Wall strengthened with steel X-braces, (b) piers exhibiting rocking behavior

An important factor influencing the effectiveness of the steel bracing system is the ration of rigidity between the URM wall and of the newly provided bracing system. It is presumed that the masonry will undergo significant cracking before the steel bracing will start to be effective [5].

It is also important to note the very strong, and stiff, fixing of the bracing ends (Figure 6.a) used in the testing. In real applications it is very difficult to achieve connections of such strength; very often there is no strong-enough elements in the vicinity of the wall to connect to. This practical limitation might prohibit the use of this method in some cases.

7.1.3.2 Rectangular mesh of steel strips

The method of using a mesh of steel strips for strengthening is a simple yet efficient way of improving the performance of masonry walls (Figure 7.a). Tests were carried out by Farooq *et al.* [8] on: (1) reference URM wall; (2) walls strengthened on one side with a finer mesh, (3) a coarser mesh; and (4) strengthened on both sides with the coarser mesh. racking test was carried out on the four configuration of walls, using a pre-applied vertical load of 18 tons.



a) b)

Figure 7. (a) Wall strengthened with steel mesh [8], (b) typical failure mode of the walls [8]

Improvement of the performance was observed. The shear strength of the walls increased by 30-40% for one sided strengthening and by 87% for the two sided strengthening. An increase of the masonry's compression strength in the range of 12-26% was also reported [8], with the effects of delayed micro-cracking, and a consequent increase of the elastic limit of the walls.

A very important advantage of the method is its simplicity, both in terms of material and labor requirements. The method can be easily applied, and it does not require special technology or qualification of the workers.

7.1.3.3 Three-dimensional tying systems

Repair or strengthening of masonry walls can also be achieved by means of tying masonry together with 3–D tying systems. One such proposal (CAM) is described in [15], based on the use of stainless steel ribbons (thickness 0.8mm, width 20mm) arranged in horizontal and vertical ties. The direction of the ties can vary and easily follow the random arrangement of construction blocks, often encountered in old masonry. The ties "package" the wall from both sides, the penetration points being strengthened with "drawpieces" (125x125mm, thickness 4mm). Slight pre-stressing is applied to the ribbons, making it possible to induce a stress state of 3D compression in the masonry.

a) b)


Figure 8. Application examples of CAM

The main advantage of the system is its versatility. The ribbons can follow the corners of walls, and virtually any imperfection of the walls. As the ribbons penetrate the wall, and form a mesh on both sides of the wall, the system is especially advantageous for double layer masonry walls with infill.

In [15] an initial testing program is reported on small scale (90x90x12cm) wall specimens loaded in shear. Specimens were tested, in shear, with and without CAM strengthening. Results show that, after initial cracking, they were able to restore the initial strength of the walls by CAM strengthening. Even a slight strength increase could be observed in the range of 15-50%. The most important improvement in the behavior of the walls was observed in terms of increase of ductility and energy dissipation. The ribbon packaging ensured a more uniform cracking pattern for the masonry and an increase of ductility due to the non-linear deformations of the stainless steel was observed. Energy dissipation is reported to have increased 30-60 times.

7.1.4 Confining masonry in RC tie columns and beams

Confining masonry in reinforced concrete (RC) tie columns and beams is the most common procedure to enhance the earthquake performance of masonry walls. The method is common practice, and is prescribed by the design codes, in most earthquake prone regions in South- America, Asia and Eastern Europe.

Vertical tie-columns are provided at every corner of the URM walls, at wall ends and door openings. In case of long URM walls such columns have to be placed at regular intervals given by the design code (e.g. every 4m). The tie-columns are linked horizontally by weak tie-beams at every floor level, or if the height is large at regular intervals (e.g. every 3.5m). The columns and beams are made of regular concrete and are reinforced with 4 bars of Ø8-16mm (Figure 9).

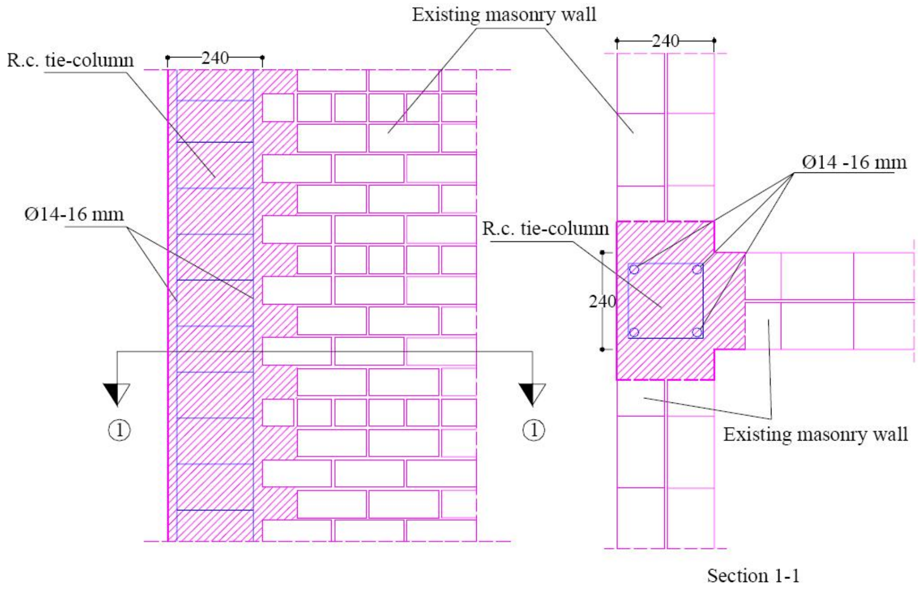


Figure 9. Example of tie-column arrangement in wall intersection [5]

The RC concrete provides a weak framing to the masonry. It enhances out of plane stability, and it confines the masonry in case of in-plane shear loading. The effect of the confinement is negligible in the un-cracked state of the masonry, but after cracking, the frame stops the masonry from disintegrating and contributes to the increase of ductility and energy dissipation. Therefore, the resistance of the wall is slightly affected (i.e. by a factor of 1.2...1.5); the cracking load can increase by a factor 1.27; the ductility and energy dissipation are reported to improve by 50% [5]. The performance of the system depends on the relative rigidity of the wall and of the RC “frame”.

The main disadvantage of the method in rehabilitation is that its labor requirements are very large. The removal of entire sections of the existing masonry in order to accommodate the RC ties is a tedious work which requires long interruptions of the occupancy of the building.

7.1.5 Center core method

In this method a vertical hole is drilled in the centre of the URM wall, on the entire height of several storeys, down to the basement or foundation. The usual diameter of the hole is 50-125mm, depending on the wall thickness, and the rehabilitation requirement (Figure 10.a).

A reinforcing bar is mounted, vertically to the centre of the hole. Filler material, usually cement based grout; polymer-sand or epoxy-sand mixture is pumped under pressure to the hole. The control of the pressure is extremely important in order to achieve a uniform and complete filling of all voids, from the base to the top, in order to assure good composite action. Supplementary connection elements (i.e. mechanical fixings) can be provided at intervals along the height; or at floor levels in order to anchor the floor and roof to the newly created “strong”-column [5]. If good bonding is achieved with the existing masonry, the newly created “strong”-column can extend beyond the limits of the hole.

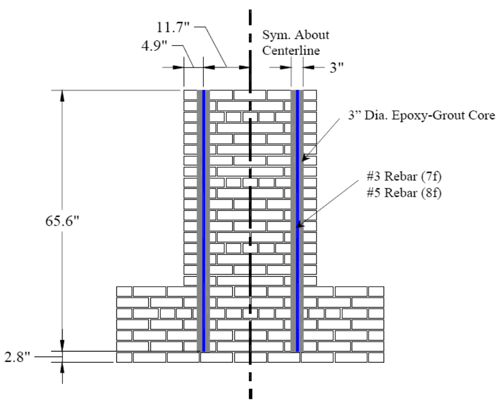
a)  b) 

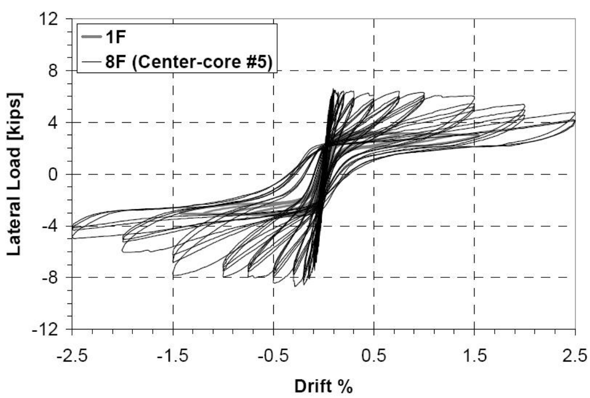
Figure 10. (a) Hole drilling and (b) grouting for centre-core rehabilitation in laboratory [6]

The procedure increases the lateral resistance of the wall to in-plane loads, mainly by providing a strong tie-down anchor in the uplift regions. The shear capacity is also slightly increased. The ties also improve out-of plane strength of the wall, but to a lesser degree.

Experiments carried out on masonry piers subjected to medium level vertical loads and shear have shown that by the centre core method the strength of the masonry can be doubled [6].

Failure in both tested specimens occurred due to rocking, and the energy dissipation was limited Figure 11.b. The large lateral displacements achieved by the specimen (Figure 11.b) are encouraging, but this may be influenced by the dimensions of the pier (Figure 11.a), not by the strengthening method used.





(a) (b)

Figure 11. (a) Dimensions and (b) characteristic curve of centre-core reinforced masonry pier [6]

The main advantages of the centre core method are related to the possibility to preserve the architectural aspect of the building, and that the intervention can be carried externally (i.e. the hole can be drilled from the roof level, with minimal disturbance to the occupants of the building). The main disadvantage is given by the fact that highly qualified personnel, high tech equipment and strict quality control are needed.

7.1.6 Post tensioning

7.1.6.1 Internal post-tensioning

Post-tensioning masonry is not a new concept. It refers to the construction method, in which the masonry wall is built in the traditional way, but after completion of the wall compression stresses are introduced to the masonry by the use of pre-mounted tendons.

Masonry has reasonably large compressive strength, but very small tensile strength. The idea of post tensioning is to introduce compressive stresses in the masonry so as to counterbalance the tension which would be generated by the external loads. The techniques used to achieve this effect are very varied:

- Tendons are mostly multi-strand steel cables, but mono-strand cables are also used. Some systems [16] also use carbon fiber tendons;

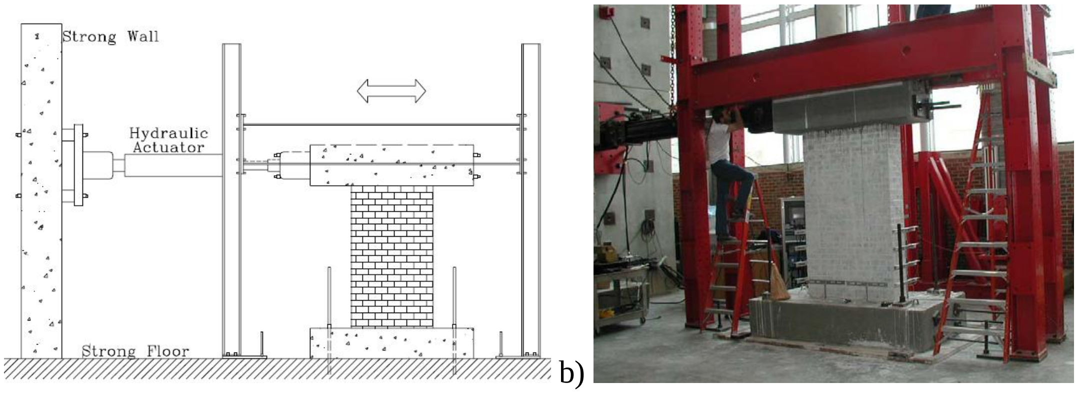
- Tendons can be placed in holes drilled into the masonry; or the masonry blocks themselves can have pre-drilled holes. In some cases [17], the masonry blocks are shaped in a way that the wall can be built around the tendon, without the need to be introduced in a special hole in the blocks;

- The hole hosting the tendons can be grouted or not. Un-grouted tendons have the advantage that the tensile force can be checked periodically, adjusted if needed, or the post-tensioning can be removed altogether;

- In most post tensioned walls the masonry blocks are laid in mortar. However, in some cases dry-masonry can also be post tensioned [17].

Comparative cyclic racking tests, on five configurations of post-tensioned walls with 3 tendons, and an aspect ratio of 2/1 [18] showed that, in order to ensure advantageous behavior, the compression part of the wall has to be strengthened. In [17] this has been achieved by~~:~~ grouting the orifices of the concrete blocks and providing confining steel plates in the toe area of the wall. The beneficial effect of supplementary mild steel reinforcements is also reported (damping increase from 7% to 10-12%). Unfortunately, such performance improvements are difficult to achieve in a rehabilitation work.

It is common practice not to bond the tendons in the holes because bonding limits the drift at failure (e.g. from 6.5% to 3% in [18]).



a) b)

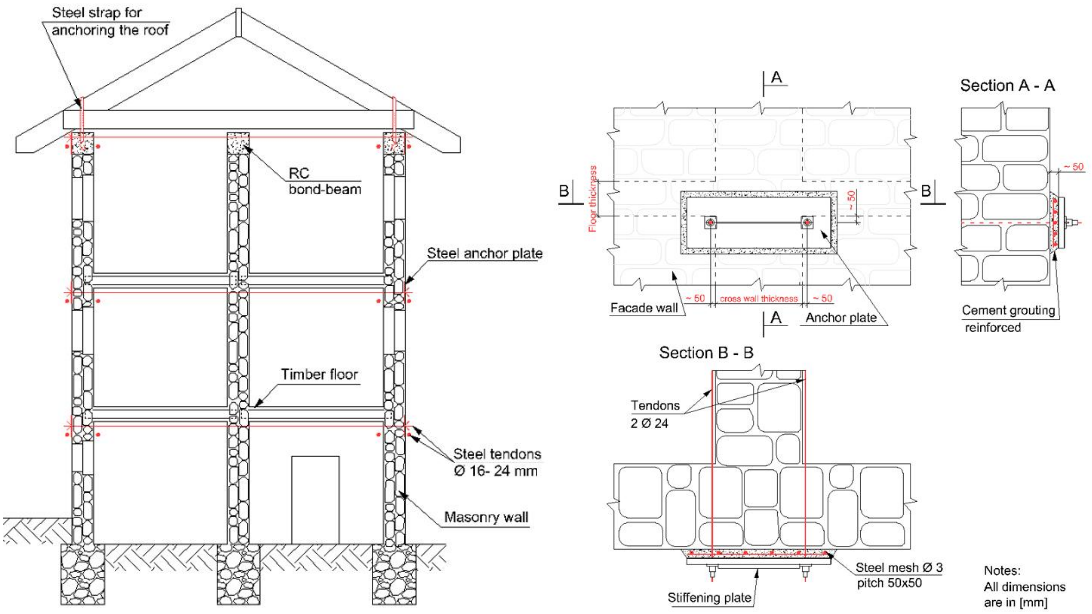
Figure 12. Test configuration of post-tensioned walls [18]

There is an abundance of studies on post-tensioned masonry walls, carried out on concrete-block masonry (e.g. [19], [20]), and a much more limited number on clay-brick masonry [18]. Unfortunately, comparative tests of URM walls vs. post-tensioned walls could not be found, so the gain due to post-tensioning can not be documented.

7.1.6.2 External post-tensioning (binding)

The good binding between different components of the masonry structure is an essential prerequisite of good earthquake performance. This requirement, with special focus on joints, is also discussed in Chapter 7.5 of this document.

Opposite walls can be connected with steel or FRP rods (Figure 13). The rods may be post-tensioned for better effectiveness. Similar strengthening method is traditionally used against opening of arches due to outwards compression.



(a) (b)

Figure 13. Binding of opposite walls. Layout of the rods (a) and fixing details (b) [14]

The effect of post tensioning rods depends very much on the configuration of the building and the way rods are used. Few systematic studies of this method exist; some are unusual and they are proposed for low-cost structures [22].

The biggest disadvantages of the method are that external straps and connections might affect the architectural aspect of the buildings, and the post-tensioning elements, being external, are exposed to corrosion.

7.2 Walls - FRP based strengthening

7.2.1 FRP typologies and application methods

Fiber Reinforced Polymer (FRP) composites are common in rehabilitation and strengthening of masonry walls. Strong, lightweight and relatively easy to apply FRP materials show large potential for strengthening masonry structures against their most critical failure mechanisms.

Glass, Carbon and Aramid Fiber Reinforced Polymers (GFRP, CFRP and AFRP) and, at least in one case, PolyVinylAlcohol Reinforced Polymers (PVAFRP) [27] have been used. The most common binder material (i.e. matrix) in which reinforcing fibres are embedded, is epoxy, because it has better bond strength to the substrate materials than other polymer matrices.

FRP has been applied in different forms: Uni-Directional (UD) laminate strips, bidirectional fabrics and Near Surface Mounted FRP bars, rods or strips [36]. Large strength increases have been reported in masonry wall in-plane [23], [27], [28], [29], [30], [32], [34], [35] and out-of-plane [23], [24], [25], [29], [31], [33], [34], [36] strengthening. Most of these studies have focused on masonry walls or panels, but also masonry infill walls [35] have been studied. However, the authors believe that the whole potential of FRP's has not been utilized yet; and the studies, and in some cases even practical design, have not taken all important earthquake engineering aspects fully into account.

7.2.2 Uni-directional FRP strips

7.2.2.1 Unidirectional (UD) FRP in X assemblies

UD FRP strips have been utilized in various assemblies. Perhaps the most typical assembly for shear strengthening is the X assembly (Figure 14, [28]) with two strips or wider plates along the diagonal of the wall [26], [28], [29], [35].

Foster *et al.* [26] tested URM buildings strengthened with various FRP assemblies under static-cyclic lateral loading. The dimensions of the buildings were 2.84 m height, 3.25 m width and 4.47 m length. The shear walls were perforated with two 82 cm wide doorway openings in two structures (aspect ratios of the piers 1 and 2.7). The roof diaphragm was rigid. Besides the lateral load, weight of two additional floors was simulated.

Repairing a damaged concrete masonry building with GFRP laminates (X-assembly + vertical at jambs) increased strength by about 65%, displacement capacity by 240%

and energy dissipation (average of tension and compression values of strain energy) by 500%. The mode of failure was masonry substrate failure.

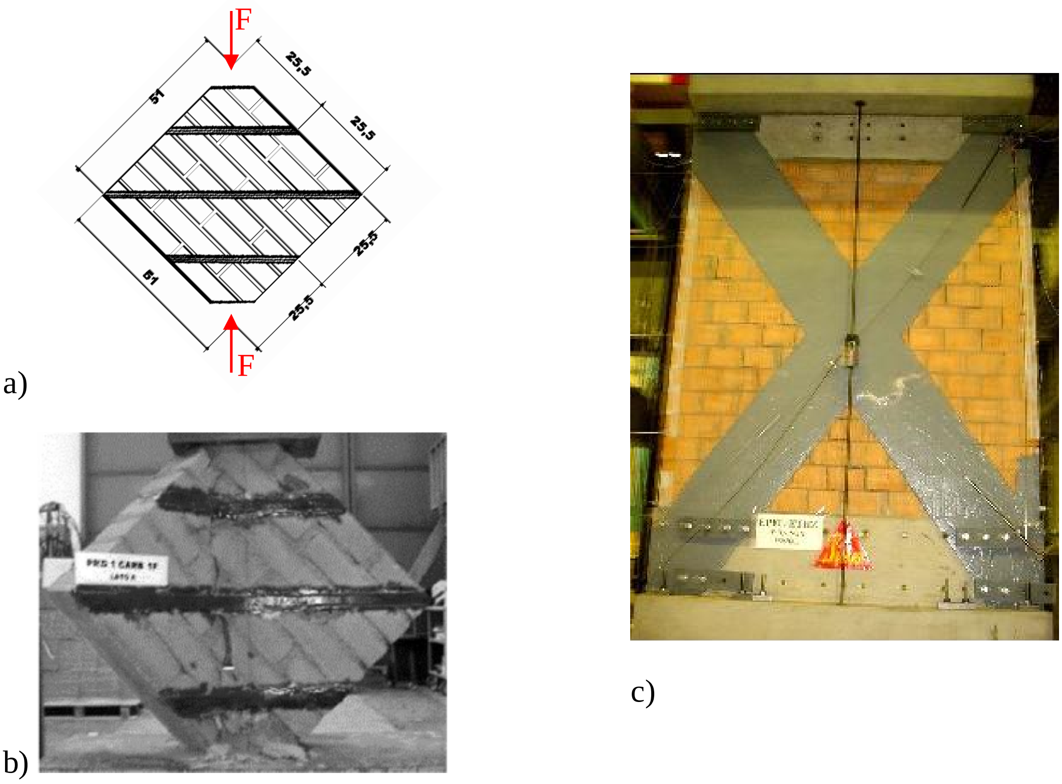


Figure 14. (a) 1.7 cm, 2 layer CFRP diagonal strengthening and (b)testing of wallettes [27]; (c) FRP assembly with UD laminate placed in X [28]

Valluzzi *et al.* [27] tested concrete and clay brick masonry wallettes (HLW=51×51.5×12cm) reinforced, in X, with UD FRP strips for monotonic in-plane loading. The FRP materials used were CFRP, GFRP and PVAFRP. They used one-sided and double-sided strengthening.

One-sided strengthening leads to sharp diagonal cracking on the opposite side and may even weaken the wall compared to the URM case. Strengthening on both sides changes the failure mode to more spread, but still globally diagonal, shear cracking of the masonry, followed by either debonding between the FRP and the masonry (peeling of the superficial layer of the masonry), or cracking of the FRP.

The strength increase obtained with one-sided X reinforcement was 10-15%; whereas the strength increase obtained with double-sided X strengthening was 45-74%. Only the diagonal GFRP and CFRP assemblies on both sides of the walls gave strength increases that exceeded the variance of results on the baseline specimens.

ElGawady [28] tested ‘slender’ (HLW = 160×157×7.5cm) and squat (HLW = 70×157×7.5cm) hollow clay brick URM and retrofitted masonry walls under in-plane static cyclic and dynamic loadings. The walls were retrofitted with FRP and the effect of post-tensioning in the rocking mode was simulated by vertical forces. All FRP retrofits were on one face only. The URM walls were first tested to some damage and then retrofitted.

The dynamic excitations were synthetic earthquake acceleration time histories compatible with Eurocode 8 for rock soil type A with a peak acceleration 1.6 m/s^2. Under dynamic testing, GFRP fabrics in X on one face failed by fiber rupture (i.e. shear failure of the wall) and increased strength 35% and displacement at failure 45%. In the squat wall GFRP XX assembly on one face increased strength about 15% and displacement at failure about 45%.

Moon [29] tested a two-storey clay brick masonry URM building with several window and door openings for cyclic lateral displacement loading. One wall (Wall B, [29]) was retrofitted with GFRP strips in an X-assembly plus vertical strips in each pier on one side and horizontal GFRP NSM rods in the spandrels on other side. This mixed retrofitting allows no comparison of the effectiveness of the system. About 30% increase in shear capacity and a 105 % increase in deformation capacity were observed.

The force-displacement response of the structure showed a large amount of ductility due to gradual debonding of the NSM GFRP rods and the GFRP. Strong interaction between adjacent walls was observed, with cracks extending from one wall to another. The adjacent walls reduced out of plane displacements of the walls considerably. According to Moon [29] it is enough to use reinforcement on one side of the wall. It was noted that low-level rocking reduces the shear strength of the lowermost mortar joints and thus reduces the sliding resistance of the walls.

Grillo [32] tested URM piers (HLW=120×120×20cm) of concrete masonry strengthened with FRP in conjunction with ductile steel dowels to restrain in-plane sliding and rocking. The loading was static-cyclic lateral loading.

Table 1. Effectiveness of X retrofitting with FRP

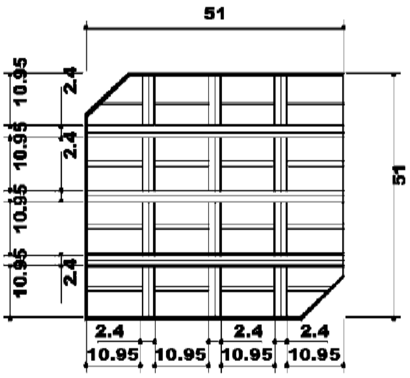
Ref.	**Base mat.**	**H/L**	**Load**	**BC sides**	**BC top-bot.**	**Retrofit mat.**	**Retrofit config.**	**Side**	**Str. (%)**	**Disp (%)**	**En.diss. (%)**
[26]	CMU	1/2.7	cyc.	free	no reinf.	GFRP	X+vertical at jambs	1	+67	+240	+500
[27]	CBM	1	mon./diag	free	free	CFRP	diagonal parallel	1	+15	-	-
[27]	CBM	1	mon./diag	free	free	GFRP	diagonal parallel	1	+11	-	-
[27]	CBM	1	mon./diag	free	free	CFRP	diagonal parallel	2	+45	-	-
[27]	CBM	1	mon./diag	free	free	GFRP	diagonal parallel	2	+74	-	-
[28]	CBM	1	dyn.	free	FRP anch. + steel post tens	GFRP	X	1	+35	+47	-
[28]	CBM	0.45	dyn.	free	FRP anch. + steel post tens	CFRP	XX	1	+16	+43	-
[29]	CBM	1-1.7	cyc.	free/ cons	no reinf.	GFRP	piers/(X+ver) + horiz. NSM	2	+30	+105	same
[32]	CMU	1	cyc.	free	steel dowls + FRP fabric	GFRP	X+vert. + bidirect. top & bottom	1/2	+200.. +800	-	-

The GFRP assembly was diagonal UD strips in X configuration and vertical strips at the jambs. Improvements in shear strength of 200% to 800% were achieved according to the author. However, supplementary to the X GFRP bi-directional FRP composite fabric with a ±45º fiber orientation was applied along the top of the pier at the pier/lintel interface; FRP composite was placed at the top of the base, along its length; shear reinforcement was added by covering the base with bi-directional FRP. This system ensured good integrity between the pier and the lintel as well as between the pier and the base, and emphasizes the beneficial effect of good connection of the wall with the surrounding elements.

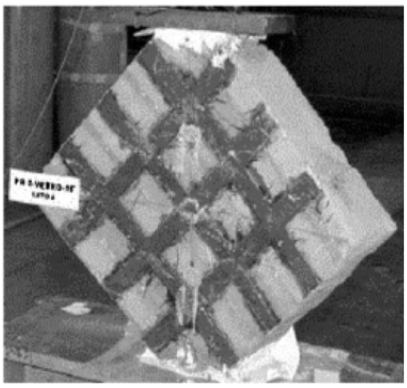
A summary of the gain given by FRP retrofitting in X assembly is given in Table 1.

7.2.2.2 Rectangular unidirectional (UD) FRP grids

Rectangular grids (Figure 15.a [27]), vertical strips or plates [28], [29], [30], especially near both ends of the wall or at the jambs of a pier, horizontal strips [29] and also several parallel diagonal strips (Figure 15.b [27]) have also been used for retrofitting URM.



a)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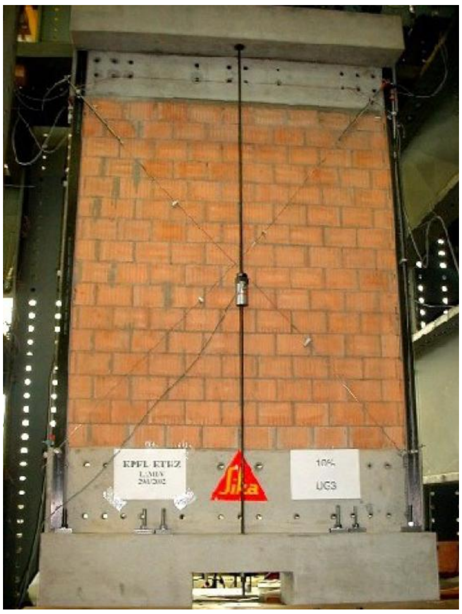

c)

Figure 15. Rectangular FRP grids:(a) typical grid configuration and (b) test results by [27];2 vertical UD CFRP plates placed at ends on one face [28]

Foster *et al.* [26], by strengthening an undamaged CMU building with GFRP (vertical at jambs) increased strength 35%, displacement capacity 80 % and energy dissipation 170%. The mode of failure was diagonal tension cracking in all piers.

Valluzzi *et al.* [27] tested wallettes (HLW=51×51.5×12cm) described at §7.2.2.1. The one-sided strengthening was reported to be not effective for rectangular grid assembly (similarly as to X assembly).

Compared to X assembly the rectangular grid offers a better stress distribution that causes crack spreading and a less brittle failure. In most cases less stiff FRP material appeared to be more effective both in terms of ultimate strength and stiffness increase. The variance of the baseline specimen results overshadowed the effectiveness of the rectangular grid, except for PVAFRP grid on both sides, which gave strength increases of +47%.

ElGawady's [28] (see §7.2.2.1) 'slender' walls (HLW=160×157×7.5cm) showed in dynamic testing: no change of strength when retrofitting with 2 vertical CFRP plates on one face, but a decrease of displacement at failure of 55%; strength increase of 55% and displacement increase of 36% when retrofitting with full face GFRP grid on one face. The 'slender' wall retrofitted with 2 vertical CFRP plates failed in combined rocking and shear.

Moon [29] tested a two-storey clay brick URM building with several window and door openings for cyclic lateral displacement loading, using different retrofitting systems in the pears and spandrels of the different walls of the first floor. In addition to the in-plane retrofitting the joints between the walls and every third floor joist were retrofitted by joist anchors. The aspect ratios of the piers varied between 0.4 and 4.0 with values 1.0, 1.2, 1.7 and 2.0 being typical.

Due to the difficulty of anchoring to the foundation the aim of the retrofitting was to suppress diagonal shear failure and flexural failure, but not wall sliding (cracking in the lowest mortar joints). Reinforcement for toe crushing was considered not necessary due to the relatively small vertical load.

Moon [29] reported 10...15% decrease of strength and 500% increase of deformation capacity using a retrofit with an assembly of vertical and horizontal GFRP strips in each pier (Wall 1, [29]). The gain given by retrofitting in rectangular assembly is:

Table 2. Effectiveness of rectangular FRP grid assemblies

Ref.	**Base mat.**	**H/L**	**Load**	**BC sides**	**BC top-bot.**	**Retrofit mat.**	**Retrofit config.**	**Side**	**Str. (%)**	**Disp (%)**	**En. diss (%)**
[26]	CMB	1/2.7	cyc.	free	no reinf.	GFRP	vert.	1	+35	+80	+170
[27]	CBM	1	mon./diag	free	free	CFRP	rect. grid	1	-10	-	-
[27]	CBM	1	mon./diag	free	free	GFRP	rect. grid	1	-5	-	-
[27]	CBM	1	mon./diag	free	free	PVAFRP	rect. grid	1	0	-	-
[27]	CBM	1	mon./diag	free	free	CFRP	rect. grid	2	+3	-	-
[27]	CBM	1	mon./diag	free	free	GFRP	rect. grid	2	+14	-	-
[27]	CBM	1	mon./diag	free	free	PVAFRP	rect. grid	2	+47	-	-
[28]	CBM	1	dyn.	free	FRP anch. + steel post tens	CFRP	2 vert. plates	1	0	-55	-
[28]	CBM	1	dyn.	free	FRP anch. + steel post tens	CFRP	full-face grid	1	+55	+36	-
[29]	CBM	1-1.7	cyc.	free/ cons	no reinf.	GFRP	piers/vert + spandrels/ horiz.	1	-14	+500	small +

Rectangular UD FRP strips are also very effective for out-of plane (i.e. bending) strengthening of URM. Galati *et al.* [37] reported that if the wall behaves as a simply supported element (out-of plane) external FRP reinforcement is very effective, but for fixed end condition the effect is limited, due to stronger "arching effect" [37]. Masonry walls, with H/W ratio of 12.8 and 13.2, reinforced with vertical UD GFRP laminate strips were tested with the two end conditions. For simply supported end conditions a strength increase of 175%-325% was obtained, compared to 25% increase in case of restrained end condition.

Tumialan *et al.* [24] obtained 460-1300%/210-790% strength increase for CMU/clay-brick masonry walls (HLW=120×60×9.5cm) by using externally bonded GFRP and AFRP strips. In contrast, Barros *et al.* [36] obtained only 90% bending capacity increase with externally bonded FRP strips on masonry panels with H/W ratio of 3.7, as arching effect must have a considerable influence on the bending strength of such a stocky panel.

7.2.3 Bidirectional FRP laminates

Bidirectional laminates [28], [30], [34] have been used either covering the whole face of the wall or only the piers.

a) 


(b)

Figure 16. FRP bidirectional laminates. (a) rocking failure with rupture of FRP [28]; shear failure of wall strengthened with FRP fabric with vertical fibers [50]

ElGawady's [28] (see §7.2.2.1) 'slender' walls (HLW=160×157×7.5cm) with one sided full-face GFRP fabric, failed by rocking under dynamic loads. The full cover of GFRP fabric on one face increased strength about 85% and displacement at failure 20%. Under similar dynamic loading, for the squat walls (HLW=70×157×7.5cm), single sided full-face GFRP/AFRP fabric increased strength at least 140 %/130% respectively (the specimens did not reach failure).

Under static cyclic tests, the same type of squat walls experienced 50..150% strength increase with similar, single sided full-face GFRP fabric. The displacement at failure decreased about 55% in one case, but increased 400% in another case. The full face GFRP fabric on both sides increased strength about 240% and displacement at failure 500%. 2 layers of full face AFRP fabric on one face increased strength about 470% and displacement at failure 1000%.

A large part of the displacement values is caused by the rocking. The test arrangement was such that the rocking increased the vertical force and the lateral resistance. Thus the strength values are not fully comparable. The dynamic and static tests of squat walls with one layer of fabric on one face gave strength values in the same range, but the debonding observed in dynamic tests was not observed in static cyclic tests.

In Moon's [29] two-storey URM building (see. §7.2.2.2), Wall 2 was retrofitted with bidirectional GFRP overlays covering the first storey. About 10..15% increases in shear capacity and 195% increase in displacement capacity were observed.

Vandergrift *et al.* [34] tested URM walls (HLW=121.9×243.8×19.4cm) retrofitted with multiple layers of CFRP composite fabrics for in-plane static cyclic loading.

One as-built shear wall specimen was tested monotonically and two in static cyclic loading. The monotonic in-plane loading gave almost twice as high strength for the as-built wall as the static cyclic in-plane loading. According to the authors a ±45 layout for the FRP laminate is most effective for shear loads. Retrofitting with ±45 laminate gave about 1055% increase in shear strength. Retrofitting with 0_2 laminate gave about 920% increase in shear strength. Retrofitting with 0/90 laminate gave about 835% increase in shear strength. These values are very high, but one should note that multiple layers of FRP were used. All retrofitting resulted in large increase of displacement capacity.

Table 3. Effectiveness of bi-directional FRP laminates

Ref.	Base mat.	H/L	Load	BC sides	BC top-bot.	Retrofit mat.	Retrofit config.	Side	Str. (%)	Disp (%)	En.diss. (%)
[28]	CBM	1	dyn.	free	FRP anch. + steel post tens	GFRP	full-face fabric	1	+84	+20	-
[28]	CBM	0.45	dyn.	free	FRP anch. + steel post tens	GFRP	full-face fabric	1	>+139	-	-
[28]	CBM	0.45	dyn.	free	FRP anch. + steel post tens	AFRP	full-face fabric	1	>+132	-	-
[28]	CBM	0.45	cyc.	free	FRP anch. + steel post tens	GFRP	full-face fabric	1	+42	-57	-
[28]	CBM	0.45	cyc.	free	FRP anch. + steel post tens	GFRP	full-face fabric	1	+150	+400	-
[28]	CBM	0.45	cyc.	free	FRP anch. + steel post tens	GFRP	full-face fabric	2	+239	+500	-
[28]	CBM	0.45	cyc.	free	FRP anch. + steel post tens	AFRP	full-face fabric	1	+471	+1000	-
[29]	CBM	1..4	cyc.	free/ cons	no reinf.	GFRP	full-face fabric first floor	1	+14	+195	small -
[34]	CMU	2	cyc.	free	grouting, shear studs	CFRP	full-face fabric, ±45°, many layers	2	+1056	large +	-
[34]	CMU	2	cyc.	free	grouting, shear studs	CFRP	full-face fabric, 0°, many layers	2	+919	large +	-
[34]	CMU	2	cyc.	free	grouting, shear studs	CFRP	full-face fabric, 0/90°, many layers	2	+833	large +	-

Hamoush *et al.* [31] tested CMU walls (HLW=180×120×20cm) in monotonic out-of-plane bending. One of the strengthening systems used was full-face covering with continuous glass fabric. This reinforcement generated an increased of bending strength of 1830-2150 %.

Good surface preparation and careful lamination was found to be very important. If an initial imperfection or separation occurs in the fiber overlay due to an entrapped air bubble or fault during fabrication, premature failure of the fiber overlays may occur due to stress concentration resulting from rapid propagation in the joint opening adjacent to the faults.

7.2.4 Near Surface Mounted techniques (NSM)

Near Surface Mounting (NSM) is a technique in which FRP bars or FRP UD strips are installed into slots grooved into the masonry surface and the slots are then filled with epoxy based or cementitious grout. When using this method for hollow block masonry, care must be taken not to make the grooves too deep. If the FRP is installed in slots grooved at the mortar joints between the blocks, the method is called FRP structural repointing.

NSM FRP has been used as horizontal [23], [33] and vertical [23], [25], [33] reinforcement. NSM is less laborious than many other strengthening methods. It requires no surface cleaning and leveling. The change in the appearance of the structure is very small, especially in the case of FRP structural repointing.

a)  b) 

Figure 17. (a) Installation of NSM GFRP bars; (b) test set-up of wall [49]

Foster *et al.* [26] reported test results of URM buildings (see. 7.2.3) strengthened with NSM CFRP and GFRP. According to [26], repairing a damaged brick masonry building with NSM CFRP bars increased strength by 120%, displacement capacity 170% and energy dissipation by 220%. Strengthening undamaged brick masonry building with NSM GFRP rods increased strength 10%, displacement capacity 45%

and energy dissipation 35%. The mode of failure was shearing of the GFRP rods in this case.

As mentioned in §7.2.2.1, Moon [29] tested a two-storey clay brick URM building, where in one wall (Wall B, [29]) one side was strengthened with GFRP NSM rods. Unfortunately, this was part of a mixed retrofitting which does not allow comparison of the effectiveness of the system. It is mentioned in [29] that the gradual debonding of the NSM GFRP rods contributed to the large amount of ductility observed in the test.

Tumialan and Nanni [23] tested CMU wallettes (HLW=61×122×9.5cm), strengthened by NSM FRP bars, for monotonic in-plane shear. Walls were strengthened with FRP bars at every horizontal mortar joint. The loading was monotonic along one diagonal.

In the reference URM wall the failure was brittle diagonal shear cracking with the crack stepping along the mortar joints. The NSM FRP bar strengthened wall failed when the shear cracks widened and the GFRP bars were not able to carry the tensile load any more due to debonding of the GFRP from the masonry. The shear capacity increased about 80%. The increase in deformation capacity was very large.

NSM FRPs are also effective for out-of plane strengthening of masonry. Bajpaj & Duthinh [25] reported on out of plane bending of under-reinforced slender (HLW=285×80×20cm and HLW=285×40×20cm) CMU masonry walls reinforced with NSM GFRP bars. The walls were subjected to 4 point bending (Figure 18). The flexural failure was consistently initiated by tensile rupture of the reinforcement with an increase of the moment capacity of 30..50 %. This moderate gain can probably be attributed to the fact that~~:~~ already existing steel reinforcement increases the bending strength in the baseline case, and the tensile strength of the NSM GFRP bars was limiting the bending capacity.

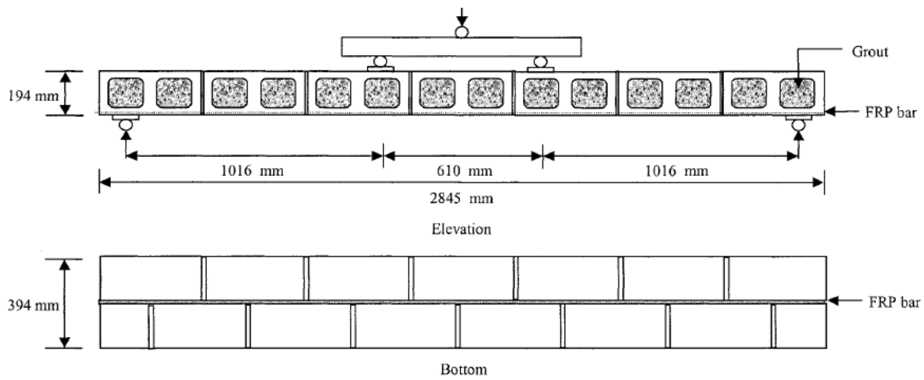

Figure 18. Out of plane bending tests arrangement by Bajpaj & Duthinh [25]

In fact Korany & Drysdale [33] obtained much larger out-of-plane strength increase on slender clay brick masonry walls, using NSM CFRP strengthening. The dimensions of the walls were HLW=126×59×9cm for vertically spanning walls, and HLW=119×59×9cm horizontally spanning walls. With carbon fiber ropes, placed in straight continuous grooves cut at head joints and through brick units, the strength increase was 1240% for vertically spanning walls, and about 220 % for horizontally spanning walls for cyclic loading. Surprisingly, only 100% and 10% increase was obtained from monotonic test on the vertically and horizontally spanning walls. There was a large difference in the strength of the reference walls between the monotonic and cyclic tests.

Barros *et al.* [36] obtained 105% bending strength increase on stocky wall (H/W=3.7) strengthened with NSM FRP strips, and loaded monotonically.

It appears that the scatter of these results is very large, and the efficiency of the NSM FRP in out-of-plane bending depends strongly on the configuration of the masonry wall and the anchoring strength. As most of the tests have been conducted on relatively slender walls as simply supported, there is a risk of overestimating the strength increase obtainable by FRP strengthening.

7.2.5 Strengthening of toe by FRP confinement

Sometimes FRP has been used also at the toes (lower corners of the wall of the first floor) of slender walls to protect the toes against crushing. The FRP fabrics used for strengthening have mostly been stitched fabrics, but also woven fabrics have been used [26][31].

7.2.6 Strengthening of mortar joints

In old masonry the mortar is often very weak, or there are multiple wythes poorly interlinked. Several authors have suggested ways to improve the strength of the mortar joints or to tie the wythes, adjacent walls and walls/floors together. Some of these methods involve use of polymers and/or carbon fibers. Zhu and Chung [40] studied improvement of brick-to-mortar bond strength by the addition of short carbon fibers to the mortar. 110% increase in bond strength under shear loading and 150% improvement in bond strength under tensile loading was obtained. Sofronie [39] presented a method for strengthening the mortar joints by polymer grids and stated that this method strongly improves the ductility of the mortar joints. Anagnostopoulos & Anagnostopoulos [41] studied polymer-cement mortars for improvement of the mechanical properties of ancient masonries. Large improvements in flexural and shear strength were obtained by mortars with high latex content. Burdette *et al.* [42] and Straka [43] studied FRP ties for masonry walls. FRP ties were found feasible, but the fire safety was stated to be a problem. The smaller energy dissipation compared to steel ties was not discussed in these papers. The creep of polymers may also become a problem.

7.2.6.1 Textile reinforced mortars (TRM)

A special methods related to FRP strengthening is the Textile Reinforce Mortar (TRM). In TRM the epoxy matrix is replaced by cement based mortar [38] in external strengthening of walls. This alternative may be feasible if there is danger of high temperatures or the strengthening is needed to be done on wet surfaces or in too low temperature to cure epoxy matrix.

Papanicolau *et al.* [38] compared TRM with FRP as strengthening material for URM in static cyclic out-of-plane loading. Slender (HLW=130×40×8.5cm) and squat (HLW=40×130×8.5cm) clay-brick masonry wallettes were tested. The conclusions were that if failure occurs in the textile (tensile failure) the FRP is more effective than TRM; but if the failure occurs in the masonry (compression/shear failure), the TRM is more effective.

7.2.6.2 Polymer grids

The strengthening of mortar joints with polymer grids increases the tensile strength and ductility of the masonry [39]. This method can be used for retrofitting of existing walls only if the existing mortar is partially or totally removed. It can be applied easier when replacing badly deteriorated walls or wall parts by new construction. Especially applicable this method is in strengthening wall-to-wall, wall-to-column or wall-to-foundation joints.

7.3 Advanced techniques - Damping devices, dissipaters, base isolation

Masonry walls usually have very low in plane deformability (i.e. they are very stiff). Only very slender walls, with a failure mode dominated by rocking can develop more significant horizontal deformations without sudden, and dramatic, decrease of the load bearing capacity.

Therefore, connecting any passive structural control system parallel to the wall would not be efficient due to lack of sufficient deformability. Passive control devices can be efficient: (1) if they act perpendicularly to the direction of the masonry shear walls and they form a separate load bearing system; (2) they are coupled in parallel with very slender walls which fail by rocking; or (3) they act between the foundation/roof/floor and walls, so that they can accommodate motion developing between the components of the building.

The other option for using passive control devices is that the masonry walls are allowed to crack before the devices are activated by the significant deformation after cracking. This implies that damage to the structure is accepted, and the devices are used only as “collapse-prevention” tools in the non-linear deformation range.

7.4 Columns

7.4.1 Goals of column strengthening

The literature on FRP jacketing of RC columns is extensive, but not many papers have been published on strengthening of masonry columns with FRP. Some results exist on FRP jacketing of masonry columns for vertical compression loading [47], [48]. However, it is uncommon that columns are strengthened for vertical loads. Especially in earthquake loading situations the main problem with columns is related to flexural capacity and ductility. More rarely, in short columns, insufficient shear strength might also be a problem [4].

The usual goal of columns strengthening is to ensure that shear strength is sufficient to allow the column to develop flexural failure. Shear failure of the columns is non-ductile, it lead to the sudden loss of the load bearing capacity, and can trigger structural collapse. On the other hand, flexural failure is usually ductile enough to ensure sufficient displacement capacity.

The increase of shear strength can be achieved by active or passive confinement of the column, especially in the region of the potential plastic hinge. Passive confinement can be achieved by external jacketing. Jacketing can be made with steel or advanced composite material (i.e. wrapping), over the prepared and cleaned surface of the column. Circular columns only need surface preparation, but the cross-section of rectangular columns is usually changed into oval or circular by external concreting. This is necessary for the jacket to develop radial tension stresses [4]. For active confinement, the jacketed column is pressure-grouted.

7.4.2 FRP jacketing

Bieker *et al.* [47] studied post-strengthening of URM columns, with rectangular cross section (HWL=50×24×24cm), by FRP jacketing, using UD CRFP and UD GRFP tapes, with varying number of layers. The specimens had rounded corners.

The increase in ultimate load by the jacketing was 250..300% for solid brick columns, with calcium mortar; and 30..60% for vertical-coring brick columns with calcium-cement mortar.

Corradi *et al.* [48] reported on the effect of confinement with CFRP, of brick masonry columns, under compression. The cross sections of the specimens were rectangular (24.5×25cm) and octagonal (8×10cm). The height of the columns was 50cm. The corners of the rectangular section were rounded by steel elements.

53..105% strength increases was obtained for the rectangular columns, and 95..104% for octagonal columns. The rectangular columns showed vertical rupture of the jacket at the corners, whereas the jacketing around the octagonal cross section columns was broken horizontally to stripes before final vertical rupture. In both cases the masonry inside the jacket was severely crushed.

Performance improvement [4] resulting from cyclic tests on RC columns range from 1 to 2 times increase of lateral strength, and 2 to 4 times increase of displacement ductility. The obtained gain depends on the geometry of the column, the magnitude of the axial load, the ratio of longitudinal reinforcement. Higher values of strength increase correspond to columns with rectangular cross-section, where the added material of the oval concrete bolster has a significant contribution (Figure 19). Lower strength increase was reported [4] for circular columns.

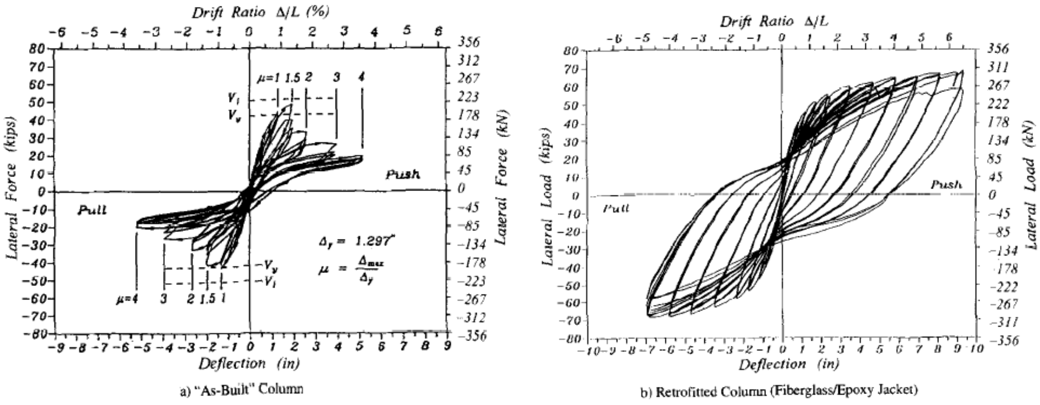

Figure 19. Force-displacement response of flexure dominated circular column. "As-built" and retrofitted with fiberglass/epoxy jacket [4]

7.4.3 Discussion

Only two papers on strengthening of URM columns were found, addressing vertical compression only. Thus it is impossible to make conclusions on earthquake strengthening of masonry columns by FRP jacketing.

Most of the principles used in FRP jacketing of RC columns should be applicable for masonry columns. The aim of the confinement should be to prevent shear failure before bending failure and insure flexural deformation capacity.

The main difference between strengthening of URM and RC columns is that the tensile and shear strength of masonry is smaller than that of RC. If URM columns are jacketed with FRP, one should avoid using predominantly horizontal UD fibers, because when the jacket fails the masonry inside is already cracked and crushed so badly that it cannot sustain flexural or shear loads any more. Thus, fabrics with ±45 or ±30 degrees orientation could be used instead of horizontal UD fibers. This would prevent the striping failure of the jacket, while not increasing the bending capacity of the column too much.

Interesting to note that providing extra longitudinal reinforcement, either by steel bars or by having longitudinal fibers in the wrapping composite, is not always advantageous, because it increases the flexural strength of the column. The bending strength of columns rarely affects significantly the capacity to the structure, because in plane walls are much stiffer and attract most of the load. However, the displacement capacity of the columns is important so that they can follow the deformation of the structure.

7.5 Joints

The proper jointing of structural elements is a pre-requisite of good earthquake performance. These joints have to ensure the smooth transition of forces between the elements: slab-to-wall; wall-to-wall; slab-to-column, wall-to-foundation details should all be investigated. During the refurbishing of masonry buildings it is extremely important to insure that there is a good 3D structural assembly. This is first priority. The rehabilitation of the individual elements is only the second step, and as strengthening joints almost always improves earthquake performance, strengthening measures for elements have to be more carefully considered [1]. In this chapter a few joint typologies, used to connect masonry walls to different elements, are presented.

7.5.1 Traditional techniques

If existing floors are upgraded to behave as rigid diaphragms (e.g. by over-concreting), the newly created diaphragm has to be anchored into the masonry walls (Figure 20) against both vertical and horizontal forces. The same applies if new floor diaphragms are created in the building.

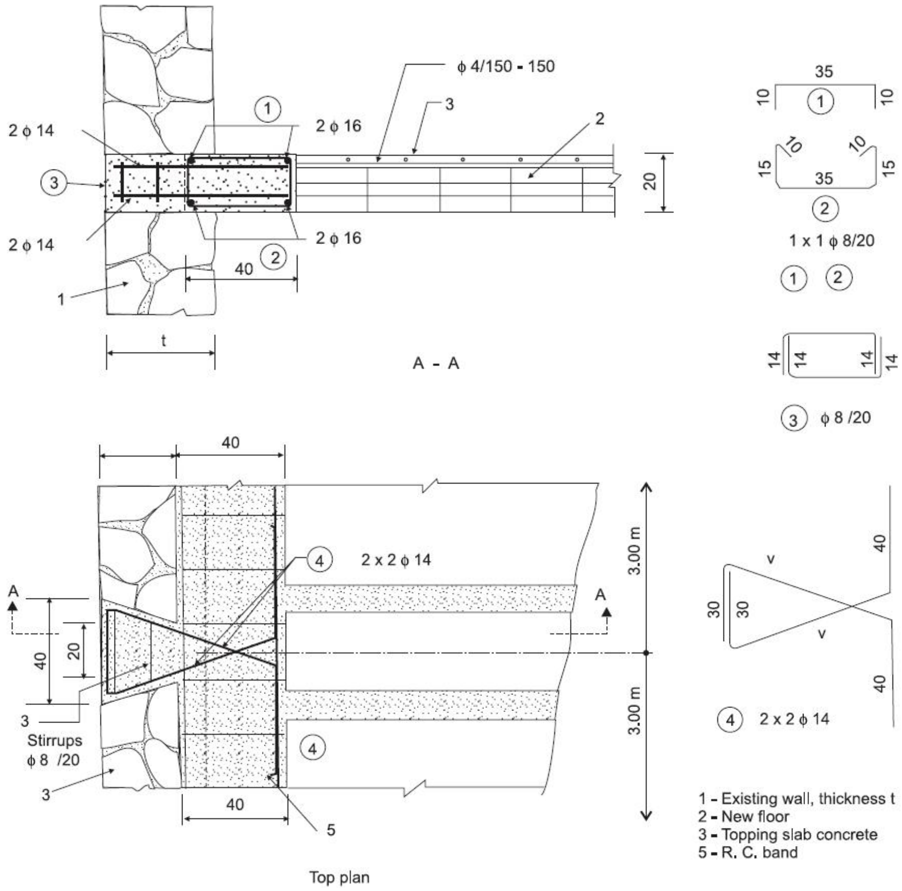

Figure 20. Integrate slab in masonry wall (efficient for new or existing slabs) [21]

Floor not behaving as horizontal diaphragms can affect the force distribution to the vertical load bearing elements. However, often the proper jointing of the existing floor, or of the over-concreting, to the masonry is difficult to be achieved. There are valid arguments against using heavier r.c. slabs instead of the wooden ones [4]. With proper justification, and based on detailed calculations, at least the detailing presented in Figure 21 should be provided for the floor.

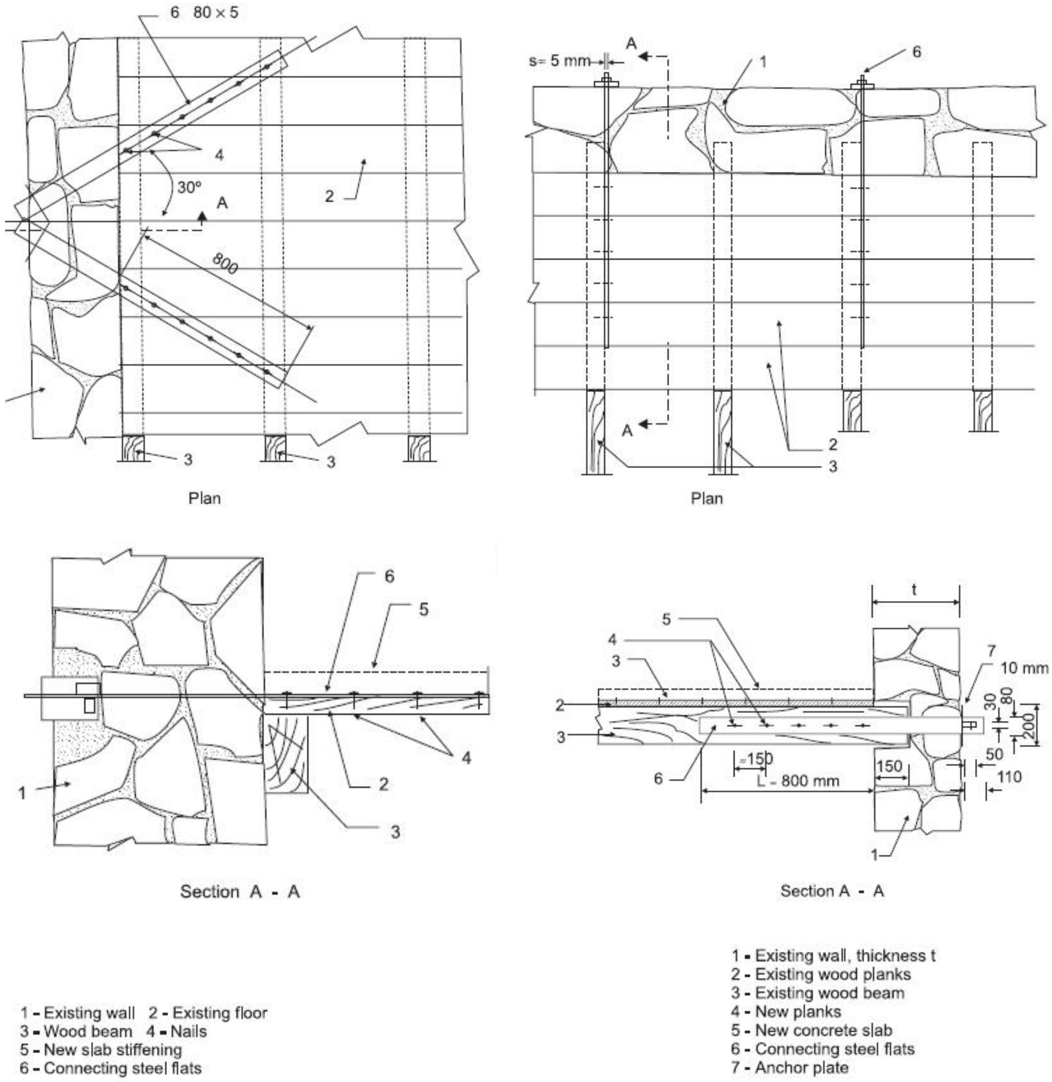

Figure 21. Anchoring of timber floor to masonry walls [21]

The jointing of new masonry to the existing walls should also be well detailed in order to avoid detaching of the two walls. In Figure 22 an example of recommended jointing is presented which uses a concrete column at the intersection of the walls.

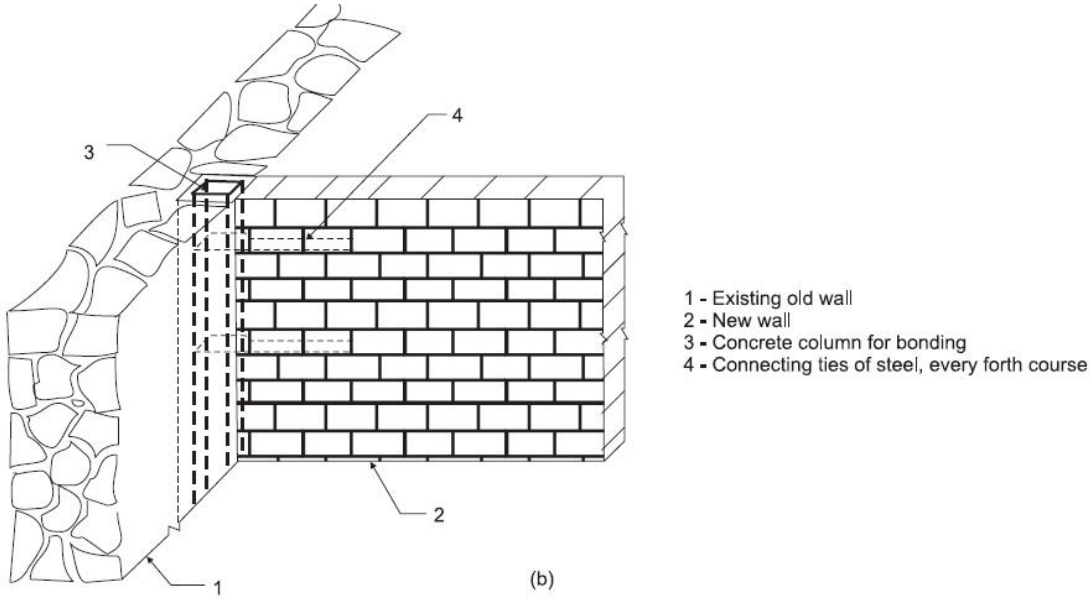

Figure 22. Anchoring of new to existing masonry wall [21]

7.5.2 Anchoring FRP fibers

Most of the time when spectacular increases of the wall strength were reported using FRP strengthening, one crucial aspect was the very strong tying/jointing of the FRP fibers to the elements supporting the walls. This aspect is exploited in tests, but in real work such strong tying arrangements can rarely be made on site.

As a rule, the FRPs should be properly anchored on all sides. It would be beneficial if the masonry wall itself were anchored to the foundation by steel rods and the multiple wythes anchored to each other by steel ties, possibly injection grouted and even RC cast between wythes if necessary. The steel components or polymer grids would add ductility to the structure. Of course, using multiple strengthening techniques increases the retrofitting cost severely, as many different devices and work teams are needed.

In some cases it may be necessary to add a steel, timber or RC beam on top of the wall to reinforce the connection of the wall to the floor or roof truss above. In many cases replacement may be necessary for the lateral strengthening of the floor or roof truss anyhow. In such cases, replacing a large beam is a laborious job and requires tearing down much of the existing structure, temporarily supporting of the roof and long interruptions in the use of the building. FRPs lose most of their competitiveness based on simplicity and rapidity in such cases.

A workaround would be taking the FRP over the weak existing roof/floor beam, truss, girder or joist and attaching it to the FRP on the other side. The bond of FRP to itself is very good.

The anchoring of the FRP fabric to the foundation can also be done in many ways. Adding a RC or steel beam can be feasible. A more discrete way of anchoring is to insert the FRP into a slit cut to the foundation and then fill the slit with epoxy. Or one can break the skirts of the fabrics to bundles of fibers and take these bundles through holes drilled to the foundation, spread the fibers below the foundation and attach them to the underside of the foundation (provided that there is room for this); or one can even take these bundles through the foundation one more time, spread the fiber bundles against the FRP on the other side of the wall and epoxy them to it.

In shear strengthening of walls it is possible to increase strength 10 times by full face covering with several layers of FRP only if the wall and FRP are both anchored well at top and bottom of the wall to prevent rocking and base sliding. Of course a wall retrofitted this way is much stiffer than the original wall.

If the top and bottom are not reinforced (by for example steel dowels and FRP anchoring), the strength is limited by sliding shear strength (Figure 1) of the wall at the base. Without the top and bottom strengthening and anchoring the strength increase obtained have been at maximum about 120 %. Promising ways to improve the shear strength of the mortar joints are to add carbon fibers or latex to the mortar. This will enable even larger strength gains. Adding steel dowels will improve ductility.

The aim of this type of retrofitting is to create a solid, moderately stiff structural element (wall) that acts as an integral part of the three-dimensional, well-anchored, multimaterial frame the load bearing elements of the whole structure form together. The failure mode of the wall should be distributed cracking and crushing.

8 Discussion

The comparison of different strengthening methods for URM is not an easy task. The goal of the strengthening is varied between studies and has not always been stated out very clearly. In addition, some tests have been conducted with monotonic loading, others with cyclic or dynamic loading. In some cases vertical and horizontal loads were simultaneously applied, in other cases tests was conducted under horizontal loads only, or by diagonal compression.

Many walls have been tested with monotonic loading. In earthquake, the members, or parts of members, are loaded in tension and compression in several reversals. This is especially problematic for FRP strengthening, as the FRPs have a tendency to debond from the masonry and buckle, leading to premature fiber rupture or anchorage failure. Thus monotonic loading may give too optimistic values for the capacities.

Some tests have been conducted using static cyclic loading, usually a rather fatigue-type cycling, with relatively small increases in amplitude between successive cycles. This does not represent typical loading in strong earthquake, where the failures often occur after only a few cycles with very large displacement demand. Comparison of the results of ElGawady [28] from dynamic tests and static-cyclic tests gives comparable values for the shear strength, but the debonding observed in dynamic tests was not observed in the static cyclic tests.

Perhaps a reasonable way of testing retrofitted walls for in-plane loads could be static cyclic tests involving larger steps in amplitude between cycles. For out-of-plane bending it is difficult to create a good static cyclic test setup. Particular attention has also to be paid to the effect of the vertical load acting simultaneously with the horizontal one.

8.1 In-plane strengthening

The in-plane strengthening of an URM wall can be deliberately aimed at changing the failure mode from diagonal shear failure to in-plane flexural failure (rocking) that involves alternating uplift and crushing of the corner (toe). Rocking supplies large horizontal deformation capacity and, provided that crushing of the toe occurs in relatively small area, it does not lead to sudden loss of capacity like diagonal shear failure. It is also a self-centering failure mode, i.e. after the excitation stopped the structure has the tendency to return to the original position - which vastly reduces the cost of future restoring work.

If needed, good ways to strengthen the toes are adding extra FRP layers or steel reinforcement to the toe regions. Another way to protect the corners against crushing might be adding relatively stiff rubber pads under the corners or replacing the toe parts of the masonry by some more flexible material. There is also a possibility to exploit the corner deformation in order to insert in the wall supplementary damping or yielding devices/materials.

In “real life” retrofitting, unlike in laboratory tests, the foundation is often weak and does not allow proper anchoring of the FRP, the shear strength of the lowest mortar joints often governs the strength of the wall. The strength of the mortar joints may be improved by replacing the old mortar by mortar with added carbon fibers or latex. Also conventional grouting methods can be used.

In FRP retrofitting of masonry walls thinking in three dimensions is essential. If the retrofitting increases shear strength only, without adding deformation capacity and energy dissipation, it may be disadvantageous. A sudden, brittle, failure of FRP strands, subjected to high tension, causes fast load redistribution to adjacent structures and may trigger collapse; or falling of loose material released in the sudden failure may cause injuries or deaths.

The suppression of the shear failure with a system (e.g. vertical FRP strips, vertical post tensioning with steel rods) that causes large tensile stresses may reduce the capacity of the wall against rocking or sliding. Some retrofits (e.g. horizontal FRP strips) are aimed at forcing rocking or sliding instead of shear failure, because these failure modes occur at higher loads than shear failure and provide displacement capacity. One must remember though, that one rocking or sliding wall will cause large bending or even torque loads to the adjacent walls. Together with the vertical cracks that create near the corners, these may lead to out of plane bending or tilting failure of the adjacent walls, or to out of plane failure of the rocking wall. These failures may again be rather sudden and cause injuries or deaths.

The research - and to some extent also design - seems to have been too much strength oriented, at the cost of ductility and energy dissipation; and too much one wall oriented, causing the danger of breaking the adjacent structures. Also, the larger load attracted due to the stiffness increase of the strengthened wall has often been neglected. Even adding FRPs to a masonry wall does increase stiffness. Extensive use of UD CFRP designs may alter the stiffness distribution, and thus load distribution, of the building considerably. Often it might be advantageous to use less stiff and cheaper glass fibers installed as a full face bidirectional fabric instead of UD carbon fibers in X configuration or rectangular grid. One could also consider woven fabrics instead of stitched ones, because they damage in a less brittle way, although they are weaker (and less stiff) than stitched fabrics.

The essentiality of good ties in the corners between walls and between a wall and floor/roof cannot be overemphasized.

8.2 Out-of-plane strengthening

In tests of flexural/out-of-plane strengthening it is possible to achieve even 20 times the original strength if the test is made on a slender wall (large height-to-thickness ratio) simply supported at the ends (representing top and bottom of the vertical wall able to rotate). This is natural as adding, for example FRP strips, makes the wall behave like a sandwich beam with stiff skin taking the tensile stresses and masonry core taking the shear stresses. The mechanism is similar with external steel reinforcement or post tensioning.

If the test is made with constrained ends (top and bottom of the vertical wall not able to rotate) and/or the height-to-thickness ratio of the wall is small, the gains obtained with such external strengthening are much smaller, because the bending strength of the URM wall is much larger (than in the simply supported case) due to the arching effect.

In tests by Galati *et al.* [37] two similar walls with two similar retrofitting gave 325% bending strength increase as simply supported but only 25% increase as end-restrained. It is a very good question which boundary condition is closer to reality in each case.

9 Conclusion - Possible focus of STEELRETRO

Undoubtedly, the strongest competition to steel, in URM rehabilitation, is given by FRP materials. These materials are:

- very versatile,
- high-strength,
- light,
- non-corrosive and
- relatively easy to apply on URM.

However they have several disadvantages too:

- they are expensive compared to steel,
- non-ductile,
- have smaller modulus of elasticity, at cracking strain of the masonry the stress level in the FRP is very low, compared to the strength of the material,
- sensitivity to temperature (some epoxy starts softening at 45-70ºC)
- and very often their anchoring creates serious problems, with the need to bring in, alongside FRPs, more classical techniques like RC foundation or roof beams etc. In these cases, the advantages of easily applying FRP are lost, due to the heavy interventions needed for the anchoring.

The most important first step of URM strengthening is to decide on the strengthening strategy. What is the goal of the intervention? Increasing strength of the wall? Increasing displacement capacity of the wall? 3D tying together of the building? Ensuring floor diaphragm, and tying it to the walls? Once this decision is taken, competitive steel solution for the achieving of these goals can be found. Particularly, for many strengthening techniques using FRP methods one can find steel equivalents.

One such case is NSM FRPs. This method has the advantage that the plastering of the wall does not have to be removed completely; unlike in case of using bidirectional FRP laminates. Essentially, FRP laminates or ropes are glued in slots cut in the masonry. Steel strands or cold-formed strips could be used as NSM with several obvious advantages over FRP: (i) larger elastic modulus, resulting in increased cracking strength of the masonry, (ii) larger deformation capacity in the cracked state, due to plastic deformability of the steel, (iii) possibility to increase adherence to the masonry by using profiled steel strips and (iv) significantly reduced cost.

One of the major weaknesses of URM is the complete lack of tensile strength of the material. Therefore, the intervention usually consists of providing this missing tensile strength. Based on data from [44], Figure 23 shows the “price” to provide tensile strength with different materials. As it can be observed, steel is very competitive both in the low (Group I) and high (Group II) tensile requirement regions. FRPs compensate with lower (i) installation costs and (ii) intervention times (i.e. reducing interruption of the use of the building).

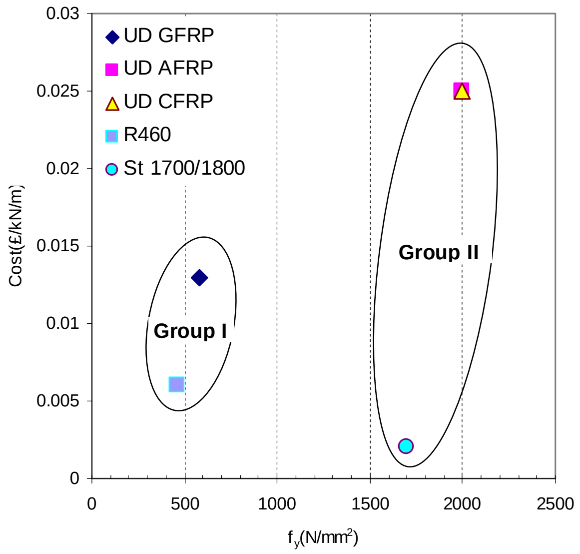

Figure 23. Costs of providing tensile strength in UMR strengthening work

Most probably, these two costs are more significant in rehabilitation work, and less important in strengthening work. Therefore, FRPs are probably more competitive in rehabilitation interventions, and steel can be more competitive in strengthening.

A comparison, of the different methods is presented in the following tables. The criteria used are:

- The availability of the rehabilitation method for different configurations of masonry, and the major limitations in use of each method (Table 4)
- The main failure mechanism affected/improved by the rehabilitation method, representing the "goal" of intervention. Very often secondary/unintended mechanisms are also acting. They are also mentioned. (Table 5)
- The possible performance improvements, in terms of strength gain, stiffness gain and energy dissipation increase, reported in the reviewed literature. In the same Table 6, a comparison of the economic impact of each intervention is represented. The economic aspects of the different methods can only be quantified in concrete case studies. It is also probable that in some cases one, in other cases another method will be more economical. In Table 6 therefore, only a qualitative assessment of the economic impact can be given.

Table 4. Suitability for wall typologies and main limitations of rehabilitation method.

	Single leaf walls	Cavity walls with rubble filled core	Bonded brick-work	Stone masonry walls	Light-weight CMU units	Concrete block walls	Brick column	Stone column	Joints	Applicability on irregular or rough surfaces	Applicability with weak adjacent material	Visibility for workmanship quality control	Chemical and environmental durability	Fire safety	Aesthetic change
Ferro-cement	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	A	A	G	IM	G	S
Shotcrete	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	A	SC	G	IM	G	S
Reinforced plaster	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	NA	SC	G	IM	IM	S
Grout injection	Yes	Yes	Yes	-	Yes	Yes	Yes	-	-	A	A	P	IM	G	-
Diagonal steel strips	Yes	Yes	Yes	-	-	Yes	-	-	Yes	SC	NA	G	IM	P	M*
Rectangular mesh of steel strips	Yes	Yes	Yes	-	-	Yes	Yes	Yes	Yes	NA	SC	G	IM	P	M*
3D steel tying	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	A	SC	G	P	P	M
RC tie columns and beams	Yes	Yes	Yes	Yes	Yes	Yes	-	-	Yes	A	SC	G	G	G	S
Centre core reinforcement	Yes	-	Yes	Yes	Yes	Yes	Yes	-	-	A	A	P	G	G	-
Internal post-tensioning	Yes	-	-	Yes	SC	Yes	Yes	-	-	A	A	P	G	IM	-
External post-tensioning	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	A	SC	G	P	P	M
UD FRP in X	Yes	Yes	Yes	-	Yes	Yes	-	-	Yes	NA	NA	G	G	P	M*
UD FRP rectangular grids	Yes	Yes	Yes	-	Yes	Yes	-	-	Yes	NA	SC	P	G	P	M*
BiDir FRP laminate	Yes	Yes	Yes	-	Yes	Yes	Yes	-	Yes	NA	A	P	G	P	M*
NSM FRP	Yes	Yes	Yes	-	Yes	Yes	Yes	-	Yes	A	SC	IM	G	IM	S
Toe confinement	Yes	Yes	Yes	-	Yes	Yes	-	-	-	SC	SC	P	G	IM	-
TRM	Yes	Yes	Yes	-	Yes	Yes	Yes	-	-	A	A	IM	IM	G	-
Polymer grid	Yes	Yes	-	-	Yes	Yes	Yes	-	Yes	A	A	P	G	IM	-

NOTES:

NOTES:	Yes	Possible to use the method for both restoration and strengthening.
	Int	Only on the interior surface of the wall
	*	If the wall had plastering which can be remade than S or "-"

A – Applicable;	NA – Not Applicable;	SC – Special Care
G – Good;	IM – Intermediate;	P – Poor
M – Major;	S – Small;	- – None

Table 5. Failure mechanism improved by the rehabilitation method

	Sliding shear	Oblique shear		Bending			Overall confining of the masonry	Connect layers in multi-wythe wall	3D interaction
		In mortar	In stones	Corner uplift, Anchoring	Tension cracking	Toe crushing			
Ferro-cement	Yes	Prim	Prim	If mesh is anchored	Yes	Yes	Yes	-	Yes
Shotcrete	Yes	Prim	Prim	-	Yes	Yes	Yes	-	Yes
Reinforced plaster	Yes	Prim	Prim	If mesh is anchored	Yes	Yes	Yes	-	Yes
Grout injection	Prim	Prim	-	-	-	Yes	-	Yes	-
Diagonal steel strips	Yes	Prim	Prim	If anchored	-	-	-	-	-
Rectangular mesh of steel strips	Yes if anchored	Yes	Yes	Yes if anchored	Yes	-	Yes	-	-
3D steel tying	Yes	Yes	Yes	Yes	Yes	-	Yes	Prim	Prim
RC tie columns and beams	Yes	Yes	Yes	Prim	Yes	Prim	Prim	Yes	Yes
Centre core reinforcement	-	Yes	Yes	Prim	Prim	Yes	-	-	-
External post-tensioning	-	Yes	Yes	Prim	Prim	-	-	-	Prim
Internal post-tensioning	Yes	Yes	Yes	Prim	Prim	-	-	-	-
UD FRP in X	Yes if anchored	Prim	Prim	Yes if anchored	-	-	-	-	-
UD FRP rectangular grids	Yes if anchored	Prim	Prim	Yes if anchored	Yes	-	Yes	-	-
BiDir FRP laminate	Yes if anchored	Prim	Prim	Yes if anchored	Yes	-	Yes	-	If connected
NSM FRP horizontal	-	Prim	Prim	-	Yes	-	-	-	-
NSM FRP vertical	Yes if anchored	Yes	Yes	Prim	Prim	-	-	-	-
Toe confinement	-	-	-	-	-	Prim	-	-	-
TRM	Yes if anchored	Prim	Prim	-	Yes	Yes	Yes	-	-
Polymer grid	-	Yes	Yes	-	-	Yes	Yes	-	Yes

NOTES: Prim – Primary goal of the intervention/failure mode mainly affected
Yes – Strength in this failure mode also improved, even if it was not primary goal of the intervention

Table 6. Possible performance benefits and economic consequences of the use of different rehabilitation techniques

	Strength increase (%)	Stiffness increase (%)	Ductility increase (%)	Energy dissipation increase (factor)	Space use	Tearing down and rebuilding parts	Effect on existing finishing	Material cost	Work amount
Ferro-cement	50	I	I	I	IM	S**	L	IM	L
Shotcrete	200-500	I	-	-	L	S	L	IM	IM
Reinforced plaster	25-200	-	I	I	-	S**	L	S	L
Grout injection	0-40	10-20	-	-	-	S	-	IM	L
Diagonal steel strips	350-900	-	I	I	-	S**	IM	S	IM
Rectangular mesh of steel strips	40-90	-	I	-	-	S**	L	IM	L
3D steel tying	15-50	-	I	30-60	-	-	IM	S	IM
RC tie columns and beams	20-50	I	50	50	L*	L	IM	L	L
Centre core reinforcement	100	I	I	-	-	S	-	L	L
Internal post-tensioning	I	I	I	-	-	S	S	S	L
External post-tensioning	I	-	I	-	S	-	IM	S	S
UD FRP in X	50-200/800 (if FRP anchored)	I	50-100	I	-	S**	IM*	S	S
UD FRP rectangular grids	10-50	I	80	170	-	S**	L	S	IM
BiDir FRP laminate	100-1000	I	20-1000	I	-	S**	L	IM	IM
NSM FRP	10-80	-	45	35	-	-	S	S	S
Toe confinement	-	-	-	-	-	-	S	S	S
TRM	-	-	-	-	IM	S	L	IM	IM
Polymer grid	-	-	-	-	-	L	-	S	L

NOTES: I – Increased but values not reported in literature
L – Large; IM – intermediate; S – small - – not relevant
* If used externally. The value is "-" if the r.c. elements are placed within the wall.
** Replacement of the plaster.

Abbreviations

AFRP	–	aramid-fiber reinforced polymer
CFRP	–	carbon-fiber reinforced polymer
CMB	–	clay-brick masonry
CMU	–	concrete masonry unit
FRP	–	fiber reinforced polymer
GFRP	–	glass-fiber reinforced polymer
HLW	–	height × length × width
H/L	–	in-plane aspect ratio, height/length of the wall
H/W	–	out-of-plane aspect ratio, slenderness, height/width of the wall
NSM	–	near surface mounted
NSM-FRP	–	near surface mounted fiber reinforced polymer
PC	–	Portland cement
PVAFRP	–	PolyVinylAlcohol-fiber reinforced polymer
TMR	–	textile reinforced mortar
URM	–	un-reinforced masonry
UD	–	uni-directional

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APPENDIX 1
DOCUMENT RECORD

DOCUMENT RECORD	
Document no. VTT-R-11115-07	Constructive and performance analysis of the retrofit systems for vertical masonry elements

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