Deliverable 3.2

Critical review of retrofitting and reinforcement techniques related to possible failure

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WORKPACKAGE 3: Damage based selection of technologies

Leader: POLIMI

PROJECT N°: 244123

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TITLE: New integrated knowledge based approaches to the protection **of cultural heritage from earthquake-induced risk**

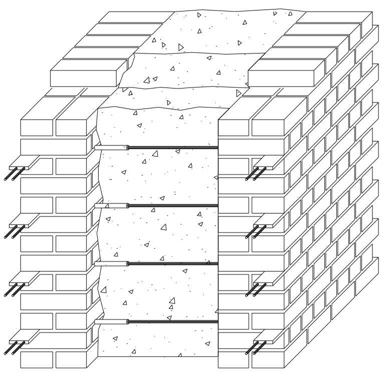
COORDINATOR: Università di Padova (Italy)

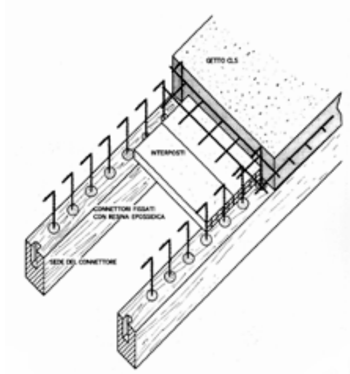
START DATE: 01 January 2010 DURATION: 36 months

INSTRUMENT: Collaborative Project

Small or medium scale focused research project

THEME: Environment (including Climate Change)





Dissemination level: PU

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1 INTRODUCTION

1.1 DESCRIPTION AND OBJECTIVES OF THE WORKPACKAGE

WP3, in general, is aimed to the collection of information for increasing the existing state of knowledge linking earthquake induced failure mechanisms, construction types and materials, interventions, assessment techniques. According to the project document, other aims of the WP are the follow:

- Development of concepts for materials and intervention techniques based on the structured database; definition of the main design parameters and requirements for materials and intervention techniques;
- Definition of the main on site control techniques and strategies;
- Development of advanced materials and improved techniques for intervention and to produce/assemble those required for testing and case studies;
- Development of laboratory procedures and choice of parameters for the final validation of the durability, compatibility, and effectiveness of new techniques and materials;
- Parameterization of all the above mentioned information to set the basis of optimized design and required laboratory testing in following WPs.

1.2 OBJECTIVE AND STRUCTURE OF THE DELIVERABLE

The deliverable critically reviews the main retrofitting and reinforcement techniques available in the state-of-art, related to possible failure mechanisms and requirements, described in the Deliverable D3.1.

This project proposal tackles the problem starting from the basic consideration that the best guarantee of compatibility, low intrusiveness, removability/reversibility, etc..., is essentially based on a ‘minimum intervention’ approach. This means that interventions, utilizing materials and components with different properties compared to the original substance and/or implying significant changes of the original local/global structural behaviour, but also physical and chemical properties, should be avoided as much as possible. This also means that, rather than assertively assuming confusing concepts, we should provide the technical data to define performance levels. The definition of the minimum required intervention depends in fact on the local seismicity, on the state of the structure, on its importance (in terms of cultural and artistic value), on the level of damages that can be allowed, on the use of the building.

The problems arising here are related to analyses performed on the basis of limited information regarding the original structural system; the use of unsuitable analytical tools; the adoption of behaviour models developed for modern structures. Furthermore, very often, dated design methods, which do not reflect the actual strengthening technique as they are based on rule-of-thumb and/or odd material properties, are applied.

The deliverables introduces the general problems of the intervention on the historic heritage, and the fundamental criteria given by international organisation for the preservation of the historic procedures.

The repair techniques are analysed on the base of the hierarchy from the single panel or structural elements, to the whole structure. Where available, information about possible mistakes, problems or documented seismic performances are given.

More in detail, the report starts from the intervention of a single wall, describing the main techniques aimed at a general strengthening, a localise repair of cracks or damaged area, to prevent overturning. Some techniques could be applied for more then a purpose, changing only the extension on the structure.

After this, the connection between walls and walls/floors/roofs are considered. It is one of the main aspects to take in consideration during the building strengthening process, being at the base of an effective seismic behaviour.

Vaults and floors are then considered.

For masonry vaults, local repair techniques and criteria could be the same of masonry panels.

2 GENERAL REMARKS

The experience of the last decades in repair, strengthening and prevention for the preservation of masonry buildings in historic centres of seismic areas did teach that compatible or friendly techniques have to be chosen in order to obtain positive results. It is also rather clear that too strict theoretical position on the preservation of the Cultural Patrimony as a historic document of the past cannot be always practically applicable.

Many experiences in restoration and rehabilitation of damaged masonry buildings have been carried out in Europe during more than fifty years; in a high number of cases the interventions were performed after a long period of misuse and lack of maintenance following the Second World War. During these fifty years period also major earthquakes have taken place in most of the Mediterranean countries.

Several unsuccessful results have underscored the need for adequate assessment prior to any restoration or rehabilitation. In fact, when neither the real state of damage nor the effectiveness of repairs is known, the results of the intervention are also unpredictable. This was clearly shown by some repair failure even when advanced materials had been used; there are now enough information to support the choice of compatible materials and techniques for repair.

If that is the most important point, then a deep knowledge is needed of the: (i) building history and evolution, (ii) geometry, (iii) structural details, (iv) crack pattern and material decay, (v) wall construction technique and materials, (vi) material properties, (vii) structure stability. This knowledge can be reached through on site and laboratory experimental investigation, structural analysis using appropriate models and final diagnosis.

Only then and knowing the functional destination, the design for repair should be set up, remembering that there is not a unique way of repairing, consolidating, and preventing. Several techniques can be discussed and the optimal one chosen from the point of view of the most friendly intervention among the ones economically compatible with the available budget, but also respecting the safety necessities for the building. Also, a project for a maintenance program to be applied in the long term should be set up, since no intervention can last forever.

It has already been stressed out that the conservation will be successful only if it is respectful of the existing buildings. This affirmation means first of all that the new function assumed for the building should be compatible with its performance; in fact, if the new function requires the resistance to much higher state of stresses than originally, an invasive strengthening is usually required. But the affirmation also means that the new materials used for repair and strengthening will be chemically, physically and mechanically compatible. Nevertheless, it is impossible to ask that they have to be exactly the same as the original ones. In fact it will be never possible to find after centuries mortars, bricks, stones, timbers equal to the original ones, even if they come from the same place or quarry. As a consequence of that, repair and improvement will be better than substitution. Today there is frequently reference made to the fact that in the past the architect frequently inserted new parts in the existing buildings, (Bellini, 1986) and (Carbonara, 1997). Nevertheless we should remember that for centuries the same types of materials and structural elements were used; in practice, the traditional use of materials and techniques was respecting compatibility.

In fact, their chemical, physical and mechanical differences were not as large as the ones between masonry and reinforced concrete, FRP or organic resins, lime mortars and cement mortars, etc... Today the need for rehabilitation and repair of damaged masonry buildings allows for the application and experimentation of both traditional and advanced techniques; the latter are frequently used at first without previous control, sometimes because of shortage of time.

2.1 REPAIR CRITERIA

2.1.1 General - Selection of optimal solutions

Intervention in heritage structures, involving either stabilization, repair or strengthening, (in particular, seismic retrofitting), should be subjected to a series of requirements or criteria oriented to ascertain the efficiency of the solution together with its compliance with recognized conservation principles. These principles are stated in international documents such as the Venice Charter of 1964 and, in a more specific way, in the ICOMOS / ISCARSAH Recommendations for the Analysis and Restoration of Architectural Heritage, (ICOMOS/ ISCARSAH, 2005), and the Annex on Heritage Structures of ISO/FDIS 13822, (ISO/TC96/SC2, 2010).

These criteria should not be understood as absolute requirements, but as recommended conditions assisting in the definition of optimal repair or strengthening solutions. In fact, complying with all these criteria may be impossible in some cases, and some prioritization or choice, based on engineering judgment, is often necessary. Trying to satisfy these conditions will assist in conceiving and designing both efficient and respectful interventions consistent with conservation principles.

Respect for structural authenticity

Monuments are not only interesting because of the value of their artistic content or geometrical conception. Monuments are also interesting and valuable because they constitute a structural achievement and provide an immediate and tangible experience on past construction technologies. Structures of monuments do not only constitute a document; they are in fact living legacies which, centuries after their construction, still carry out their resisting mission and keep on enduring loads, wind and earthquakes; they are a living and persistent proof of the skills of their creators and builders.

Proper restoration of monuments must focus on preserving the original features of the structure. If repair or strengthening works are needed, they should cause the minimum possible alteration. This is not only applicable to the geometry and materials: the authenticity of the mechanical and resisting principles governing the structural response (the nature of the structure and its resisting mechanisms) is also to be preserved to the possible extent.

Structural reliability requirements

A frequent reason for strengthening and seismic retrofitting is to achieve public safety, with various levels of structure and material survivability determined by considerations related to the importance of the buildings. The goal of public safety is to protect human life, ensuring that the structure will not collapse upon its occupants. Conventionally, it is required or accepted that structures of high cultural significance should be upgraded to remain unaffected (undamaged) by possible earthquakes. It must be noted, however, that this requirement may often lead to very impacting an invasive upgrading measures causing a significant loss in terms of cultural heritage. In some cases, the cultural loss meant by the upgrading might equate or surpass that of the damage caused by the earthquake. While the requirement of structure survivability seems obvious or reasonable for cultural heritage buildings, the requirement for non-damageable structure may be excessive and lead to counter-productive actions. The extent of seismic upgrading in heritage constructions needs to be carefully considered in every individual case based on a cost-benefit analysis which takes into account the cultural losses conveyed by the upgrading itself.

In the case of valuable monuments, seismic upgrading leading to non-damageable structure may be also considered to reduce the material or artistic losses (ultimately, cultural losses) that the building could experience due to an earthquake.

Minimal intervention

Interventions causing only a reduced impact on the original structure should be preferred, provided that they are enough to warrant the required safety level. Among possible solutions, all of them providing the required level of safety, the one causing minimal alteration (the minimum intervention) should be preferred.

From an engineering point of view, the optimal solution is obtained after envisaging and analysing a series of alternative solutions. Reaching an "optimum" solution requires, as a first step, conceiving and tentatively developing a set of alternative solutions. Similarly, and as a first step, reaching an "optimum" intervention requires to the engineer or architect to foresee and develop a set of alternative possibilities involving different strategies, techniques or materials. Each different solution should then be evaluated regarding both its structural efficiency and compliance with conservation criteria (or, in other words, its cost in terms of loss of authenticity and cultural value). According to the ISO/FDIS 13822 final draft, the solution finally adopted should consists of a "minimal intervention", defined as "an intervention that balances the safety requirements with the protection of character-defining elements, ensuring the least harm to heritage values".

The possible impact of an intervention on a monument or building, in terms of loss or alteration of the original material and structural features, must always be investigated and quantified. As stated by the ICOMOS/ISCARSAH Recommendations, no action should be undertaken without ascertaining the likely benefit and harm to the architectural heritage. Comparing benefit and cost will permit the evaluation of the different solutions and the selection of the optimum one.

Compatibility

The ICOMOS/Iscarsah Recommendations clearly state that "The characteristics of materials used in restoration work (in particular new materials) and their compatibility with existing materials should be fully established (as for example to avoid risk of negative chemical reaction, etc...). In any case, it has to be clear that compatibility is a necessary condition but not sufficient to accept a product because its benefit has to be demonstrated. This must include long-term effects, so that undesirable side effects are avoided".

The materials and the technical devices used for repair or strengthening must be compatible with the original ones, in the sense that no undesirable side-effect should result from their physical or mechanical combination. Compatibility problems may be related to chemical, physical, mechanical, thermal and rheological phenomena, among other. Ancient materials should not experience any form of chemical deterioration when in contact with the new materials or with substances delivered by them (chemical compatibility). New materials or mechanical devices should not deform too differently from the original ones when subjected to environmental thermal variations (thermal compatibility). Repair materials or strengthening devices must have stiffness similar to that of the original material when embedded or externally attached to the latter, again to prevent cracking or other mechanical damage due to external loading (mechanical compatibility). New materials should not experience any deformation or flow causing imposed deformation and hence damage on the original ones (rheological compatibility). For instance, Portland cements may free soluble salts which, after penetrating lime mortars or stone, may experience expansive crystallization and cause cracking (chemical incompatibility). Moreover, the shrinkage of Portland cement or concrete, or their thermal deformation, may cause cracks to stone or brick masonry attached to it (rheological or thermal incompatibly). A mass of very stiff repair material inserted within the existing one may cause the latter to crack or crush due to the application of additional loading (mechanical incompatibility).

Durability

Conservation and restoration aim at significantly enlarge the life expectancy of heritage structures. Hence, the repair materials or strengthening mechanical devices used must be satisfactorily durable. Both the overall safety of the structure and the durability of the original parts can be compromised by the decay of new repair material. According to the ICOMOS/Iscarsah Recommendations, no action should be undertaken without ascertaining the likely benefit and harm to the architectural heritage and long terms side effects.

Non-intrusiveness (non-invasivity)

Non-intrusive (or non-invasive) repair or strengthening techniques should be preferred to more invasive alternatives. They will, for obvious reasons, contribute to preserve the material integrity of the existing structures. Among possible alternatives, preference should be given to the least invasive one. As mentioned by the ICOMOS/Iscarsah Recommendations, “the choice between “traditional” and “innovative” techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, consistent with the need for safety and durability”.

Non-obtrusiveness

Obtrusiveness refers to the quality of being undesirably noticeable. The Venice Charter for the Conservation and Restoration of Monuments and Sites, 1964) states that “replacements of missing parts must integrate harmoniously with the whole, but at the same time must be distinguishable from the original so that restoration does not falsify the artistic or historic evidence. Additions cannot be allowed except in so far as they do not detract from the interesting parts of the building, its traditional setting, the balance of its composition and its relation with its surroundings”.

According to this understanding, any additional structural device included as part of a strengthening action must integrate harmoniously with the existing structure and should not cause a significant alteration of its initial aspect. It should, however, be distinguishable from the original parts or materials.

Removability

Where possible, any measures adopted should be reversible, or at least removable, so that they can be replaced with more suitable measures if new knowledge is acquired. In any case, interventions should not compromise possible later interventions.

In other words, it must be possible to dismantle any intervention without leaving any severe damage or producing any significant lasting alteration or deterioration to the original material and structure.

Nowadays, full reversibility (meaning that no any deterioration or impact, even if small, should be caused by the removal of the former intervention) is regarded as a too demanding and unrealistic requirement. Removability (or dismantlability) is considered at present as a more realistic and viable condition than full reversibility. Reversibility or removability leave open the possibility of eventually replacing the strengthening by another more adequate or effective one.

Monitorability and controllability

Repair or strengthening measures whose performance and effect on the building are impossible to control should not be allowed. In order to carry out such control, monitoring should be applied after, during and after the execution of the repair or strengthening. The intervention must be designed in such a way that monitoring can be implemented and successfully used during and after the execution. Any proposal for intervention must be accompanied by a programme of monitoring and control.

In the case of provisory or emergency actions, the monitoring of the provisional strengthening is of large importance. It will normally be important to know whether the strengthening (for instance, a propping system) is actually working and resisting some load, and thus partly or totally relieving the original structure, or whether the strengthening has not been in fact mobilized. This will lead to very different decisions regarding the new strengthening system and the way to implement it.

2.2 COMPATIBILITY OF THE INTERVENTION

2.2.1 Traditional technique

Repair materials should respect the requirements mentioned in Section 2.1, with emphasis in the compatibility (chemical, physical, mechanical, etc…) with the original materials and their durability.

As aforementioned, an important requirement to be considered in the selection of any material or technology used for repair and strengthening lays on the needed compatibility (chemical, physical, mechanical, thermal, rheological, etc...) between the newly added and the original parts.

A critical choice, regarding compatibility, is found in the use of traditional materials and techniques against modern (or innovative) ones. The first ones are normally compatible to the original parts due to the combination of similar properties. Moreover, the compatibility in the long term, and the absence undesirable side-effects, has been proven through an experience of centuries, if not millennia. Ancient or traditional materials mortars, such as lime mortar, have already proven their durability and compatibility with other historical materials across long periods of time.

These techniques use traditional methods, materials and tools. They are easy to implement, and can be carried out by companies of small size.

Another advantage of traditional or historical structures is found in the fact that, having been used historically to improve or strengthen many ancient structures (as in the case of ties), they can be regarded as a “historical” devices which, if implemented now in a heritage structure, do not severely impact on its original character and authenticity.

Some relevant ancient techniques, still used nowadays for repair and strengthening, are discussed below.

Local dismantling and reconstruction (scuci-cuci)

The existing masonry pattern is locally removed where major deterioration has occurred and it is replaced with new masonry reproducing closely the mechanical properties of the original one. Local dismantling and reconstruction contributes to preserving the mechanical efficiency and regaining the continuity in a masonry structure. Local reconstruction can be considered partially reversible, while fully compatible and durable thanks to the use of materials similar to the original ones.

Structural repointing

Consists of the partial, but deep removal of deteriorate mortar in bed joints and substitution with new mortar (possibly with better mechanical properties and durability). Repointing can be considered consistent with traditional/historical maintenance or repair practices. It is partially reversible (if the new mortar can still be removed from the joints) and satisfactorily compatible and durable, provided that an adequate lime mortar is used to substitute the original one.

Separation or debonding at the unit mortar interface (be the unit a brick or a stone block) does not constitute a severe damage and can be easily repaired by just filling the crack with new mortar. The continuous refilling of this type of cracks after soil settlements or other possible actions has been historically a common repair or maintenance practice. If properly executed, the so-repaired structure recovers its initial material continuity and structural performance. Moreover, the structure can be re-repaired used the same procedure. Cracks affecting bricks or stone blocks are more difficult to repair but can also be treated by means of traditional/historical or modern techniques. Masonry with damaged units has been traditionally repaired by substituting the affected material by new one.

Tying

Iron or steel bars, anchored with plates or other devices to the structure, have been successfully used in the past to improve the overall structural behavior by ensuring an adequate connection between structural elements. They are work have different practical applications all aiming at stabilizing or improving the connectivity between parts or subsystems of the structure. Tie bars are non-invasive and can be easily removed. They provide an efficient and durable tying action provided that their end anchorages are maintained in good condition.

Ties in dwellings, palaces, churches and towers have a traditional character in many countries and have been widely used in both seismic (as in Italy) and also non-seismic places (as in Northern Europe). However, when visible (as when placed across the springing or arches), their use for modern strengthening purposes in countries where they have not been traditionally utilized may pose some presentation problems because they may be sensed as “obtrusive” by people.

As observed in recent earthquakes, ties show significant mechanical compatibility with masonry structural systems. Ties contribute very satisfactorily to improve the seismic performance of buildings thanks to their massless and elastic (or flexible) character, as opposite to alternative solutions such as new stiff concrete floor slabs or roofs.

Intramural tying

Application of punctual confinement to the wall, either with transversal iron or steel bars, anchored to plates or other metal or timber devices on both sides of the wall. The technique contributes to control/avoid leaf separation in the case of walls composed of several leaves (such as three-leaf Roman walls), or to avoid or stabilize internal cracking in the case of walls with poor internal interlocking. If the holes are not injected or are injected within a confining socket, the technique can be considered mostly non-invasive and reversible.

Fastening

Different blocks or members are tied together by means of stiff devices such as pins and cramps or short ties. It was used in the past to tie together parts with poor connection or interlocking and to prevent from partial failure. Local tying is meant to develop a micro-continuity in the structure thus improving structural connectivity and strength. It constitutes a simple and effective technique, mostly reversible, allowing the increase of the resistance of the element. However, some solutions, such as cramps, may show limited mechanical efficiency while requiring significant maintenance as they tend to lose their mechanical efficiency due to thermal effects and plastic deformation.

Confinement of piers by means of stiff rings

Application of steel rings in critical sections of the pier with the aim of stabilize damaged material or improving the compressive strength, stiffness and ductility of the pier. It is a fully historical and traditional, fully non-invasive and reversible technique characterized by its high effectiveness. However, and while effective for repair and strengthening for gravity loads, this technique may not significantly increase the overall seismic response of a building.

Overall substitution of a structural member

The materials and technologies used to carry out material substitution can be similar to the original ones or, else, alternative solutions can be used to modify and improve the behaviour and mechanical properties of the structural member. A typical example is overall substitution of floors and roofs when the original ones have severely decayed in a building. The aim is normally at recovering the original function of the element, correct possible design defects or modify the seismic response. In the framework of conservation of historical monuments, it is generally agreed that repair, when possible, is preferable to substitution. However, in some countries (as in China and Japan), substitution is privileged over the maintenance of the deteriorated members or materials as a way to keeping alive historical or traditional construction techniques and skills. As mentioned by Annex on heritage structures of the final draft of the new ISO/FDIS 13822, “judgments about heritage value and authenticity can differ from culture to culture, and thus, there are no fixed criteria. In some geographical areas, keeping alive traditional construction practice is privileged over the conservation of original materials”.

Enlargement

Enlargement refers to the addition of new material (such as an additional leaf of new masonry) to an already existing member, with adequate connection or interlocking, in order to increase its section and hence mechanical capacity. Enlargement has been used traditionally to increase the load bearing capacity of walls or, by applying it on the intrados, to increase the capacity of vaults. Mechanical compatibility requires (1) the use of a not too different material, regarding stiffness and strength, with respect to the original one, and (2) a good connection between the original member and the added material. In turn, the removability of the enlargement depends on the possibility of dismantling the added parts without causing significant harm to the original material. Such removal will be normally possible (with limited damage on the original part) if stone or brick masonry, with limited interlocking, has been used for the enlargement.

Buttressing

It consists of the addition of massive elements made of concrete or masonry to laterally prop a structure. Buttresses resist lateral forces and deformations essentially thanks to their self weight. Buttresses contribute to prevent from failure mechanisms related with lateral deformation. Buttresses originally built as part of the entire original construction may be very efficient, as they are normally built in a homogeneous way with the rest of the structure (with the same type of masonry, well interconnected to the rest, while also sharing a unique foundation). Conversely, buttresses built as a later strengthening device, after the construction of the original parts, may show limited efficiency due to lack of satisfactory interlocking or differential settlements separating them from the rest. Furthermore, when the buttresses are built as later additions, the structure will need to deform to significant extent in order to mobilise the new buttress.

These problems may be overcome by building flying arches in between the structure and the new buttresses, as done in the case of some Byzantine churches and monasteries. Flying arches produce stabilizing forces, without the structure having to laterally deform, as soon as they are built. Moreover, these forces will persist in spite of possible differential soil settlements between the buttress and structure. External buttresses combined with flying arches may provide a very efficient way to increase the seismic capacity of the structure. Due to its external character, this solution will normally be fully non-invasive and removable.

Strutting

Placing struts between different parts of the structure or between the structure and an external system. Struts are members designed to resist a compressive load and are used to laterally prop a structure or structural member. Struts can work in horizontal, vertical or inclined position. Strutting can be used to stabilize damaged structures or elements risking collapse, or not able to carry out their load-bearing function. Inclined struts increase the lateral stiffness of the structure and are used to counteract the out-of-plane forces. Horizontal struts consisting of stone arches or timber beams are not uncommon as traditional or historical stabilizing elements. Struts can be considered non-invasive and fully reversible.

2.2.2 Modern techniques

Modern and innovative materials and techniques may be considered for repair and strengthening purposes provided that sufficient scientific research and experience are available on their adequate performance and lack of compatibility problems with the original material.

It shall be noted that modern materials and techniques show in some cases severe compatibility problems (as in the case of Portland cement or epoxy resins) when used to restore or strengthen brick or stone masonry structures. In other cases, problems may not have been identified, but not enough experience may have been gathered as to show that no damaging side effects will occur in the long term.

According to the Venice Charter, where traditional techniques prove inadequate, the consolidation of a monument can be achieved by the use of any modem technique for conservation and construction, the efficacy of which has been shown by scientific data and proved by experience. In turn, the ICOMOS/Iscarsah Recommendations mention that "the choice between "traditional" and "innovative" techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, consistent with the need for safety and durability".

A selection of modern and innovative techniques is discussed below with focus on possible advantages and disadvantages regarding compatibility and durability with original materials. Some of these techniques are presented, in spite of having shown frequent and important problems, because they have been widely used in the recent past.

Grout Injection of the cracks

Injection of fluid mortar or other adequate repair materials through cracks or holes previously drilled. The purpose is to fill cracks, existing cavities and internal voids. Injection improves the continuity of masonry and contributes to enhance the average mechanical properties of masonry. Injection is should only be carried out using injected materials with proven compatibility with the original material. To ascertain full chemical and mechanical compatibility, as well as satisfactory efficiency in improving the initial properties, a previous research (as for instance, involving some tests) is always necessary with regard to the characteristics of the material to be injected and its effect upon the existing walls.

Grouting and injection, when carried out using materials sufficiently similar and compatible to those existing in the structural members, should not be regarded as an "invasive" or "intrusive" technique.

External reinforcement

Application of high-performance materials on the exterior leaves of existing structural members, adequately connected by chemical or physical bonding, or mechanical anchors, with the purpose of increasing their strength capacity and stiffness. The reinforcing material may consists of reinforced concrete jacketing, reinforced plasters, external FRP laminates or sheets, or wood. When concrete or reinforced plasters are used, external reinforcement is normally impossible or very difficult to remove due to the need to connect the original and the added wythes.

The connection with the susbtrate is normally obtained with the use of epoxy resins, mortar and fasteners. An effective use of this technique by bonding requires some regularity in the masonry surface. In the case of seismic strengthening, it seems necessary to place external reinforcement on opposite sides and to properly connect both sides with ties. Reinforcement enhances the strength and stiffness of the structure by adding a material that can resist tension. In several cases, also ductility can be increased. External reinforcement is normally irreversible but non- or moderately-invasive. External reinforcement is normally irreversible, and hardly removable, as removing the added material from the wall will normally cause the peeling off of the brick or stone surfaces. External reinforcement is also normally obtrusive as it requires hiding the original masonry and paraments behind the new material.

Internal reinforcement

It consists of the insertion of a high-performance material with large tensile strength within perforations produced in the original masonry material. Reinforcement is normally provided by means of steel or FRP bars. Filling the perforations with an adequate mortar is essential in order to provide sufficient bonding between the reinforcing bars and the surrounding material. Internal reinforcement is intrinsically invasive and non-reversible, as it will normally be impossible to extract the bars after the hardening of the injected mortar. In addition, compatibility and durability problems may arise depending of the type of material used for the reinforcement and the type of mortar. For instance, conventional steel and Portland cement, used in the past for this purpose, may produce significant problems due to limited durability and the corrosion of steel. More recently, stainless steel and titanium are preferred as reinforcing material. Seeking for a better compatibility, hydraulic lime micro-mortar has also been used as injection material in some cases, instead of Portland cement or epoxy resin.

Internal reinforcement has been proposed in some cases as a possible way to strengthen or stabilize earth constructions. However, combining earth with very still materials, such as metals or FRP, may prove inadequate due insufficient bonding and lack of mechanical compatibility, resulting in inefficient and even counter-productive interventions. Instead, earthen structures can be more successfully strengthened using wood internal members.

Reinforced injections (stitching)

Stitching constitutes a particular case of internal reinforcement in which short bars are inserted at different angles from the exterior leaf. Holes are drilled in the element and filled with bars composed of adequate and durable metals (stainless steel, titanium…) or FRP's. The holes are injected or filled with fluid mortar or grout. Stitching acts by improving or reinforcing the material or structural member, or by tying different parts (as converging walls) together. Reinforced injections will cause some deterioration to the wall or stone in which the drilling is executed and, in principle, should not be applied when the walls or stones show fixed artistic contents (paintings, carving, artistic treatments or decorations). The use of Portland cement grout should normally be disregarded because of incompatibility with the surrounding stone or masonry. Epoxy resin may also generate some severe compatibility problems. Reinforced injections constitute a full invasive and irreversible technique. In some cases, a socket is used to confine the injection material; however and even in these cases, removing the reinforcing and injected materials is normally not possible.

Stitching may have inadequate mechanical side effects due to the fact that, while improving the overall strength and ductility of the member, it may also increase the likeliness of cracking and damaging in the units (stones or bricks) due to soil settlements, earthquakes or other actions. Without reinforced injections, cracks will more likely develop along mortar-unit interfaces and thus cause less significant and more easily reparable damage. Under moderate actions, unreinforced masonry will experience cracks, but they will mostly develop along mortar joints and will not affect the bricks or stone blocks. This type of cracks is, in principle, not difficult to repair. However, stitching will prevent this type of response; once strengthened with reinforced injections, damage (cracks, local crushing) will develop across the bricks and stone blocks. Damage will appear in a more generalized and smeared way, showing numerous thin cracks instead of a single or few individual cracks. Cracks, thinner, more numerous and having formed across the blocks, will be much more difficult to repair. In short, reinforced injections may vary significantly alter the resisting nature of the structure, the type of expected damage, and its repairability.

Reinforced repointing

Combines traditional repointing with the inclusion, within the bed joints, or reinforcing bars. The reinforcement is normally made of ductile and durable metals or FRP's. Reinforced repointing increases the compressive and shear strength in small thickness masonry. It is normally more effective as a way of reducing the deformation. Reinforced repointing has also a confining effect on the walls, may be used to improve the connection between parts and may provide additional ductility.

Reinforced repointing is indicated for masonry walls with regular horizontal joints. It is usually applied in combination with other interventions. Unlike internal reinforcement and stitching, reinforced repointing can be considered only moderately invasive and mostly removable, as in many cases removing again the mortar and even the bar from the bed joints will be possible. Adequate durability of the reinforcement (especially when using metals), as well as adequate mechanical connection, require the placement of the bars at sufficient depth within the bed joint.

Jacketing

It is based on the application of an outer covering of high-performance material around an existing structural element. Jacketing has been applied often using cast-in-place reinforced concrete or steel.

It is applied to stabilize damaged members working in compression (such as pillars and piers), or to strengthen elements subjected to high compressive stresses, members showing excessive lateral deformation or formed by parts poorly connected. The target is at producing continuous confinement and thus improving the strength and stiffness of the material.

The covering does not need to be connected to the strengthened member when the only sough effect is actual confinement. In this case, jacketing may be fully non-invasive and removable. However, pure confinement action can be also attained by placing a few metal rings along the pier or pillar, as done in historical times. Jacketing is obtrusive since it requires hiding the original masonry and paraments behind the new material.

Providing connection between the covering and the original material with generate, in addition, a reinforcing or enlarging effect. In this case, and due to the connection, jacketing can be hardly removable, while some mechanical compatibility problems may appear when the material used for the confinement is much stiffer than the original one. In turn, and due to its continuous nature, jacketing may produce chemical and physical compatibility problems because of the creation of a new and external water-proof barrier preventing the natural perspiration of the original masonry or stone.

Prestressing

Prestressing by means of steel or FRP bars or strands provides a number of very interesting applications for the repair or strengthening of masonry and timber structures. Due to its versatility, to the light character of the technology, as well as to the possibility of using external solutions, prestressing-based applications may result in satisfactorily non-invasive, removable (if no injection or grout is used) and mechanically compatible solutions.

A first application of prestressing is found in the generation of pre-compressed conditions by means of fully controllable means. Providing an initial state of compressive stresses will, in many cases, increase the capacity of the masonry members to resist flexural and tensile forces, including those induced by earthquakes. A side effect is usually the increase of the stiffness of the element due to crack closure and delayed cracking upon loading.

Prestressing by means of bars or strands can be also used to improve confinement or tying effects by introducing an active action. By providing some initial prestressing force, ties and confinement rings will, since its implementation itself, generate an stabilizing force against the structure or the structural members (for instance, in the case of arches, vaults and domes) without them having to experience further deformation.

Yet another application of prestressing can be found in the generation of frictional contacts and connections. Providing compressive stresses perpendicular to the contact surfaces (among different parts, or cracked members) generates significant friction and hence prevents from relative sliding. Generating frictional forces across different members can be used as a way to mechanically tie the two parts in shear. A frictional union can be generated by means of prestressing bars or tendons, either embedded or externally applied, and adequately anchored in the two parts.

Dismantling and reassembling

Consists on the complete dismantling of an element or a structure to repair, extract or substitute part of the components, and then rebuilding it accurately according the original organization and shape. The purpose is to recover the functionality of a structure while maintaining its historical and cultural value. According to the ICOMOS/ISCARSAH Recommendations, dismantling and reassembly should only be undertaken when required by the nature of the materials and structure and/or when conservation by other means is more damaging. Dismantling and reassembling has been used for repair and restoration, as for instance when there are problems caused by the corrosion of iron or steel used embedded in the masonry or stones, put there during the construction or during later restorations.

Secondary structures

In some cases, existing masonry structures have been strengthened by building a secondary reinforced concrete or steel structure, connected to them and aimed at provide additional strength and stiffness. Accurate and realistic structural analyses are necessary because of potential mechanical incompatibility problems. A stiffer secondary structure (for instance, a new concrete frame) will tend to attract more loads, and may eventually become the only part actually resisting either the vertical or horizontal actions. Instead, a flexible steel frame will provide resistance only after the masonry has cracked. Additional strengthening of the original structure may be needed in order to preserve its integrity.

An extreme variant is found in the functional substitution of the structure, consisting of the creation of a new structure intended to take entirely the resisting role, while the original one preserves its historical and aesthetical values. In principle, this type of operation does not comply with the modern understanding of conservation or upgrading of cultural heritage structures. However, structural substitution may be designed to ensure full reversibility and non-invasivity and can be considered as a radical possibility for very severely damaged or seismically weak structures whose upgrading by other means would require the use of other more invasive and impacting procedures.

3 IMPROVEMENT OF MASONRY QUALITY AND EQUILIBRIUM

The improvement of the masonry quality can be necessary to repair local damages, (e.g. cracks, local discontinuities, etc...) or most extended damages as in case of long term lack of maintenance or general poor quality.

The repair technique, then, should be chosen considering the general criteria described in Section 2.1 in base to the extension of the damaged area, as well.

The equilibrium improvement of a single panel is related both to its masonry quality, geometric characteristics (e.g. slenderness) and the connection with other structural elements and portions, (Almeida, 2010). This last point will be extensively studied in the Section 3.3.

3.1 STRENGTHENING OF MASONRY WALLS

3.1.1 Grout injection

Repair and strengthening by grouting of brick and stone masonry walls has been largely applied in Italy on historic buildings and dwellings in the seventies and eighties, after the main earthquakes of Friuli and Irpinia; nevertheless no great effort was done in advance and during the time to test the effectiveness of this technique.

Even if experimental and analytical research has been carried out in the past decades on these techniques, nevertheless the effectiveness was always checked in terms of strength increase rather than on chemical, physical and mechanical compatibility with the original masonry, (Modena et al., 1997a,b,c) and (Binda et al., 1997). Few research was carried out in this direction on the effectiveness of grout injections, (Tomaževič and Turnsek, 1982), (Tomaževič, 1992), (Binda et al., 1993a,b), (Binda et al., 1994), (Modena and Bettio, 1994), (Bettio, 1996), (Laefer et al., 1996) and (Valluzzi et al., 2004). The conclusions recommended a careful approach and suggested a previous knowledge of the masonry wall morphology and of the masonry characteristics, since some types of walls could be not injectable.

Furthermore, grout injection is a non-reversible technique, the use of which can result in durability and compatibility issues.

This intervention consists on injecting grout mixtures of different types into the masonry, using particular techniques, in order to:

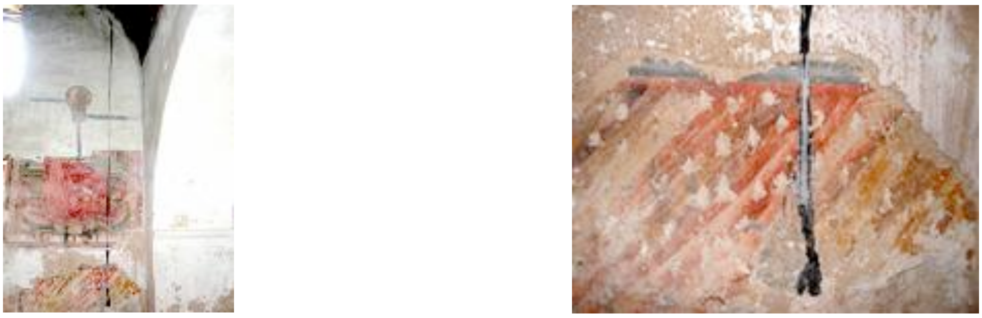
1. Reconstitute the structural continuity of the masonry;
2. Increase the masonry homogeneity, filling voids, if existing;
3. Improve the masonry mechanical properties, (increase the masonry strength);
4. Fill cracks in wall and other masonry structural elements (vaults, domes, etc...).

The aim can be fulfilled only knowing with good precision the morphology of the wall section, the materials constituting the wall and their composition in order to avoid chemical and physical incompatibility with the grout, the crack distribution, the size, percentage and distribution of voids, (Binda et al., 1993a,b), (Binda et al., 1997), (Binda et al., 1994), (Laefer et al., 1996), (Valluzzi et al., 2004).

The main problems connected to the grout injection can be summarised as follows: (a) the lack of knowledge on the size distribution of voids in the wall,(b) the difficulty of the grout to penetrate into thin cracks (2.0-3.0mm), even if microfine binders are used; (c) the presence in the wall, of fine and large size voids, which make difficult choosing the most suitable grain size of the grout (injecting large size voids with a fine grained mix can in fact induce segregation); (d) the segregation and shrinkage of the grout due to the high rate of absorption of the material to be consolidated; (e) the difficulty of grout penetration, especially in presence of silty or clayey materials; (f) the need for sufficiently low injection pressure to avoid either air trapping within the cracks and fine voids or even wall disruption. As known, the injection technique can fail when badly applied, with limits connected to the masonry morphology, to the desegregation and sedimentation of the grouts, to the mix characteristics (grain size distribution), to the operative technique.

Therefore, the effectiveness of a repair by grout injection depends not only on the characteristic of the mix used, but also on its mechanical properties and on the injection technique adopted and once again on the knowledge of the wall type. The injectability of the grout is influenced by its compatibility with the masonry to be repaired, as well.

Furthermore, the control of the grout penetration and correct spreading is difficult to predict. In case of frescoes, the technique should be used very carefully with the presence of restorer in order to prevent damages to the paintings.



(a) (b)

Figure 3.1 Percolation of the grout on a fresco after a grout injection (Source: POLIMI).

The technical improvements of the last years has developed new grouts with specific properties, such as a low salt content and a ultra fine size of the aggregate, has shown how to optimise the injection methodology, such as the injection pressure or the distance between the injectors, in function of the masonry characteristics. Multiple leaf walls can be made with very poor mortars and stones but have very low percentage of voids, (less than 4% of voids is not injectable), and have internal filling with loose material, which is not injectable.

Sometimes, in the case of disastrous events such as earthquakes, an apparent ineffectiveness of the consolidations using injection is observed. Figure 3.2 and Figure 3.3 show two of the cases where injection were very poor (Binda et al., 2003a), (Binda et al., 2006) and (Binda et al., 2009). In fact, in these cases there is an inhomogeneous execution of the intervention due to: (1) a poor design of the injection mixtures, (2) a rough and uncontrolled execution of the intervention, (3) a punctual distribution of the mixture due to the excessive distance between the injection holes, (4) the inapplicability of the technique due to the lack of voids (Figure 3.2 and Figure 3.3). The ineffectiveness of the interventions injection, therefore, is due to poor execution of the same, not the technique itself.

Surveys after the 1997 earthquake in Umbria on damaged walls often showed the difficulty of diffusion of the grout injection within stone masonry sections. Frequently only some spots were injected in the case of this wall with a very low percentage of voids (Binda et al., 2003a), (Figure 3.2 and Figure 3.3).

Figure 3.2 Poor results of applied injection, (Binda et al., 2003a).

Figure 3.3 Only some spots were injected in the case of this wall with a very low percentage of voids, (Binda et al., 2003a).

The choice of the mixture to inject is done by selecting the best characteristics for the type of wall and the crack extension on which to intervene; the mechanical strength of the mixture and its deformation characteristics (Elastic Modulus and Poisson's Ratio) should be similar to those of the original wall. To obtain a high penetration of the mixture, the size of the composing materials should be as small as possible and homogeneous and the mixture should present a low-viscosity in the fluid state, the curing and hardening period should be suitable for the execution of the intervention and the shrinkage must be limited, the chemical properties of the binder should be maintained throughout time and should be compatible with those of the original mortar, the mixture should not be soluble in water, should not vary in volume with moisture and should not obstruct the movement of vapour; finally, the material used to create the mixture should be easily found and inexpensive. Gypsum mortars or stones rich in sulphate require particular grout composition.

The injection procedure, (Figure 3.4), necessary to perform an injection intervention is the following:

1. study of the grout composition;
2. choice of the injection point, the distance between injectors and their layout, according to the masonry characteristic (presence of cracks, porosity, geometry, etc.); 2-3 injections point/m^2 could be effective;
3. removal of the damaged plaster and superficial crack filling (to avoid loss of grouts);
4. hole drilling (diameter: 40.0mm);
5. positioning of the injection devices and repointing by mortar
6. preliminary water injection in order to remove dust and disaggregate materials but also to saturate the wall, avoiding the masonry suction. The segregation and shrinkage of the grout due to the high rate of absorption of the masonry should be prevented.
7. evaluation of the injection pressure
8. grout injection, starting from the lower part of the crack.

(a)

(b) (c) (d)

Figure 3.4 Grout injection procedure. (a) General overview of the process. (b) Drilling holes in mortar joints. (c) Fixing plastic pipes. (d) Injecting grout (Source: UNIPD).

Preliminary applicability tests on selected area are highly recommended, as well as their visual inspection by local dismantling.

Regarding possible negative effects, one could mention problems due to the use of grouts with high cement content. Nevertheless, as such grouts are not any longer used, one could not refer to them.

No specific standards were developed, even if it is a wide used technique. Recommendation for the application procedures are shortly described in the following documents developed after the earthquake in Friuli (in Italian): Legge Regionale Friuli Venezia Giulia, DT 2 del Novembre 1977. Decreto 2 Luglio 1981, Circolare 30 Luglio 1981.

If applicable, the technique does not require any particular periodic control or monitoring but, as for ternary and hydraulic lime based grouts, (now in use), there may be problems of durability. Some results are given in the publication by Kalagri, (paper in press). The text is also attached to this file.

A methodology for on site and laboratory testing of grout injections in multiple leaf stone masonry walls was set up firstly by Binda, Baronio in collaboration with Modena who allowed on site injection of walls, in 1992, (Baronio, 1992). Injectability tests proposed by Binda et al. in (Binda et al., 1993b) can be carried out in laboratory on materials sampled from the internal part of walls. The sampled material can be inserted in cylinders and the injected in laboratory.

Compressive and splitting tests (Brazilian tests) on the injected cylinders in laboratory can be carried out on the cylinders after the time necessary to reach the hardening of the grout; other recommended tests are the sulphate content of the grout, grain size distribution of the grout, segregation and shrinkage of the grout, mechanic characterisation of the grout, water retentivity of the grout, together with addressed characterisation tests on the masonry. The methodology is presented in Figure 3.5, (Binda et al., 1993b) and (Binda et al., 1994).

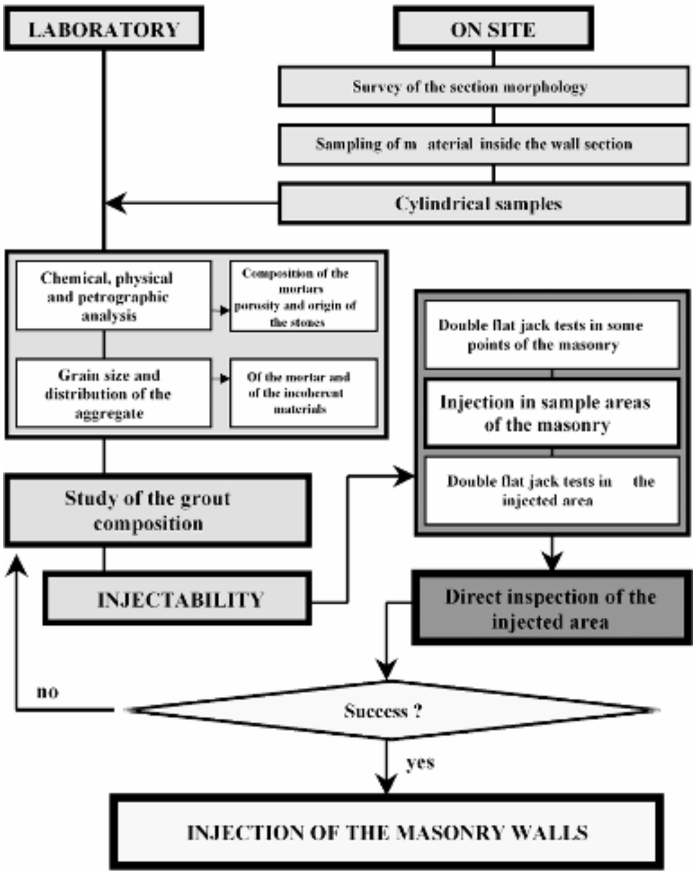


Figure 3.5 Methodology for testing repair by injection on site and in laboratory, (Baronio, 1992) and (Binda et al., 1993).

Injectability tests proposed by Binda et al. (Binda et al., 1993), (Laefer et al., 1996) calibrating to masonry a procedure applied to soil. Plexiglas cylinders are filled up by sampled materials and injected with the grouts appropriately chosen and studied, (Figure 3.7).

The filling material can be sampled from the interior of masonry sections on site, (Figure 3.6). Then after hardening the cylinders should be cut and the distribution of the grout studied, (Figure 3.8), (Binda et al., 1997).

The grouts can be injected from the bottom of the cylinders through a lateral injector at a pressure varying from 0.02 and 0.04N/mm^2. When the grout reaches the highest level the filling can be considered completed by pouring the grout from the top. When the bottom of the cylinder contains high quantity of loose material only the filling by gravity is possible.

After 28 days the cylinders can be cut and inspected or mechanic tests can be carried out. The cylinders which are not cut can be submitted to compression tests in order to study the effects of the injection of different grouts, (Laefer et al., 1996).

The following remarks can be made:

- while injecting: segregation could took place between the water (absorbed by the loose material and by the mortar) and the remaining part of the grout;
- due to the segregation some cracks can appear in the injected grout;
- when the loose material formed a complete layer through the section of the cylinder, only pouring material from the top was possible;
- air bubbles.

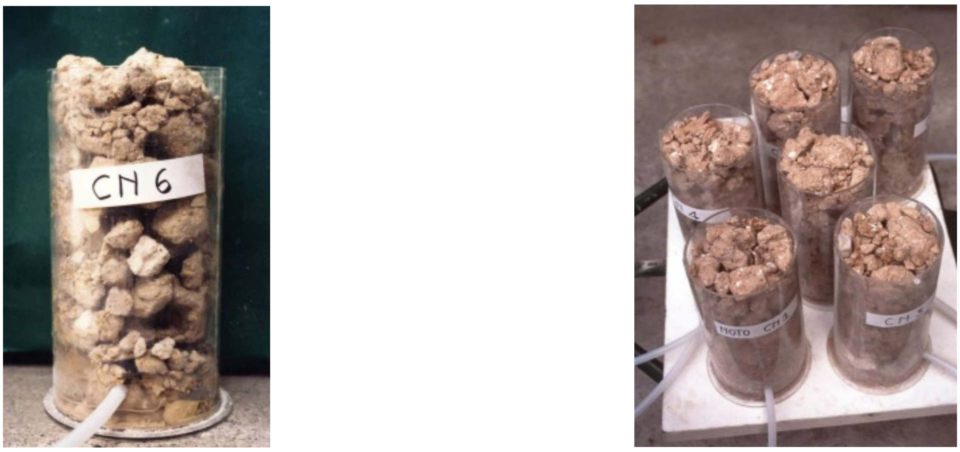


Figure 3.6 The transparent cylinders filled on site by internal leaf materials, (Laefer et al., 1996) and (Binda et al. 2003b,c).

Figure 3.7 Injection of the cylinder in laboratory, (Source: POLIMI).

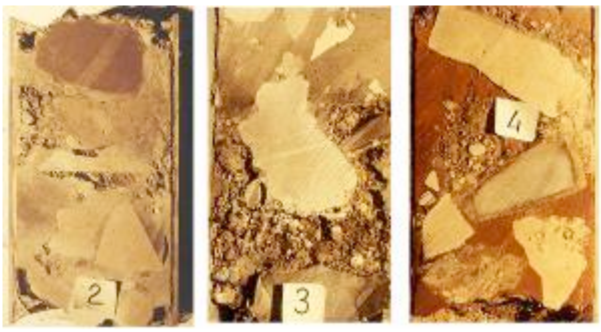


Figure 3.8 Observation of the distribution of grouts in the cylinders, (Binda et al., 1993) and (Binda et al., 2003b,c).

Miltiadou et al., due to the necessity of strengthening a historic structure made of three leaf stone masonry with clay mortar (free of material that could swell, however), have developed a cement/lime/pozzolan grout that should be able to fill as much voids as possible (minimum size 0.2-0.3mm). They tested the injectability of the grout using the standard test of the sand column, as well as using cylinders with material taken from the structure to be strengthened, (Binda et al., 1993a,b). The laboratory tests were successful; the site application was successful, as well. However, for the time being, there are no experimental results measuring the improved mechanical properties of the grouted masonry.

3.1.2 Jacketing by R.C. plaster

The aim of the technique is to connect better the different leaves of a wall in damaged conditions producing a new section constituted by the old one increased by the two jacketed reinforced parts. The idea behind it is to have a thicker section, to increase compressive, tensile and shear strength and ductility, (Binda et al., 1994). The same technique has also been applied to connect load-bearing and shear walls and also large cracks, as well.

The technique consists in positioning a reinforcing net (Φ = 6.0 to 8.0mm) on both faces of a wall, connecting the two nets with frequent steel connectors and applying on the two faces a cement mortar based rendering, which constitutes a sort of slab, (Figure 3.9). The masonry panel, then, acquires high strength and stiffness, which is not always a positive point by considering the overall behaviour of the building. Furthermore, the discontinuity due to the floor presence produces further local decrease of stiffness and weakness, (Figure 3.10), as well as the unregular layout of the reinforcing.

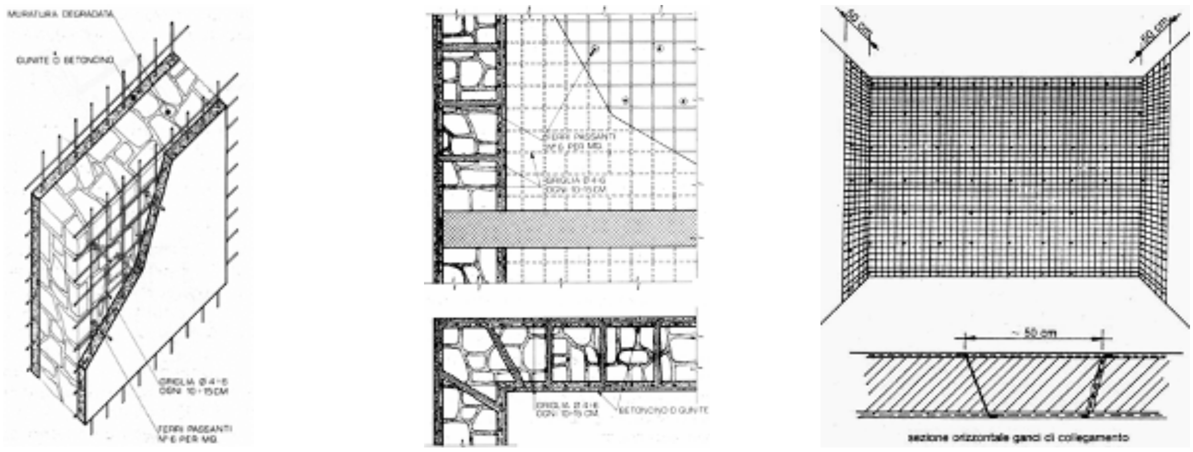


Figure 3.9 Layout of the R.C. jacketing reinforcing.

(a) (b) (c)

Figure 3.10 (a) Damage to the change of stiffness; (b) wrong design: only a wall is reinforced (c) lack of connection between the walls and in correspondence to the floors, (Source: POLIMI).

This technique was largely applied particularly to irregular multiple leaf stone walls in Italy and it is recommended by the Italian Code. Nevertheless, its execution on site is not very easy due to the inhomogeneity of the walls, to the cost and difficulty of connecting the two faces of the wall, (Figure 3.11).

In fact, it is possible to observe frequently local failures of jacketed walls, almost always very clearly connected to poor detailing. Examples are shown in (Figure 3.12a, b), representing failures respectively due to insufficient steel mesh overlapping (Figure 3.12a), and insufficient transversal ties confining action (Figure 3.12b), (Modena and Bettio, 1994) and (Modena et al., 1997b).

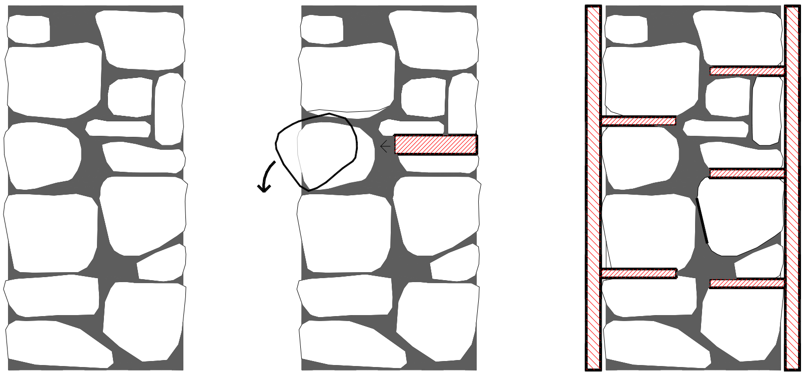


Figure 3.11 Difficulty in the application of jacketing to multiple leaf stone masonry, (Binda et al., 2003a).

The most diffused mistakes made on site are described in the following together with the consequent damages: (i) lack of connection between the nets in orthogonal walls and in correspondence to the floors; they cause discontinuities between the walls (Figure 3.12a), (ii) lack of overlapping between two different sheets of the net (Figure 3.12c), (iii) absence of steel transversal connectors (Figure 3.12d), (iv) use of too short connectors (Figure 3.13), (v) lack of uniformity of distribution of the repaired areas in the structure; this can cause torsion stresses due to non uniform distribution of the stiffness.

Furthermore, it was often observed a low durability of the technique, due to an insufficient thickness of the steel cover with consequent steel corrosion (Figure 3.13). This problem is of a great importance in building where capillary rise and diffuse moisture are widely recognisable. The presence of the R.C. jacketing often produces an increase of moisture, being a sort of barrier for the evaporation.

(a) (b) (c) (d)

Figure 3.12 (a) Failure due to insufficient steel mesh overlapping and (b) insufficient transversal ties confining action; (c) lack of connection between nets; (d) absence of connectors, (Binda et al., 2003a).

(a) (b) (c)

Figure 3.13 (a) Too short connectors; (b), (c) corrosion of the steel net in a jacketed wall, (Binda et al., 2003a).

3.1.2.1 Earthen Walls: Steel mesh with cement plaster

Nailing wire mesh bands to both sides of adobe walls and covering them with cement mortar was a relatively common retrofitting in South American in the past decade, (see Blondet and Aguilar 2007). Although Zegarra et al. (2001, 2002) describes how houses reinforced by means of cement-plaster-covered steel mesh performed better than unreinforced houses in recent seismic events in Peru, simulation tests performed at PUCP resulted in brittle failure (Blondet and Aguilar 2007), probably due to the fact that the reinforced mortar bands were much stiffer than the adobe walls, (Torrealva et al., 2009a).

In addition, cementitious materials are chemically, mechanically and physically incompatible with earth due to its low strength, high porosity and a different hygrothermal behaviour in comparison to cement based material, resulting in a decrease in durability performance due to two factors, the first being incompatibility between cement plaster and earthen materials as described in section 3.2.6.1; the second being steel corrosion, expansion and contraction due to temperature increase and decrease underneath the plaster, which can cause the earth plaster to spall. The latter phenomenon is a hypothesis which has not been studied in depth and is not supported by evidence in research, but can be observed on site (see Figure 3.14). The corrosion of the steel mesh and spalling of the overlaying cement render seems to be attributable to the insufficient thickness of the cement render, resulting in carbonation.

Figure 3.14 Cracking and spalling of earthen mortar over steel mesh reinforcement, Chile (Source: BAM).

3.1.3 Structural Repointing (Deep repointing)

Deep repointing is a widely applied technique in all types of masonry. However, deep repointing is expected to be efficient in enhancing the mechanical properties of masonry only in some cases and under several conditions. This operation involves the partial replacement of the mortar joints with better quality mortar, in order to improve the masonry mechanical characteristics, and it should be applied in the case the deterioration is localized only in the mortar.

The described operation can increase the masonry resistance of both vertical and horizontal loads, but the best results are obtained especially in terms of deformation, which are also greatly diminished due to the confinement effect of the joints.

Actually, strength enhancement is expected only when a significant percentage of the initial weak or deeply damaged mortar is replaced by a new more compact and rather stronger one, but not excessively rigid and resistant to avoid creating areas in the masonry with inhomogeneous behaviour. This is the case in rather thin masonries, provided that deep repointing is applied on both sides of the masonry elements. Due to the fact that, normally, stone masonry elements are rather thick, one cannot expect this technique to be efficient in this case. Nevertheless, deep repointing (more or less deep) is applied to stone masonry as well, either in rather thin elements (45.0 to 60.0cm thick) or for other purposes, such as a preparatory step before the application of grouting to masonry. The technique can integrate other repair technique as grout injection; in this case it can be applied in order to better confine the injected material.

In the case of consolidation using repointing, the inability to maintain the original plaster must be taken into account, as so, this type of intervention cannot be use in the presence of fine plaster or frescos, i.e., in the case of buildings of historical and artistic importance.

A possible disadvantage of the deep repointing is that its efficiency strongly depends on the quality of on site application. Actually, the mechanical properties of the mortar produced on site for deep repointing may vary significantly from batch to batch, whereas the removed mortar may be partly replaced by stronger one, thus leaving the joints partly un-filled.

The damage to multiple leaf masonry walls due to earthquake occurs due to shear in plane forces or more frequently out of plane forces e.g., due to the thrust of roofs and floors. The failure process is due to differential behaviour of the two leaves of the wall under the seismic load, when the leaves are not connected by passing through stones. Another cause is the lack of bond between mortars and stones; this causes an easy separation of the stones from the mortar.

There can be different causes of the process, which appears similar in all cases; (i) the horizontal force due to the earthquake, acting in the plane of the wall finds a differential response of the two leaves and tends to separate them; (ii) the thrust of the roof causing flexural stresses is differentially opposed by the leaves which partially collapse after separation; (iii) the execution of a heavy stiff reinforced concrete tie beam to connect floors and walls causes the migration of the highest stresses under the beam, leaving the other leaf unloaded and therefore allowing for local differential movements.

The aims of the deep repointing, provided that it is carried out with very good workmanship, are multiple: (i) to connect in a rather thin section the stones of the external leaf substituting the original mortar in the joints when it is damaged, cracked and in any case very poor, (ii) to confine the wall at a less extent than the jacketing, but with better results since the bond with the existing stones and mortar can be better assured, (iii) to confine better the injected material when grout injection is carried out, (iv) to provide a better penetration (140.0mm over the thickness of the wall section) and distribution of the mortar compared to the random penetration and distribution of a grout injection. When cracks are also present, then they should be injected with grout and the deep repointing becomes a complementary action. Eventually when needed steel transversal ties should also be added in order to connect the leaves of the wall. This technique can be particularly efficient in the case of two or three leaves stone walls reaching a thickness not higher than 600.0mm.

Before deciding the application of the deep repointing technique an on site investigation should be carried out in order to provide the crack pattern of the walls, the thickness of the section (it should be no more than 45.0 or 60.0cm) and the morphology of the masonry itself (number of leaves, brick and stone arrangement), physical and chemical characterisation of the materials (physical properties of the units, brick or stones, salt content, type of mortar, grain size distribution, binder/aggregate ratio, etc...).

To perform the intervention, first the wall should be prepared, after which the joints should be repointed and refined, Figure 3.15.

First it is necessary to remove any existent plaster and scrape the joints (at least 5.0 to 7.0cm), the scrape operation, if possible, should be done with traditional tools and not with power tools, to avoid vibration and percussion harmful to the masonry, but the manual operation has a high cost in terms of time and therefore too expensive. Much attention must be paid not only to the depth of stripping, but also to the perfect elimination of the original layer of mortar that is in contact with the resistant elements in order to allow the new mortar introduced to develop ties of adherence with the elements themselves. The repointing can be done on one side or both sides of the wall, with a maximum depth, on each side, of 1/3 of the entire thickness of the wall, after verification of any signs of instability of the entire wall or of single portions of it. In the case of combined interventions on both wall surfaces it is best to perform the complete repointing on one side and then begin to intervene on the other side. After the removal of impurities and waste powder of the mortar, the wall should be washed with low water pressure.

The traditional repointing basically consists on filling the joints with mortar. The operation must be careful performed in order to ensure the proper filling of the joints and, to this end, it must be done in two layers: after placing a first layer of mortar, this must be treated on the internal part of the joint to avoid formation of voids, then proceed to the application of the second repointing layer, until the total filling of the joint, taking attention to the reposition on the external surfaces of any wedges removed during the scrapping of the joints.

The final aesthetic operations are composed by scraping and cleaning of the joints, to remove the mortar smears/debris of the resistant elements.

Often, in conjunction with the repointing operations, it is necessary to intervene on the wall also with injections or scuci-cuci operations, to increase the effect of improvement due to the solely introduction of new mortar in the joints.

Attention should be given to the choice of the mortar to avoid unwanted chemical, physical and mechanical reactions: in general cement based mortars are used, as they provide higher strength, however, this type of mortars may trigger unwanted chemical reactions in the masonry.

It is also crucial to perform the intervention in the depth of the masonry; it is frequent to found, in fact, a malfunction of this technique, because it wasn't well applied in depth, but limited to an aesthetic improvement of the surfaces rather than of increase in mechanical properties.

In general, the repointing is ineffective in cases where there is a poor execution of the intervention.

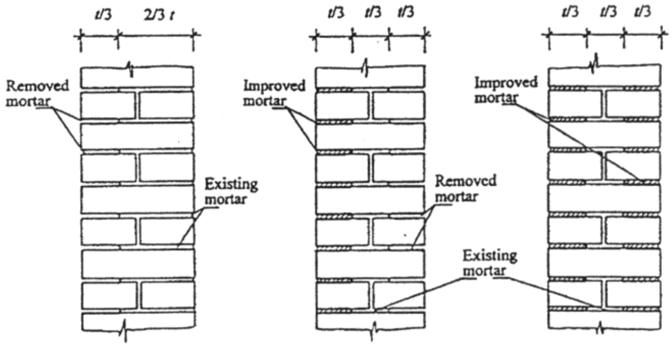


Figure 3.15 Execution stages in the case of repointing performed on both sides of the wall.

3.1.3.1 Laboratory tests to brick masonry

As proven by preliminary tests carried out at NTUA (Vintzileou, 2001), when masonry joints are partly filled with the repair mortar, the compressive strength of masonry may be decreased, compared to the initial one.

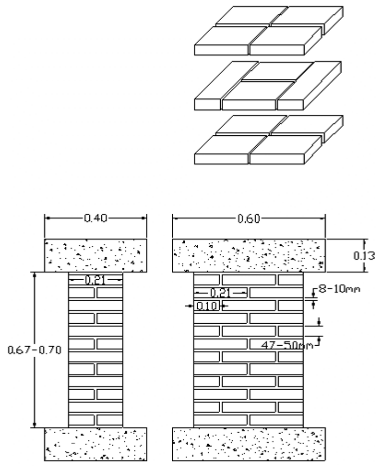


Figure 3.16 Brick masonry wallettes tested in compression before or after deep repointing, (Vintzileou, 2001).

Nine brick masonry wallettes, (Figure 3.16), were constructed using solid bricks (compressive strength ~50.0MPa) and weak lime mortar (compressive strength ~1.0MPa). Three wallettes were tested in compression as built. A compressive strength approximately equal to 6.5MPa was measured.

In three wallettes, the mortar was removed from the joints to a depth of ~40.0mm and substituted by a lime cement mortar (compressive strength varying between 5.5 and 9.2MPa).

In the remaining three wallettes, the mortar-removed to a depth of 40mm as well, was replaced by cement mortar, having a compressive strength varying between 13.9 and 17.1MPa approximately.

The experimental results have shown in some cases, a non typical failure mode, implying extensive spalling of the bricks, (Figure 3.17).

Inspection of specimens after testing has demonstrated that in those cases, parts of the joints were left un-filled. The observed failure mode, attributed to stress concentration to the faces of masonry, is associated with compressive strength of masonry lower than the initial one.

Figure 3.17 Spalling of bricks due to incomplete filling of mortar joints during deep repointing.

On the contrary, wallettes in which the removed mortar was completely replaced by a stronger one, have exhibited significant enhancement of their compressive strength. It should be noted, however, that in this specific case, almost 35% of the initial weak mortar was replaced.

3.1.3.2 On site application to stone masonry

A repair and preventive technique for double leaf masonry walls, including a repointing procedure, was proposed in (Binda et al., 2005) and (Corradi, 2006): the deep repointing of the masonry joints with an appropriate mortar, carried out on the two faces of the masonry. The repointing was carried out also in conjunction with grout injection and tested on site on a wall of a building which was later on demolished.

Within the framework of an extensive research supported by the Deputy Commissioner for interventions in the Umbrian areas struck by the 1997 earthquake, an investigation was carried out by POLIMI in cooperation with the University of Perugia and RITAM, a laboratory located in Terni and on site tests were performed on the application of various repair and strengthening techniques for multiple leaf masonry walls. Diagonal compression, simple compression and shear-compression tests were carried out on masonry panels of various dimensions, which had been strengthened with either traditional or innovative techniques. Concerning traditional method, panel injected with new lime based mixes and panels repaired by deep repointing of mortar joints were tested.

The aim of the research was to characterise the behaviour of the masonry typical of the studied areas and to study the effectiveness of the seismic upgrading and reinforcing work both on undamaged and damaged walls. The tests carried out provided interesting indications for practical utilisation of the studied techniques.

Only the deep repointing technique alone or combined with grout injection is reported here. The result show a significant increase in strength and stiffness compared to the non strengthened panels.

Description of the technique

An appropriate preliminary investigation on site and in laboratory should be carried out, (Binda et al., 2005), through: (i) an accurate geometrical survey of the masonry morphology (number of leaves in the section, dimension of the leaves, type of connection between the leaves), (ii) characterisation of the stones and of the mortars, (iii) survey of the physical and mechanical decay, (iv) crack pattern survey.

Stones and mortar were sampled from the walls and laboratory tests were carried out. Chemical, petrographic-mineralogical analyses were performed on the mortars in order to detect their composition: type of binder, type of aggregates, binder/aggregate ratio, aggregate size and dimensions. On the stones petrographic observations, physical analyses and mechanical tests were performed. Based on to the results of the laboratory tests, appropriate materials were chosen both for injection and repointing.

The morphology of the walls suggested that in some cases injection repairs were not appropriate due to the fact that inside the masonry there was practically loose material and no voids were present, (Abbaneo, 1993) Therefore before applying the technique, appropriate tests were carried out in laboratory on sampled materials and also on site, (Laefer et al., 1996).

The aims of the deep repointing are the following: (i) replace the damaged mortar on the wall surface to a depth of 70.0mm in order to adequately bond the stones, (ii) connect the stones together and to the external part of the wall, (iii) confine the wall externally also in the case of injection, (iv) provide a better penetration of the grout while avoiding leakage to the exterior. When the repointing is successful, 140.0mm of the wall section (in case of two leaf walls the thickness varies from 500.0 to 650.0mm) are well bond together and constitute good confinement for vertical loads. This technique can assure a better uniform distribution of the material in the external leaves, in comparison to the injection technique. As said before, injection is not always successful if it cannot penetrate inside the masonry and connecting the two leaves; in many cases transversal connector, are needed instead. Regardless of this, injection is necessary when diffused cracks are present.

In the following the various step of the technique application are described, provided that the choice of the walls to be repaired has already been made on the basis of the above described investigation.

The choice of the repair materials has to be made following requirements which cannot be codified, but must be based on the respect of the existing materials and structures. Therefore a previous investigation on the existing materials used for masonry, bricks, mortars for bedding and pointing (or repointing) has to be carried out. Subsequently the grout for eventual injections and the mortar for deep repointing have to be previously chosen also based on laboratory tests as: (i) grout injectability tested on site on a wall sample and in laboratory, (Laefer et al., 1996), good commercial premixes can be found or modified and adapted to the requested properties)**,** (ii) choice of a mortar for deep repointing which has good strength but no high stiffness and good adhesion with the stones, (iii) choice of a mortar for the external repointing, compatible and aesthetically acceptable compared to the existing repointing.

Deep repointing of the panels.

The following steps were performed in the application to a case of repair in Umbria, but they can be proposed as a general procedure for other cases, (Binda et al., 2005):

1. External filling of the eventual cracks with a material which can be removed after injection, in order to prevent the grout to exit, (Figure 3.18). The dark material appearing in the figure is only temporary set to stop the fluid grout from coming out of the wall;
2. Injection of the eventual diagonal cracks with a grout chosen on purpose having good injectability, stability and structural strength, (Figure 3.18);
3. Joint cutting and cleaning up to a depth of 70.0mm made by hand with hammer and chisel, (Figure 3.19). Figure 3.20a and Figure 3.20b show the joints after cleaning by sandblasting. These operations have to be carried out on both faces of the wall. It is worth to remind that sandblasting is not always allowed, and alternative technique should be used for the cleaning;
4. Wetting of the wall with water on both sides in order to reduce the water absorption by the material and consequently shrinkage phenomena of the pointing;
5. pointing with a side made mortar or a ready mix. The repointing is made by hand and in subsequent layers, (Figure 3.20c), well tooled on both sides of the wall, (Figure 3.21) so that the right consistency can be reached. The wall is then cleaned and let to dry.

Figure 3.18 Filling of the cracks, (Binda et al., 2005).

Figure 3.19 Cleaning of the joint up to 70.0mm, (Binda et al., 2005).

(a) (b) (c)

Figure 3.20 (a) Joint after cleaning, (b) detail of the joint depth, (c) first layer of repointing, (Binda et al., 2005).

Figure 3.21 Repointing, (Binda et al., 2005).

Diagonal compression, shear-compression and compression tests on full scale panels

The diagonal compression and shear-compression tests were carried out on site and in the laboratory with the aim of determining the shear stiffness and strength of the masonry, (Corradi, 2003). The compression test was used to find the compression strength and the elastic parameters of the masonry (elastic modulus and Poisson's ratio). The on site tests were carried out on panels cut from load-bearing walls of damaged buildings which were going to be demolished. The operation was carried out with special cutting techniques with diamond wires in order to avoid major damages to the buildings. The load is given by hydraulic jacks (Figure 3.22). The on site diagonal compression test is the most frequently used and has now been assumed also by the Code due to its simplicity. The test has also been standardized by ASTM, (ASTM E 519-81).

The diagonal test was performed on site on panels of 1200.0x1200.0mm dimension with sections of different thickness and morphology in laboratory and on site.

The shear-compression test, (Figure 3.23), was carried out on panels of 900.0x1800.0mm dimension with thickness variable from 300.0 to 700.0mm. During the test, the masonry was subjected to a vertical constant stress σ_0 and simultaneously to a horizontal shear load T in the centre of the panel.

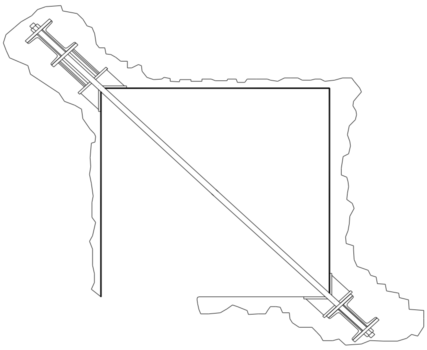


Figure 3.22 Diagonal compression test layout, (Binda et al., 2005) and (Corradi, 2008).

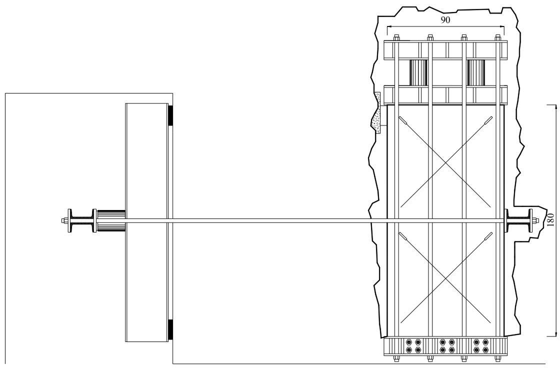


Figure 3.23 Shear compression test layout, (Binda et al., 2005) and (Corradi, 2008).

This test even if better simulates the state of stress of a masonry under horizontal loads, is much more complicated than the diagonal test.

A simple compression test is carried out before the shear-compression test on the same panels without reaching the failure, in order to determine previously some masonry parameters like the elastic modulus and the Poisson's ratio, as mentioned above. For a more detailed description of the tests, see (Corradi, 2002). In the case of the diagonal test, the calculated value of the shear stress τ is equal to the value of the principal stress σ_I as follows:

$$\tau = \sigma_I = \frac{P}{A\sqrt{2}} \qquad \text{(Eq. 1)}$$

where *P* is the diagonal compressive load generated by the hydraulic jack and *A* is the area of the horizontal section of the panel. With reference to this interpretation of the test as defined by ASTM E 519-81 Standard, it is possible to calculate the characteristic strength of the masonry through:

$$\tau_k = \tau_u = \frac{P_{ult}}{A\sqrt{2}} \qquad \text{(Eq. 2)}$$

Furthermore, it is possible to calculate the shear stiffness $G_{1/3}$ (secant value of the modulus at 1/3 of the peak load) defined as:

$$G_{1/3} = \frac{\tau_{1/3} - \tau_i}{\gamma_{1/3}} \qquad \text{(Eq. 3)}$$

Concerning the shear-compression test, it is possible to consider that panels of dimension 1800.0x900.0mm can be considered as two half panels 900.0x900.0mm, one over the other. The initial value σ_0 of the vertical stress is known:

$$\sigma_0 = \frac{P_v}{A} \qquad \text{(Eq. 4)}$$

where *P* is the vertical compressive load and *A* is the area of the transversal section of the panel. From the value of the maximum shear load T_{iu}, the maximum shear stress is calculated for the bottom half panel in which generally the shear failure is reached before, due to the highest constraint level:

$$\tau_u = \frac{T_{iu}}{A} \qquad \text{(Eq. 5)}$$

Then the value of the correspondent principal tensile stress σ_I in the bottom half panel is expressed by the following relationship:

$$\sigma_I = \sigma_0 \left[-\frac{1}{2} + \sqrt{\left(b\frac{\tau_u}{\sigma_0} \right)^2 + \frac{1}{4}} \right] \qquad \text{(Eq. 6)}$$

Through the values obtained in (Eq. 4), (Eq. 5), (Eq. 6), the characteristic shear stress τ_k at the bottom half panel is calculated:

$$\tau_k = \frac{\sigma_I}{b} \qquad \text{(Eq. 7)}$$

where

$$b = 1.543 - 0.478\frac{\tau_u}{\sigma_0} \qquad \text{(Eq. 8)}$$

is the shape factor.

The shear modulus G during the elastic phase is calculated with reference to the Sheppard static scheme assuming that the lowest half panel, which is the most highly stressed, behaves as an elastic beam perfectly constrained at the bottom. This causes a lack of symmetry in shear distribution between the upper and lower halves of the panel, which can be taken into account during the interpretation of the data. According to this hypothesis, the G modulus can be derived from (Eq. 9) in which it is the only unknown:

$$\frac{1}{K_0} = \frac{\delta_E}{0.9T_{iu}} = \frac{1.2h}{GA}\left[1 + \frac{G}{1.2E}\left(\frac{h}{d}\right)^2\right] \quad \text{(Eq. 9)}$$

where *d* and *h* are the thickness and height of masonry panel; *E* and *A* are respectively the Elastic modulus and cross section area of the panel. δ_E is the relative horizontal displacement between top and middle point of the panel behaving elastically and it is calculated as indicated in Figure 3.24.

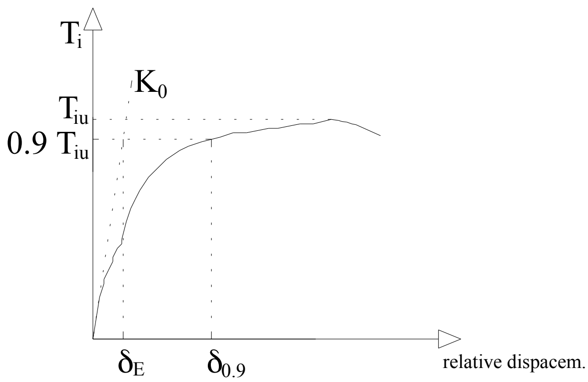


Figure 3.24 δ_E calculation procedure, (Binda et al., 2005) and (Corradi, 2008).

Preliminary calibration campaign

An on site test was set up in order to verify the effectiveness of repair; the test was carried out on site directly on a panel of a masonry wall of a house which was badly damaged by the earthquake and had to be demolished, (Corradi, 2008).

The pointing was performed on a stone panel partially collapsed after a first shear strength static tests which caused a diagonal crack. The crack was previously grout injected. This grout is declared by the producer as a grout for injection based on natural hydraulic lime. For the deep pointing the above described steps from 2 to 5 were followed. The pointing was carried out with a lime-cement mortar; the choice was made due to the fact that the stones were rather stiff and strong and that this repair was structural, then aiming to a better connection between stones but in a compatible way. 28 days after repair a diagonal test was again carried out, on the repaired panel. Figure 3.25 shows the measuring devices applied across the panel and the jack used for the test. At collapse the panel was only cracked along the diagonal, and not badly damaged.

(a) (b)

Figure 3.25 Measuring devices, (Binda et al., 2005).

In Figure 3.26 the test modality and the stress-strain plot are reported showing a rather good behaviour of the repaired panel.

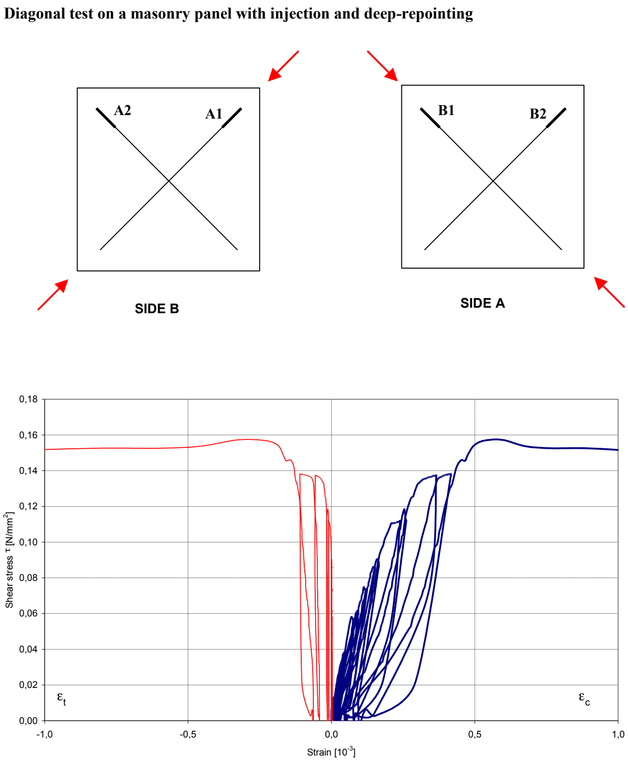


Figure 3.26 Test modality and results, (Binda et al., 2005).

Later on the panel was dismantled and the pieces taken to the Laboratory of DIS for study and control. In Figure 3.27 the good penetration of the mortar can be seen on the section of the panel.

Figure 3.27 A cut section of the repointed wall, (Binda et al., 2005) and (Corradi 2008).

The ultimate force calculated for the repointed panel was 140% more than the virgin panel, 125% more than a panel only injected and the same as the FRP repaired panel. An overall good result indeed.

Description of the further tests masonry and of the repair technique

As aforementioned, other panels were subjected to the mechanical tests before and after strengthening. Some were strengthened with injections and deep repointing, and some only with deep repointing. The grout was a ready mix hydraulic lime, the mortar for repointing was a lime-cement mortar.

Grout injection and deep repointing

The repair was carried out according the calibrated procedure and above described, as follows: at first some diffused cracks were injected in order to avoid leakage of the grout. Then the diagonal cracks caused by the previous shear tests were injected carefully. Finally, the excavation of the joints was performed and then the repointing was carried out in subsequent layers. The tests were carried out after 45 days in order to allow the grout and the mortar to harden.

The repair and testing campaigns were carried out on three different buildings belonging to three historic centres of Umbria: Ponte of Postignano, Farnetta and Trevi, (Figure 3.28), all situated in the Terni province. In the following, a brief description of the three buildings is reported.

(a) (b) (c)

Figure 3.28 (a) Ponte of Postignano (PG) bulding; (b) Farnetta building; (c) Trevi building, (Binda et al., 2005) and (Corradi, 2008).

Analysis of the results

In order to better describe the experimental programme, a list of the tests carried out in Ponte di Postigliano, Farnetta and Trevi is presented in Table 3.1 together with the masonry description and the eventual type of repair. The letters D and CD indicate diagonal compression tests, the letters TC and CS indicate respectively shear-compression tests and simple compression tests. As can be seen from Table 3.1, the walls were tested in the undamaged state before repaired.

Table 3.1 Reinforcement and panel characteristics, (Binda et al., 2005) and (Corradi, 2008).

Test No.	Building location	Reinforcement	Masonry texture
P-D-13-OR P-D-13-SI	Ponte di Post. Ponte di Post.	before repair deep repointing + grout injections	Double-leaf roughly cut stone masonry
CD-26-T-ORI CD-26-T-RIR	Trevi Trevi	before repair deep repointing	Double-leaf roughly cut stone masonry
TC-01-F-ORI TC-01-F-SIR	Farnetta Farnetta	before repair deep repointing + grout injections	Double-leaf roughly cut stone masonry
TC-02-F-ORI TC-02-F-SIR	Farnetta Farnetta	before repair deep repointing + grout injections	Double-leaf roughly cut stone masonry
CS-01-F-ORI CS-01-F-SIR	Farnetta Farnetta	before repair deep repointing + grout injections	Double-leaf roughly cut stone masonry
CS-02-F-ORI CS-02-F-SIR CS-02-F-SIR	Farnetta Farnetta Farnetta	before repair deep repointing + grout injections deep repointing + grout injections	Double-leaf roughly cut stone masonry

In Figure 3.29a,b,c the section of the walls tested in the three chosen buildings is presented. As it can be seen the walls are made with a two leaf masonry and irregular stones.

(a) (b) (c)

Figure 3.29 Section analysis: (a) Ponte di Postignano; (b) Trevi; (c) Farnetta, (Binda et al., 2005) and (Corradi, 2008).

Simple compression

These tests were only carried out on the Farnetta building. In detail, two panels of dimension 1800.0x900.0x480.0mm, were subjected to the following cycles of tests: (1) N.2 panels before reinforcing under simple compression, (2) The same two panels reinforced with deep repointing and injection under simple compression, (3) N.2 panels before reinforcing under shear-compression, (4) The same two panels reinforced with deep repointing and injections under shear-compression.

In Table 3.2 the detailed results of the simple compression test before and after repair are given.

In Figure 3.30 the stress-strain results of the same tests are presented. It can be easily seen that the increase in elastic modulus was respectively 32 and 5.69 times. Nevertheless high scatter in the results was found, as expected from such irregular masonry.

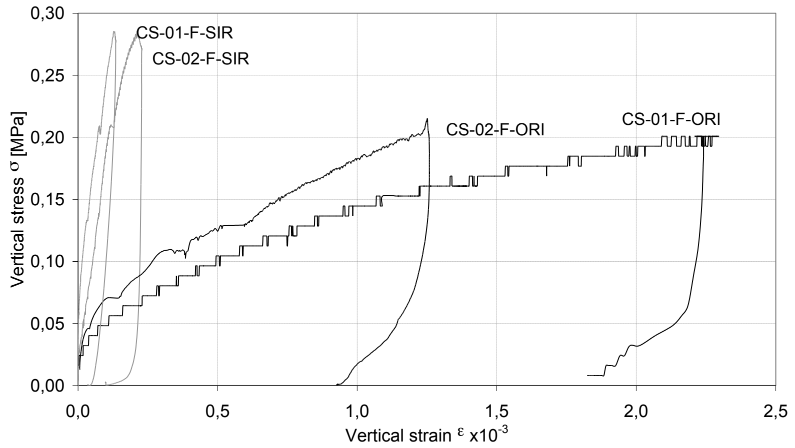


Figure 3.30 Simple compression test: vertical stress vs strain.

Table 3.2 Results of simple compression tests, (Binda et al., 2005) and (Corradi, 2008).

Test No.	Panel dimension (cm)	Vertical stress σ_0 (MPa)	Young modulus E (MPa)
CS-01-F-ORI	86x48x182	0.201	128.94
CS-01-F-SIR	86x48x182	0.286	4152.56
CS-02-F-ORI	86.3x48x180	0.215	305.56
CS-02-F-SIR	86.3x48x180	0.286	1770.43

Shear-compression tests

For the case of shear-compression tests, the results are reported in Table 3.3. The average value of the shear strength for the unreinforced panels is 0.086 N/mm^2.After repair by deep repointing and injection, the average shear strength was 0.304N/mm^2, which is about 3.5 times, the original value. For the non reinforced panels, the average elastic shear modulus was 51.69N/mm^2, while after repair is became 289.0N/mm^2. Here also a high scatter in the results was found.

Table 3.3 Results of shear-compression tests, (Binda et al., 2005) and (Corradi, 2008).

Test No.	Panel dimensions (cm)	Max shear strength τ_u (MPa)	Vertical stress σ_0 (MPa)	Shear strength τ_k (MPa) b=1.5	Shear elastic modulus G (MPa)
TC-01-F-ORI	86x48x182	0.083	0.1467	0.047	37.94
TC-01-F-SIR	86x48x182	0.412	0.2720	0.331	281.38
TC-02-F-ORI	86.3x48x180	0.089	0.1838	0.047	65.45
TC-02-F-SIR	86.3x48x180	0.196	0.2683	0.103	196.11

Diagonal compression tests.

These tests were carried out in the two buildings of Ponte and Trevi. The first two tests were performed on the Ponte building where a masonry panel was subjected to the diagonal tests before reinforcement (P-D-13-OR). The values are reported in Table 3.4. The shear strength was 0.059N/mm^2, while the shear modulus was 37.0N/mm^2. After repair by deep repointing and injection the two values became, respectively, 0.157 and 731.0N/mm^2. The high increase both in strength and stiffness is evident.

Finally the last two diagonal tests were carried out on the Trevi building (CD-26-T-ORI, CD-26-T-RIR). The panel with a thickness of 670.0mm was only repaired by deep repointing in order to check the influence of the technique when used alone. The results are also reported in Table 3.4. In this case the shear strength change from the unrepaired panel to the panel after repair, is from 0.045N/mm^2 to 0.054N/mm^2, which is only a very small increase in value. On the contrary, the shear stiffness changed from 79.6 N/mm^2 to 231.56N/mm^2. Figure 3.31a and Figure 3.31b show the comparison of the stress-strain behavior of the panels in Trevi and Ponte.

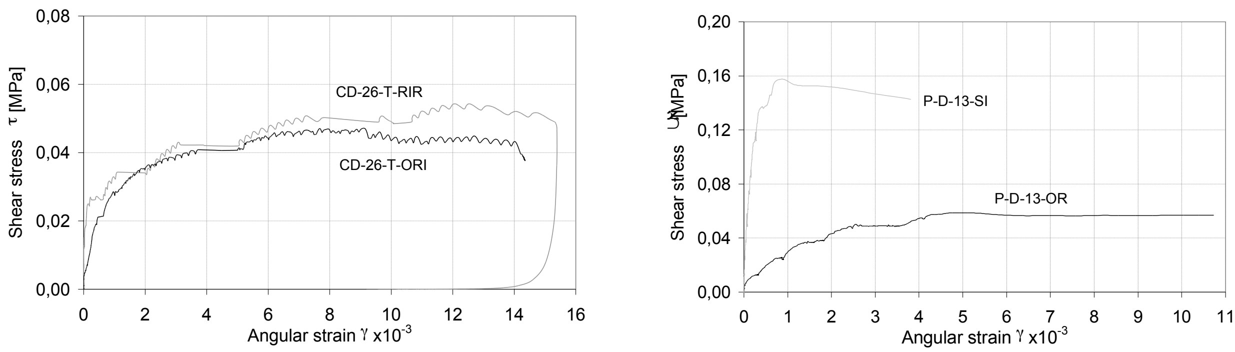
Table 3.4 Results of diagonal compression tests, (Binda et al., 2005) and (Corradi, 2008).

Test No.	Panel Thickness (cm)	Shear strength τ_K (MPa)	Shear elastic modulus $G_{1/3}$ (MPa)	Angular strain $\gamma_{1/3}$
P-D-13-OR	48	0.059	37	0.533
P-D-13-SI	48	0.157	731	0.070
CD-26-T-ORI	67	0.045	80	0.190
CD-26-T-RIR	67	0.054	232	0.076

From these results, a clear tendency is shown: the deep repointing alone can increase the shear stiffness of the masonry, while a significant increase in shear strength can be obtained by the synergic effect of repointing and grout injection. It is nevertheless important before using the grout injection technique to accurate examine its applicability. In fact when the technique was applied by the authors always a study of masonry injectability was carried out, (Laefer et al., 1996). A certain percentage of voids must be present in the masonry and no loose material or clay must be found inside the masonry section, in order to have a successful injection.

Inspection of the repaired panels.

After testing of the repaired panels, some of them where cut from the wall and taken to the laboratory in order control the penetration and diffusion of the repointing and of the injection. Figure 3.32a shows the cutting of the panel in different portions which were carefully examined to check the penetration and bonding of the repointing. Figure 3.32b and c show, respectively, the depth of penetration and the good bond between the new and the existing mortar. It is suggested that technique has a higher probability of being successful than the injection technique. Of course the two can be complementary.



(a) (b)

Figure 3.31 (a) Angular strain vs shear stress for the Trevi panel before and after reinforcement; (b) Angular strain vs shear stress for the Ponte panel before and after reinforcement (Binda et al., 2005), (Corradi, 2008).

(a) (b) (c)

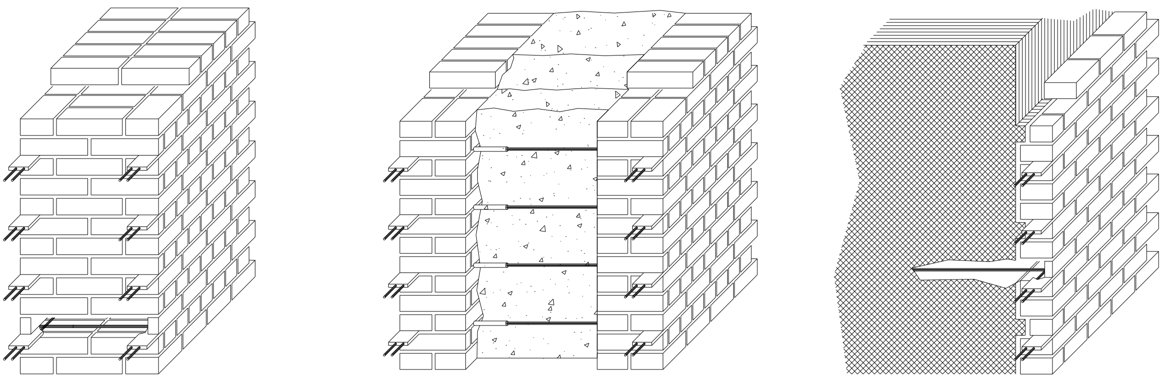
Figure 3.32 (a) Cutting of the panel in different portions; (b) depth of penetration; (c) good bond between the new and the existing mortar, (Binda et al., 2005), (Corradi, 2008).

<u>*Conclusions*</u>

The experimental results have shown that a deep repointing up to a depth of 70.0 to 80.0mm produces a significant increase of the shear strength and especially of the shear stiffness of masonry made with roughly cut stones compared to the unrepaired masonry. The results obtained for the diagonal compression tests carried out on the panel repaired by means of injection and deep repointing showed significant increase both in terms of shear strength and stiffness. However, it must be pointed out that the presence of cracks caused by the test on the unstrengthened panels facilitated the distribution of the injected grout within the panels. Deep repointing is more effective when the wall thickness is small and when coupled with injections (provided that the wall is injectable). The deep repointing is a very interesting technique for some masonry typologies.

3.1.4 Bed-Joint Reinforced Repointing

The reinforced repointing technique involves the insertion of reinforcement rods or plates inside the joints, in steel or fiber reinforced polymer, on one or both sides of the wall, eventually connected in the transverse direction. This originates a further reduction of the wall dilation and of the tensile stresses in the resistant elements; it is also useful to prevent the out-of-plane deformation of the external leaves of the multi-leaves walls and to increase the ductility and the ability to dissipate energy of the structure.



(a) (b) (c)

Figure 3.33 Reinforced repointing on (a) one leave masonry wall; (b) multi-leave masonry wall with resistant external leaves; (c) multi-leave masonry wall with external coating layer (Binda et al, 2001).

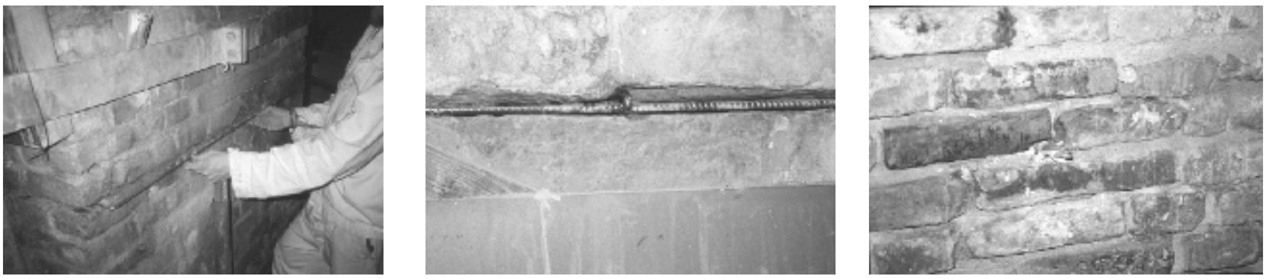
The described operation can increase the masonry resistance of both vertical and horizontal loads, but the best results are obtained especially in terms of deformation, which are also greatly diminished due to the confinement effect of the joints.

This operation involves the partial replacement of the mortar joints with better quality mortar, in order to improve the masonry mechanical characteristics, and it should be applied in the case the deterioration is localized only in the mortar. Prior to the application of this intervention technique it is necessary to first accurately characterize the original mortar from a chemical, physical and mechanical point of view, in order to choose the repointing mortar most compatible with the existing one. The filling mortar cannot be excessively rigid and resistant to avoid creating areas in the masonry with inhomogeneous behaviour. To perform the intervention, first the wall should be prepared, after which the joints should be repointed and refined.

First it is necessary to remove any existent plaster and scrape the joints (at least 5.0 to 7.0cm), the scrape operation, if possible, should be done with traditional tools and not with power tools, to avoid vibration and percussion harmful to the masonry, but the manual operation has a high cost in terms of time and therefore too expensive. Much attention must be paid not only to the depth of stripping, but also to the perfect elimination of the original layer of mortar that is in contact with the resistant elements in order to allow the new mortar introduced to develop ties of adherence with the elements themselves. The repointing can be done on one side or both sides of the wall, with a maximum depth, on each side, of 1/3 of the entire thickness of the wall, after verification of any signs of instability of the entire wall or of single portions of it. In the case of combined interventions on both wall surfaces it is best to perform the complete repointing on one side and then begin to intervene on the other side. After the removal of impurities and waste powder of the mortar, the wall should be washed with low water pressure.

After which, comes the phase of repointing with the insertion into the joints, each 30.0 to 50.0cm in the vertical direction, one or two steel or FRP reinforcement bars with a diameter of approximately 6.0mm, to increase the masonry ductility and the ability to dissipate energy. Normally are used two bars of limited diameter rather than a single larger bar, because, in ordinary masonry, the joints have dimensions between 10.0 and 15.0mm and, as so, do not allow the introduction of larger elements. The bars should be placed on a thin layer of mortar, and then be covered with hydraulic lime mortar, or with a additive or polymeric mortar, in which case, in order to ensure also a good aesthetic result, it should be included in the joint a final layer of finish, about 15.0 to 20.0mm deep, made with natural or pigmented material in order to reproduce the original appearance of the joints themselves, Figure 3.33. The final aesthetic operations are composed by scraping and cleaning of the joints, to remove the mortar smears/debris of the resistant elements.

The execution of this technique, Figure 3.34, should be as uniform as possible throughout the entire leaf object of intervention, and must proceed with continuity both vertically and horizontally.



(a) (b) (c)

Figure 3.34 Phases of the repair intervention: (a) insertion of the reinforcement; (b) detail of the wall before repointing, showing the transversal steel tie position; (c) view of the wall after repair, (Binda et al., 1999).

In the case of consolidation using repointing, the inability to maintain the original plaster must be taken into account, as so, this type of intervention cannot be use in the presence of fine plaster or frescos, i.e., in the case of buildings of historical and artistic importance.

The technique of reinforced repointing is not applicable when, for example, the joints are irregular, as in the case of most stone masonry structures. Instead, it could be useful for local or stitching and strengthening, as in case of tie anchoring.

Attention should also be given to the choice of the mortar to avoid unwanted chemical, physical and mechanical reactions: in general cement based mortars are used, as they provide higher strength, however, this type of mortars may trigger unwanted chemical reactions in the masonry.

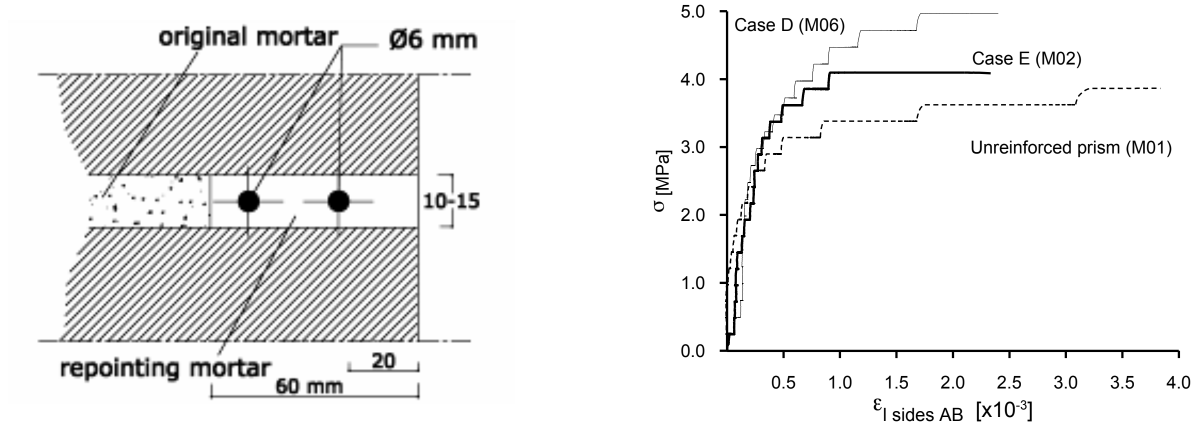
Is also crucial to perform the intervention in the depth of the masonry; it is frequent to found, in fact, a malfunction of this technique, because it wasn't well applied in depth, but limited to an aesthetic improvement of the surfaces rather than the increase of the mechanical properties.

In general, the repointing is ineffective in cases where there is a poor execution of the intervention.

Several experimental researches aimed at the control of the effectiveness of the technique to prevent or to repair long term compressive damages of masonry are available. The strengthening by confining steel bars, (Binda et al., 1999), (Binda et al., 2001), (Modena et al., 2002), (Valluzzi et al., 2005a) or CFRP thin strips, (Valluzzi et al., 2003a,b), (Tinazzi et al., 2003), (Saisi et al., 2004), (Valluzzi et al., 2005b), (Garbin et al., 2009) were particularly explored.

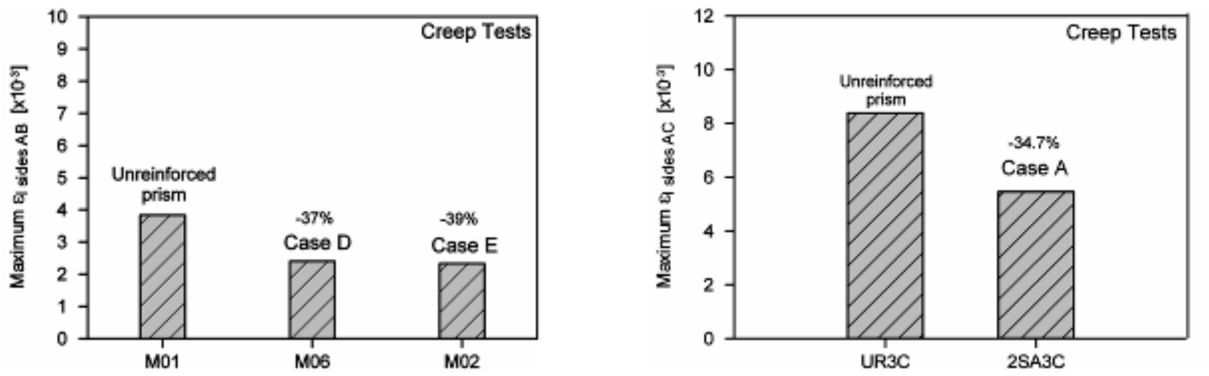
The bed joints reinforcement technique has demonstrated his effectiveness in the dilation control due to the cracking phenomena (Binda et al., 1999), (Binda et al., 2001) and (Valluzzi et al., 2005a,b). This goal is achieved by the insertion of reinforcing bars into mortar bed joints in order to bear the tensile stresses otherwise directed to the bricks and, consequently, to reduce the dilation of the wall. The main operative phases for a proper execution of the technique are widely reported in (Binda et al., 2001), (Modena et al., 2002) and (Valluzzi et al., 2005a,b). At first, stainless steel rebars were considered and embedded into horizontal mortar joints with suitable repointing mortars at every three brick courses (Figure 3.35), (Binda et al., 1999), (Binda et al., 2001) and (Modena et al., 2002). Laboratory experimental tests simulating both monotonic and creep loads, carried out on strengthened and plain masonry panels, showed a significant reduction of the lateral dilatation of about 37-39% (Figure 3.36a and Figure 3.37). Moreover, a meaningful reduction of the crack pattern was also detected (Binda et al., 2001) and (Saisi et al., 2004). A further development of the technique involved the use of CFRP rebars, in place of steel ones, and both lime-based and epoxy mortars (Figure 3.37). CFRP rebars were used in order to evaluate their effectiveness with compatible or high specific performance embedding products (Valluzzi et al., 2003a,b) and (Saisi et al., 2004). Results pointed out that the better performances were obtained with symmetric applications and that the use of high strength epoxy resins as embedding material can be inappropriate, due to the more brittle behaviour both at local and global level. Both the types of reinforcement do not influence the strength of the masonry. This confirms the suitability of the bed reinforcement technique for the counteraction of the peculiar damage occurring on overstressed masonry structures. Both reduce the lateral deformation or dilation but the steel seems to be more effective (Figure 3.36-Figure 3.37-Figure 3.39). Reduction of lateral deformation is around 40% for steel with reinforcement positioned every three joints and less than 35% for the use of FRP strips, but at every joint, (Figure 3.36). CFRP strips strengthening at failure seems to cause a higher damage then steel bars insertion, (Figure 3.40). The combination of the small width of the strips and of the high modulus of elasticity of the fibers probably provokes, in fact, an uneven distribution of stresses with concentration in the limited zone located underneath the reinforcement. Application of small diameters steel bars presents higher applicability for strengthening and repair of existing structures suffering high sustained loads effect than FRP materials. The steel confinement, in fact, despite its lower mechanical characteristics, showed a more gentle effect on masonry at the final steps of the test. This could be an important parameter in order to evaluate the repair possibility of damages.

In (Modena et al., 2002) the preliminary research and the application of bed joint steel reinforcing coupled with other global reinforcing technique, to repair an historic bell-tower damaged by the permanence of high state of stress due to compression are described in detail. In (Binda et al., 1999) the application of steel bars is aimed at the confining of a pillar, Figure 3.34.



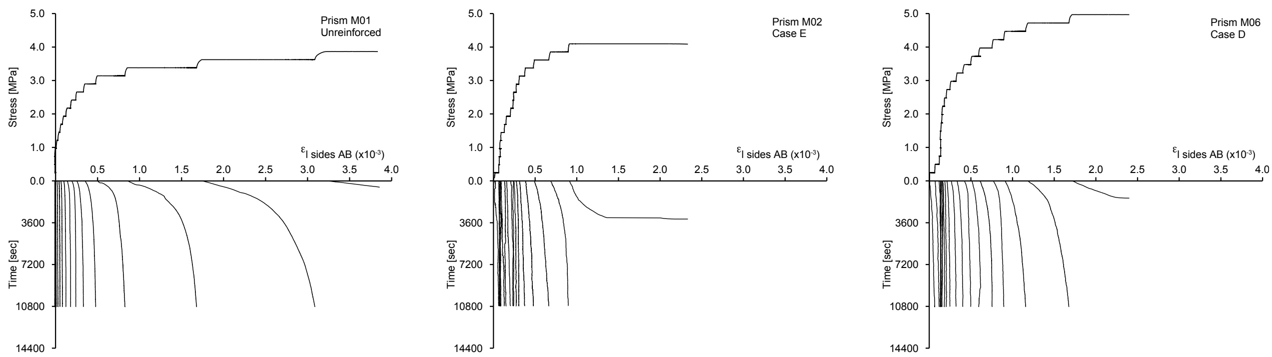
(a) (b)

Figure 3.35 General scheme of the reinforcement technique with steel bars and horizontal deformation measured on two sides of the wallettes reinforced with steel bars.



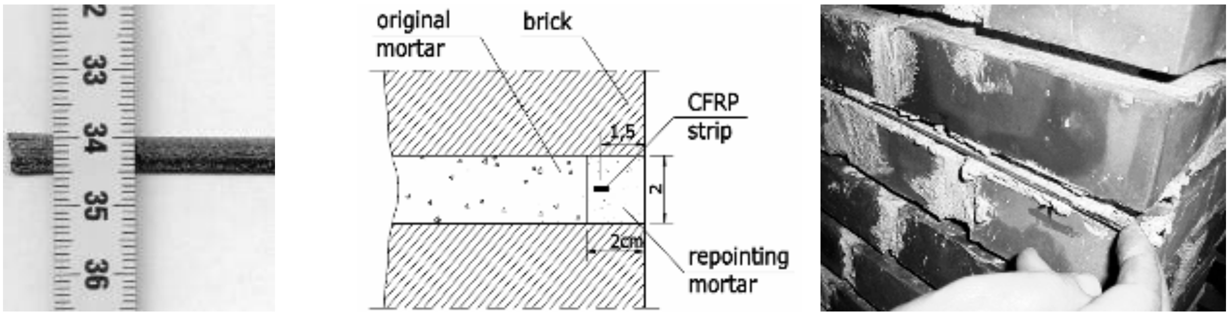
(a) (b)

Figure 3.36 Maximum transversal deformation measured during the creep tests carried out on (a) steel reinforced wallettes and (b) CFRP strip reinforced bars, (Binda et al., 2001), (Saisi et al., 2004).



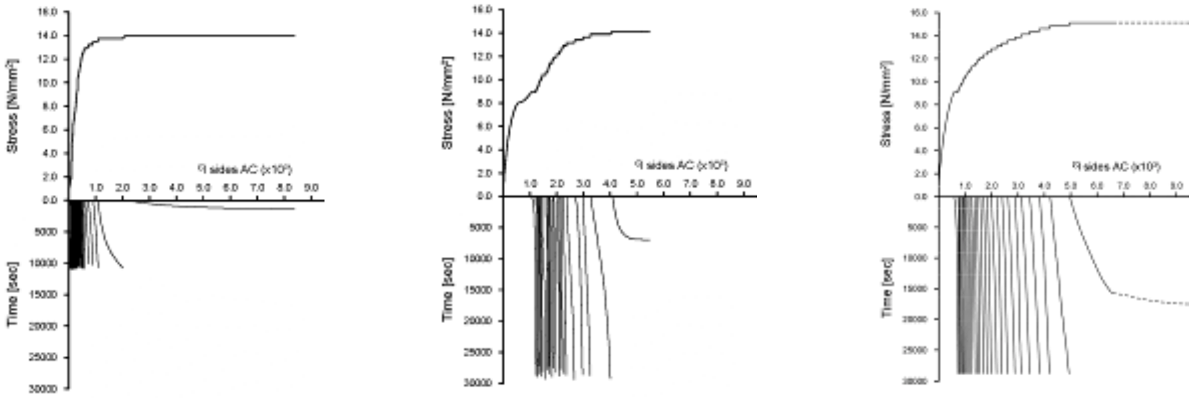
(a) (b) (c)

Figure 3.37 Compressive stress and time vs. horizontal deformation of sides A and C of the prisms: (a) unreinforced; and (b) (c) reinforced by steel bars, (Binda et al., 2001).



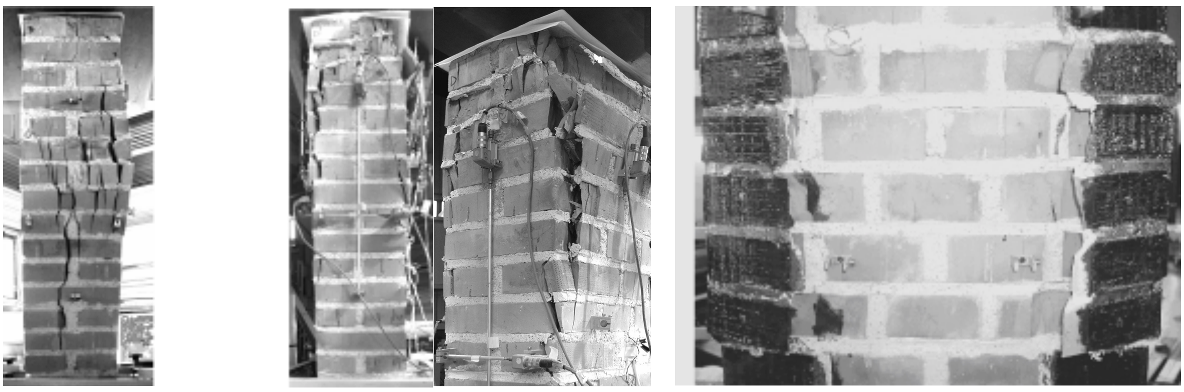
(a) (b) (c)

Figure 3.38 Bed joints reinforcement technique: (a) CFRP thin strip, (b) reinforcement positioning detail, (c) insertion of a CFRP thin strip in a masonry panel, (Saisi et al., 2004) and (Garbin et al., 2009).



(a) (b) (c)

Figure 3.39 Compressive stress and time vs. horizontal deformation of sides A and C of the prisms: (a) unreinforced; and (b) (c) reinforced by CFRP strips, (Saisi et al., 2004).



(a) (b) (c)

Figure 3.40 (a) Crack pattern of the unreinforced specimen, (b) (c) crack pattern of CRFP reinforced panels with material spalling and, (c) with peeling of CFRP sheet (Saisi et al., 2004).

3.1.5 Anti-expulsion tie-rods

The technique of inserting metal ties inside the walls has as main purposes to limit the deformation of the walls and to improve the shear behaviour of the same. This technique has as main advantages:

1. High velocity of execution;

2. Low cost of execution;
3. Good performance;
4. Partial removable intervention, in case that a better consolidation solution is found.

The two types of rods to be used to obtain these results differ on its positioning on the walls: the first ones are placed transversely to the thickness of the wall, the second ones parallel to the wall surface, vertically or horizontally.

On the first case the steel ties are placed on the perpendicularly direction to the masonry. The holes to insert the ties (diameters ranging from 4.0 to 8.0mm) are done with a rotating probe at the mortar joints, (Figure 3.41). The ties consist on simple steel bars with improved adherence or treated bars with a bolted head, (Figure 3.41). Eventually it is possible to insert the bars by hammering, taking advantage of the cracks\lesions present in the walls.

When using simple steel bars, they are fixed to the masonry by bending or injection, in this case the action of consolidation is noticeable only when the deformation of the wall undergoes an increase, in consequence of which the steel bar is loaded.

When using threaded bars, upon the insertion moment, a warming of the bar and a slight tightening can be performed, this way the contraction that occurs due to the cooling process immediately puts tension on the bar, which immediately exercises an anti expulsive action on the wall leaves.

To exercise these contrast actions to the masonry transverse deformation, the tie rods can be placed not only normal to the faces of the wall itself, but also along the diagonals. It should be inserted 4 ties per square meter, or at least it should not be less than 2 per square meter.

The ties for the improvement of shear strength of masonry are inserted into holes parallel to the wall surface, at the center of the wall, vertically or horizontally. This intervention could have serious applicability problems to multiple leaves stone masonry. After being inserted they are, by any of the available techniques, pre-stressed in order to create a uniform state of compression in the masonry, which reduces the main tensile tension generated by the main horizontal actions acting on the wall's plane.

Usually it is possible to observe a good functioning of both types of ties, it is a type of technique that presents a noticeable low execution time and cost. In particular environments protection of the bars against the corrosive action of external agents has to be ensured.

In the case of the ties for pre-stressing of the masonry, special attention should be paid to the application of the actions to the bars and, not to lose the pre-stress effect, if the holes were not injected and the anchoring systems are accessible, it is possible to perform a periodical control of the state of tension in the bars.

Figure 3.41 Positioning of anti-expulsion tie-rods.

3.1.6 Insertion of artificial headers

Insertions of artificial headers in R.C. or steel are proposed in technical literature and handbooks. The aim of the application is to reinforce multiple leaf masonry by the insertion of connecting elements.

In a passing through hole drilled in the masonry, the steel reinforcement is placed and then injected. The technique is rather invasive and requires the presence of several headers in a wall to be effective. Furthermore, other difficulties could concern the real possibility of connecting the wall and stress transferring.

3.1.7 FRP/SRP/SRG application

FRP/SRP/SRG materials that greatly meet the concept of 'engineered material', are being more and more applied for the restoration of vertical and horizontal structural elements as well as isolated monuments (statues, etc…). This reinforcement is made of different kinds of fibers (carbon, glass, polyvinylalcohol, etc...) impregnated in a polymeric matrix. Due to high strength and stiffness-to-weight ratio, fatigue and corrosion resistance, and their exceptional capacity to be tailored to specific needs, along with decreasing raw materials and production costs, advanced composites have progressively extended their initial fields of applicability, typically limited to the aerospace sector.

Their flexibility and somewhat easy application allow a wide range of intervention scenarios for existing structures, characterized by different damage conditions. The state of art is generally limited to the evaluation of the mechanical properties, without the exploring of the real behaviour of the assemblage in the natural aggressive environment, generally characterises by the presence of moisture and salt, thermal cycles, frost-defrost actions.

As demonstrate by the application of surface treatment, masonry could be damaged by the salt crystallisation, progressively spalling the treated superficial layers. Furthermore, at present, the local mechanisms involved in the failure of masonry structures strengthened by FRP laminates (such as delamination, etc...) still need to be experimentally and numerically deepened. For SRP/SRG materials a few first applications for concrete and less for masonry are available, but no guidelines are available. Testing and numerical activities to be carried out during the project, integrated for FRP by the data already being collected into the data-base of the dedicated RILEM committee, will produced developed design methods for FRP and new design methods for SRP/SRG.

Other meaningful limits concern the application procedure, which requires smooth surfaces and the removal of the ancient plaster.

As in case of r.c. jacketing, the intervention could be effective if applied uniformly on the building, on two sides of the wall and with a peculiar care to the corners, floors and to the overlapping of the materials.

The application requires:

- the removal of the plaster, cleaning by sandblasting;
- application of primer resin, organic or polymeric;
- application of a epoxy mortar layer or of other mortar to smooth and regularise the surface;
- application of the first layer of epoxy resin;
- application of the first layer of laminate, second layer of epoxy resin, and of the second laminate layer according to the selected layout;
- eventual coating.

Figure 3.42 Layout of a FRP reinforcing.

During the project, several materials and techniques will be re-engineered, updated, controlled, modified and optimised. New application of advanced materials, innovative combined use of new and existing materials/components, in order to 'exploit' the inherent properties of the existing (usually rather weak, frequently deteriorated and/or damaged) materials/components, and innovation in technologies and design procedures will be fostered into the project.

Valluzzi in (Valluzzi et al., 2002) conducted an experimental campaign on brick masonry panels strengthened by Fiber Reinforced Polymer (FRP) laminates that was aimed at investigating the contribution of FRP strips on the shear behavior of clay brick wallets. A series of nine unreinforced masonry (URM) panels and 24 strengthened panels have been subjected to diagonal compression tests, considering different reinforcement configurations, Figure 3.43.

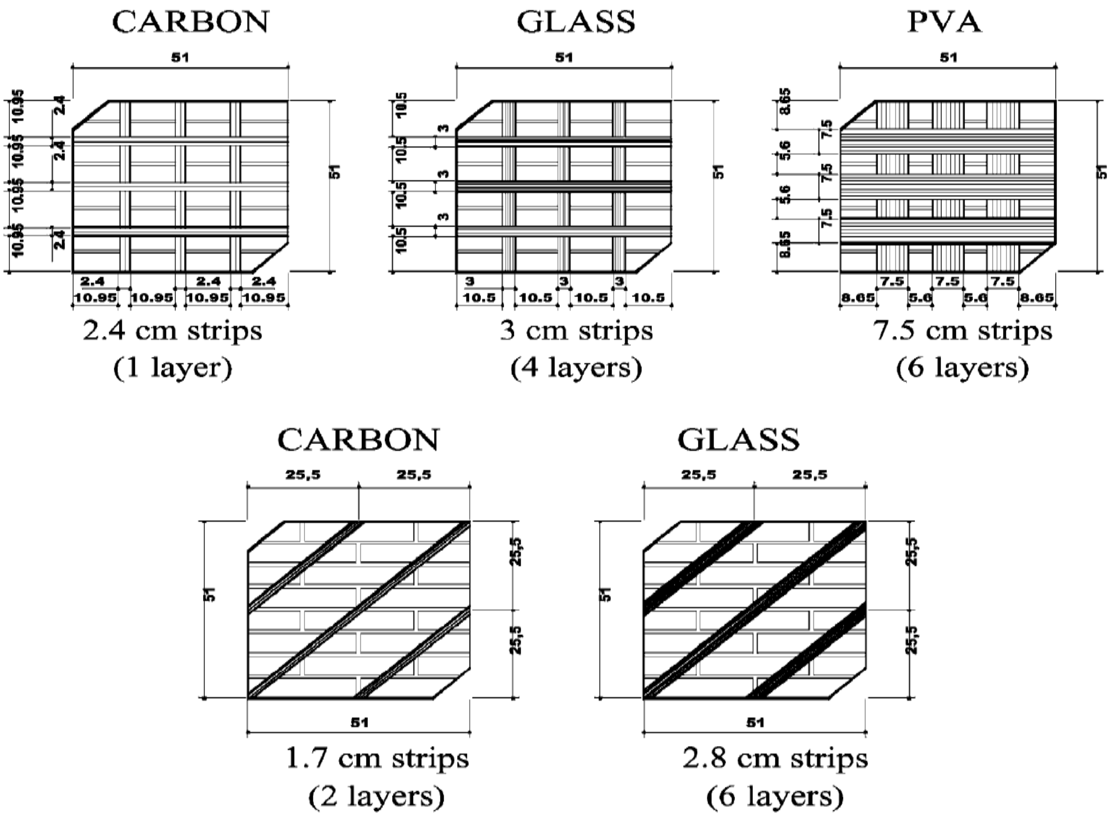


Figure 3.43 Strengthening solutions. Single-side strengthening patterns or double-side strengthening patterns, (Valluzzi et al., 2002).

The experimental campaign allowed withdrawing the following conclusions:

1. FRP reinforcement applied only at one side of the panels did not significantly modify the shear collapse mechanisms (diagonal splitting) of the URM; while double-side configurations provided a less brittle failure and a noticeable ultimate capacity increase.
2. The diagonal configuration is more efficient in terms of shear capacity than the grid set up; however, the latter offers a better stress redistribution that causes a crack spreading and a less brittle failure.
3. In most cases, less stiff FRP material appeared to be more effective both in terms of ultimate strength and stiffness (not reported for brevity) increase of the panels. That was due to the particular design criterion used (weaker material has a larger adhesion area), and also to the fact that stiffer material is more vulnerable to de-bonding, especially when the number of plies increase.

4. The type of test conducted and the specimen dimensions appear an easy and efficient system to check elementary strengthening configurations.

3.1.8 Confinement for columns and pillars

Confinement is a basic technique to overcome the horizontal actions on an element or a structure that suffer from lesions caused by compression. It increases the compression strength capacity and improves the stability of structural element or global behaviour of the structure. Steel and advanced polymers are of the common materials used for this purpose.

Flexural actions on slender elements could involve local compressive concentration which could require the repair by confinement, (Figure 3.44). The application is also addressed to prevent collapse of long term overloaded elements or to repair their damages, as in case of pillars or towers, (Figure 3.45). The intervention is aimed at the control of the transversal dilation by confinement actions.

Traditional technique of confinement with metal rings/strips are applied on pillars and columns in the cases where cracks, spalling, crushing and bulging due to over compression is observed. The method can be fully reversible depending on the material used and on the tension level and effective in relieving the pillar or column. It should be remembered that if column or pillar is to be subjected to environmental effects, the selection of the material becomes very important so that it does not corrode or interact with masonry resulting in deterioration. A steel ring needs to be prepared for the appropriate dimensions of column in a way that it does not need any action on masonry like anchoring or fixing with nails etc. Then the rings are placed in critical sections.

In general the application is possible by metal elements (iron, steel, SMA…) or more recently by FRP stripes. Emergency confinement could be done by polyester bands.

The technique is very effective and has a long tradition in case smooth rounded columns, while requires special care to the stress distribution in case of square or irregular pillars, (Figure 3.44 and Figure 3.45).

On large structures made by regular masonry, the confinement actions could be obtained by the joint steel reinforcing technique, afore described, (Figure 3.46).

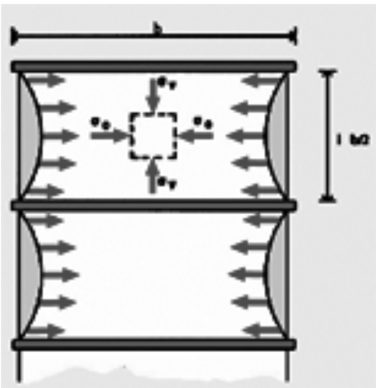


Figure 3.44 Confining action of compressed elements, (Source: POLIMI).

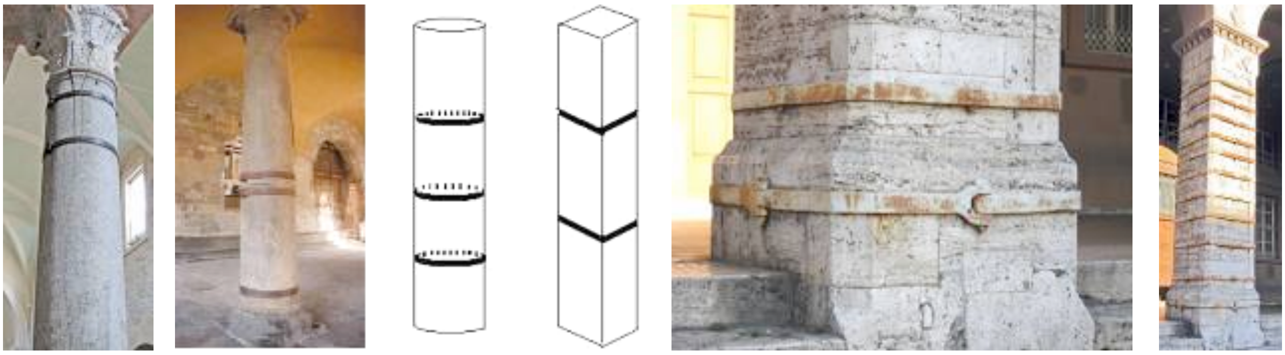


Figure 3.45 Example of confined columns and pillars.

Figure 3.46 Bed joint reinforcing of a compressed pillar, (Source: UNIPD).

Regarding the test evidence of the confinement method with steel strips/ties, the work carried out by Farooq (2006, 2008) on masonry wall panels and masonry members has shown considerable increase in compressive and shear strength due to application of surface mounted light weight steel strips (45.0mm × 1.3mm) .

An experimental study by Ilyas et al. (2009) based on Farooq aimed at investigate the behaviour of masonry columns before and after strengthening using steel strips reveals that steel strips are effective in increasing the compressive strength the percentage of which depends on the cross sectional aspect ratio of the brick column. There observed significant improvement in axial strength although the ratio of confining pressure to unconfined stress was not very high. The direct contribution of steel in strength enhancement was less and main enhancement in strength came from the confining effect of steel for columns. Figure 3.47 depicts the stress versus strain both in the lateral and axial direction of the specimens in which a positive sign indicates lateral dilation while a negative sign indicates compressive axial strains and UC is unreinforced column, MRC is moderately reinforced column and CFR is heavily reinforced columns. Accordingly, the confinement increased the ultimate stress however the ultimate lateral strain of confined columns is less than that of unconfined columns.

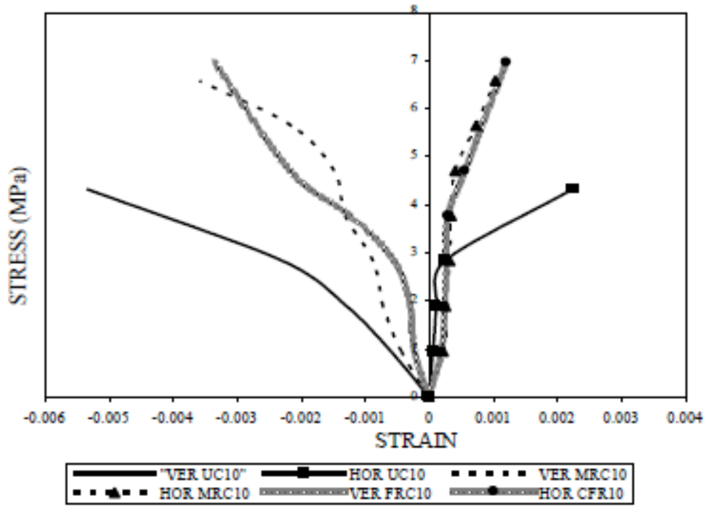


Figure 3.47 Stress versus strain both in the lateral and axial direction of the specimens in which a positive sign indicates lateral dilation while a negative sign indicates compressive axial strains. UC is unreinforced column, MRC is moderately reinforced column and CFR is heavily reinforced columns.

3.1.9 Vertical stiffening of earthen building elements

3.1.9.1 Centre-core rods

Several researches are on going on the topic and not yet extended on real case histories.

Retrofitted steel or fiberglass centre-core rods can be used to resist overturning and stiffen earthen walls, as shown by shaking table tests, (see Tolles et al., 1996).

Tests have shown the effectiveness of vertical centre-core rods installed in the adobe walls by means of epoxy grouts both in delaying and in limiting damage in both in-plane and out-of-plane adobe walls. Crack opening and crack propagation were delayed or arrested in the case of in-plane damage, whereas for out-of-plane damage they acted as reinforcing elements, with the epoxy grout providing effective shear transfer between adobe and rod.

The use of centre-core rods with full diaphragm support at the tops of the walls is particularly recommended by Tolles et al. (1996) for slender adobe walls (height/thickness ratio of 8), mostly to resist overturning. For such walls, the bond beam or roof diaphragm is said by Tolles et al. (1996) to be the most important factor in determining the dynamic behavior and, ultimately the stability of the building. Removal of rods results in loss of historic material.

The technique has been applied to adobe in laboratory prototypes only and its effectiveness for rammed earth and cob has not yet been assessed, as well as on real cases. Problems could concern the applicability and execution phases, starting from the drilling and the bond effectiveness between epoxy grout-earth material - rod.

Furthermore, compatibility of the materials in terms of stiffness should be accurately evaluated as well as durability, particularly related to use of epoxy grouts.

3.1.9.2 Earthen Walls: Cables and straps

The disadvantage of loss of historic material which is caused by the partial reversibility of the retrofitting by means of centre-core rods can be solved by using polypropylene strapping instead of centre-coring, as long as surface renderings allow so.

Cables and straps are used to strengthen earthen walls with the intent of limiting the relative displacement of cracked wall sections and enhancing out-of-plane flexure and in-plane stability.

Tolles (2006) claims that strengthening by means of cables and straps can prevent extensive damage in all types of adobe buildings if the connection between the roof or bond beam system is adequate.

Post-tensioning of such straps is proposed as yet another means to enhance the seismic response of earthen buildings (for rammed earth, see Hamilton et al., 2006) without having to introduce steel elements inside the wall.

The aforementioned techniques are proposed up to now only on prototype, and studied for mechanic aspects. Durability and applicability on real cases are not yet studied.

3.1.10 Others

3.1.10.1 Geotextile Mesh as containment reinforcement to Adobe Walls

Since 2003, dynamic testing at PUCP has moved away from the testing of steel-mesh-reinforced earthen structures, (see section 3.1.2.1), the limits of which are described in the previous section, and focused, instead, on the testing of polymer mesh reinforced walls (mesh is applied to both sides of the wall and connected through the wall thickness), the advantage of which (Torrealva et al., 2009a) lies in its compatibility with earth as a building material, and with its ability to provide ductility at high deformations, (Figure 3.48 and Figure 3.49). Cyclic testing has shown that, the initial shear strength and stiffness of walls reinforced with mesh increase significantly when mud plaster is applied over the mesh, (Torrealva et al., 2009a,b).

Also in this case the technique was tested only on new laboratory prototype and considering only mechanic aspects. Durability and applicability on real cases are not yet studied.

Figure 3.48 Adobe test-house reinforced with polymer mesh and partially plastered.

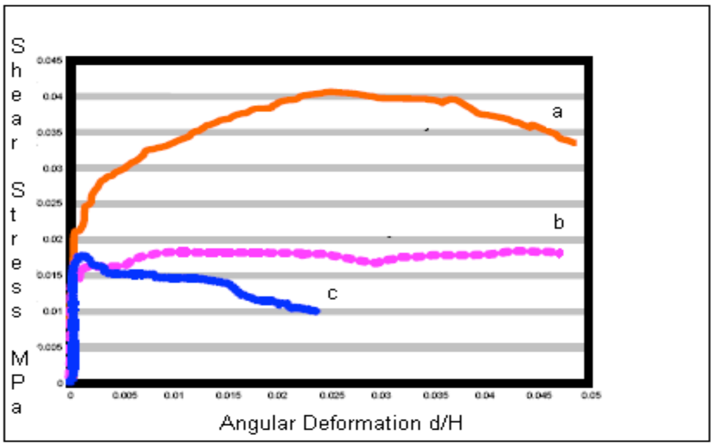


Figure 3.49 Comparison between compressive strength (Y-axis) and Angular distortion (X-axis) of: 1) Adobe wall without polymer mesh - Blue line, 2) Adobe wall without plastering- Pink line, and 3) Adobe wall with polymer mesh reinforcement- Orange line. (Torrealva et al., 2009a,b).

3.2 LOCAL REPAIR OF CRACKS OR OF DECAYED PORTIONS

3.2.1 Local Dismantling and Reconstruction (“scuci-cuci”, i.e. “unstitch-stitch”)

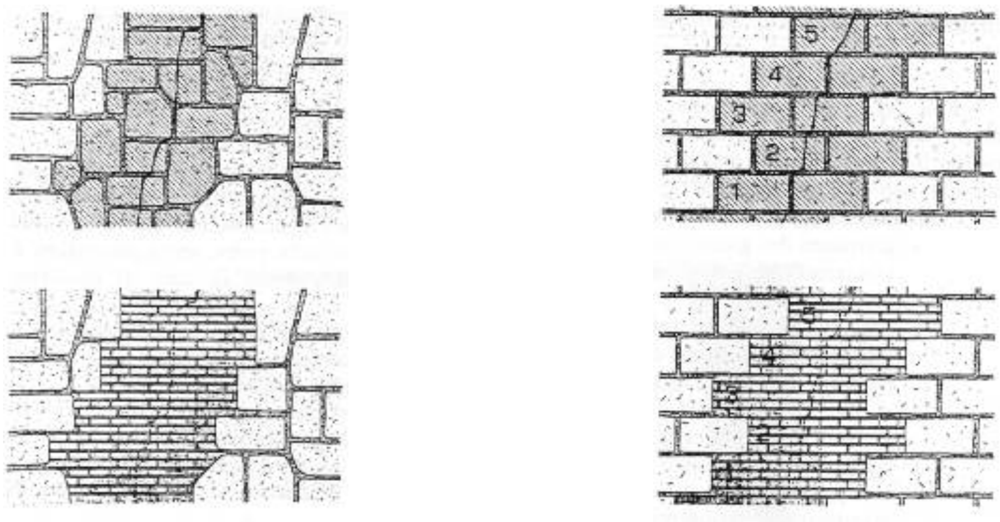
The local dismantling and rebuilding (“scuci-cuci”) methodology aims to restore the wall continuity along cracking lines (substitution of damaged elements with new ones, reestablishment of the structural continuity) and to recover heavily damaged parts of masonry walls. The use of materials that are similar, in terms of shape, dimensions, stiffness and strength, to those employed in the original wall is preferable. Adequate connections should be provided to obtain a monolithic behaviour. The effectiveness of the intervention is strictly connected to the recovering of the previous wall properties; otherwise the seismic actions could expel the intervention

This intervention consists on the rebuilding or replacing part of the masonry, locally damaged, Figure 3.50. The goal of this technique is to restore the structural integrity of the wall portion.

With this technique it is possible to: (1) intervene individually, to compensate for a crack\lesion on the masonry or for several lesions spaced apart, or, more seldom, (2) to intervene in a more extensive way, in the case of cracks spread throughout the structure and located close to each others. This last intervention should be discouraged being in contrast with the principles mentioned in Section 2.1.

The execution methods also differ depending on the lesion extension and the masonry typology. If the lesion is relatively small and affects only one leaf of the wall, the intervention could only be performed on the affected side. If the lesion is passing through the wall section, the intervention should be performed by gradually replacing the resisting elements on one side only, if it is a two-headed wall, or by acting on both sides in a coordinated manner, in the case of larger thicknesses.

In general, in the “scuci-cuci” interventions, in the replaced masonry, good quality bricks or stone and mortar are used, with a specific care to the compatibility of the new portion with the remaining of the wall structure. Sometimes, if the old masonry units are still in good condition, the original material could be directly re-used.



(a) (b)

Figure 3.50 Masonry replacement intervention. (a) Regular stonework. (b) Regular squared block masonry.

The intervention of substituting the masonry is composed mainly of a preliminary phase, and then of the execution phase in which the elements are replaced.

Before proceeding with the substitution intervention it is necessary to analyze the resistant elements, the units and the mortar that composes the masonry and their arrangement, in order to guarantee the compatibility of the new masonry portion, selecting the new elements to be introduced and, in particular, to achieve a mortar compatible with the mechanic, chemical and physical characteristics of the masonry. As so, it is necessary to establish the element replacement order, by subdividing the wall in sub parts in which to work alternately, finally the unsafe structural elements or the ones that may transmit forces to the areas to be treated are supported, to block the effects due to momentary weakening of the masonry.

The operation is executed by scrapping the joints and removing the units without causing percussion or vibration; the removal of the old masonry can be done manually "pull", or, if the thickness of the wall and the loads are high, through hydraulic jacks welded to steel sections, to ensure the supporting conditions in the area to be replaced. Hand in hand with the elimination of the old elements is the introduction of the new ones, from the bottom up and trying to obtain maximum penetration and the highest possible arrangement between old and new. However, one must be careful not to generate states of coaction due to excessive contrast, even though it is possible to include any pushing elements in the presence of large vertical loads or large sections of masonry. Finally, the area between the new and old masonry is sealed, after leaving time for the new masonry to adjust/settle itself.

Before removing the supports, it is advisable/necessary to check the curing process of the last reconstructed mortar joints in order to avoid the inevitable settling of the new masonry due to mortar shrinkage and to the progressive loading, which are harmful. To this end it is convenient to execute joints of small thickness, to limit the reductions in volume of the mortar itself, and gradually load the rebuilt portions of masonry allowing this way a continuous adjustment of the tensions as they it normally does during the construction of new masonry.

This technique gives good results, provided that (1) it is respected the arrangement and that materials as similar as possible to the original ones used of and also (2) that particular care on the connection arrangement between old and new masonry is taken.

This intervention technique is aimed at restoring the structural continuity of the masonry, as so, its malfunction\inefficiency is usually due to (1) poor execution, that leads to the creation of areas with higher stiffness than those adjacent to it (Figure 3.51); to (2) the lack of care on the connection arrangement between the masonry portions and (3) the scarce attention to the compatibility of the materials.

(Binda, 2003) describes criteria and procedures used in the choice of adequate materials in the reconstruction of the structural elements of the Cathedral of Noto, collapsed in 1996.

Figure 3.51 Masonry replacement intervention. The repairs are ineffective due to the lack of compatibility between the masonry portions (source: POLIMI).

3.2.1.1 *Earthen construction materials*

Cracks in earthen construction can be repaired by applying the “scuci-cuci” technique with an attention to the use of compatible materials, as applied by ZRS in the Al Damarki, UAE, as shown in Figure 3.52. No specific reference concerning the use of this technique or its effectiveness has been found in the literature. However, the aforementioned observation could be valid for any material.

(a) (b)

Figure 3.52 Application of the “scuci-cuci” technique to the gate at Al Damarki as crack repair, (source: ZRS).

3.2.2 Grout injection of cracks

Repair and strengthening by grouting of brick and stone masonry walls has been largely applied in Italy on historic buildings and dwellings in the seventies and eighties, after the main earthquakes of Friuli and Irpinia; nevertheless no great effort was done in advance and during the time to test the effectiveness of this technique.

This intervention consists on injecting grout mixtures of different types into the masonry, using particular techniques, in order to:

1. Reconstitute the structural continuity of the masonry;
2. Increase the masonry homogeneity, filling voids, if existing;
3. Improve the masonry mechanical properties, (increase the masonry strength);
4. Fill cracks in wall and other masonry structural elements (vaults, domes, etc…).

The choice of the mixture to inject is done by selecting the best characteristics for the type of wall and the crack extension on which to intervene; the mechanical strength of the mixture and its deformation characteristics (Elastic Modulus and Poisson's Ratio) should be similar to those of the original wall. To obtain a high penetration of the mixture, the size of the composing materials should be as small as possible and homogeneous and the mixture should present a low-viscosity in the fluid state, the curing and hardening period should be suitable for the execution of the intervention and the shrinkage must be limited, the chemical properties of the binder should be maintained throughout time and should be compatible with those of the original mortar, the mixture should not be soluble in water, should not vary in volume with moisture and should not obstruct the movement of vapour; finally, the material used to create the mixture should be easily found and inexpensive. Gypsum mortars or stones rich in sulphate require particular grout composition.

The technique is effective in the crack filling (Figure 3.53 and Figure 3.54), being most simple the control of the grout diffusion. The main limit of the technique is related to the frequent lack of voids in masonry or in the lack of communication in case of diffused cracks, and the difficulty of the grout to penetrate into thin cracks (2.0-3.0mm), even if microfine binders are used, (Figure 3.55). The grout cannot spread throughout the section or filling other cracks. In this case the technique is ineffective.

The injection procedure aimed at to fill a crack could be summarised as following:

1. study of the grout;
2. choice of the injection points, the distance between injectors and their layout, according to the crack diffusion, depth, with and localisation;
3. removal of the damaged plaster and superficial crack filling (to avoid loss of grouts);
4. positioning of the injection devices and repointing by mortar
5. preliminary water injection in order to remove dust and disaggregate materials but also to saturate the wall, avoiding the masonry suction. The segregation and shrinkage of the grout due to the high rate of absorption of the material should be prevented.
6. evaluation of the injection pressure
7. grout injection, starting from the lower part of the crack.

Preliminary applicability tests on selected area are highly recommended, as well as their visual inspection by local dismantling.

Section 3.1.1 presents a deeper description of the methodology.

An experimental research programme supported by EU from 1988 to 1991 has been developed by POLIMI in collaboration will the Laboratoire Central des Ponts et Chaussees in Paris and the Polytechnic of Athen.

Different kind of grouts were used by Binda and Baronio in Milan (Binda et al., 1988), (Binda and Baronio, 1989) to inject masonry prisms previously cracked under compressive tests, and to evaluate their effectiveness in repair cracks: epoxy resins, polymeric grouts, cementitious and hydraulic grouts were injected and tested mechanically and under ageing, (Binda et al., 1990), during four subsequent steps of the research. During the first step a rough technique was used to inject an epoxy resin and a polymeric grout into the prisms. During the second step of the research two new epoxy resins were proposed, the technique of injection improved and the injected specimens were also subjected to durability tests. When resins were injected on wet masonry, problems occurred on the effectiveness of the bond strength and on the durability of thermal cycles, even in laboratory conditions. The durability under thermal cycles even in dry conditions depends on the resins physical and mechanic properties.

Finally during the last two steps two cementitious grouts studied by LCPC in Paris and an hydraulic grout were used to inject previously damaged masonry prisms, (Binda et al., 1993a,b). After injection the prisms were subjected to compression tests and their strength and stiffness was recorded and compared to the ones shown when undamaged. The penetration and diffusion of the grouts into the damaged masonries and the different crack patterns were also studied both by applying sonic and ultrasonic pulse velocity, (Berra, 1991), and cutting of the prisms into slices.

In (Baronio, 2002) the whole campaign of investigation on the materials sampled from an ancient bell-tower and the formulation of compatible new grouts is detailed. The research is aimed at the infilling of the cracks, mainly related to the compressive damage of the masonry.

Figure 3.53 Crack injection, (source: POLIMI-UNIPD).

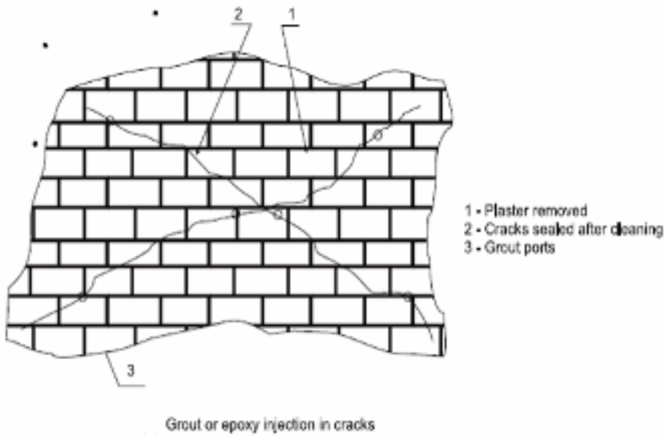


Figure 3.54 Grout injection in cracks, (Modified from IAEE, 2004).

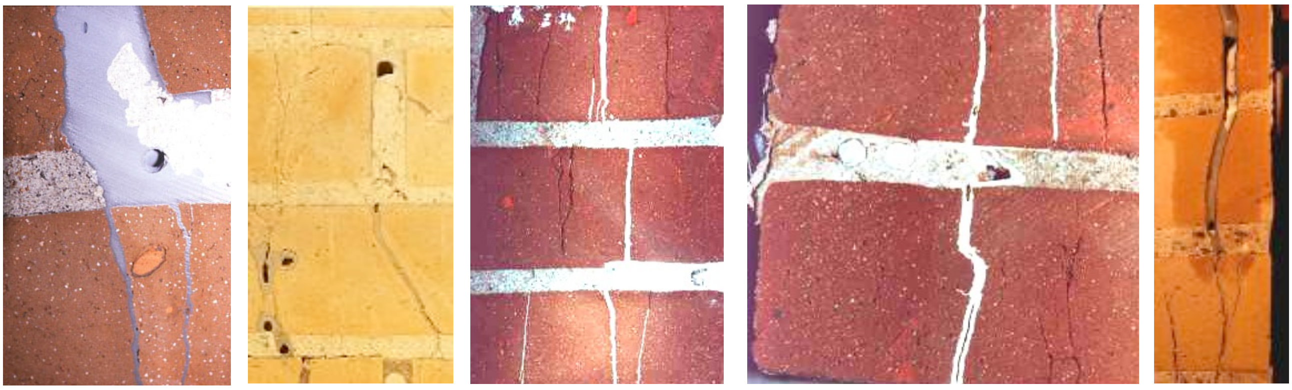


Figure 3.55 Compression cracks injected: the cracks are in some case are filled only partially, (Baronio, 2002).

3.2.2.1 Injection on earthen construction materials

Cracks in earthen construction can be repaired by grouting, (Figure 3.54), to re-establish structural continuity, and can also be used to consolidate voids and gaps, or as a complement to other strengthening techniques such as the introduction of tie-rods. Grout requirements are workability, low shrinkage, good bond to original material, chemical and mechanical compatibility with original material, and durability (see also Silva et al. (2009) for grouting of earthen materials).

Warren (1999) provides a general summary of grouts for earthen materials, and favours the use of pulverized fly ash with lime as a means to repair cracks. Grouts developed in the studies proposed by the literature are composed primarily of modified soil with the addition of either stabilisers, deflocculants, additives, aggregate or fibres to reduce shrinkage, increase strength, and when feasible increase adhesion. Typical stabilisers are lime (Fagundes de Sousa Lima and Puccioni, 1990), hydraulic lime (Nardi, 1986), and lime with fly ash (Roselund, 1990). Grouting methodology is based on low-pressure pumping system or gravitation.

A practical example of applying grouting to strengthen an earth structure is presented in (Tolles et al., 1996): the Pio Pico Adobe, repaired in 1991 by means of an adobe-flyash grout, was found to suffer little damage during the 1994 Northridge Earthquake in California.

While more recent studies present the effectiveness of grouting for brick and stone historic masonry buildings (Vintzileou and Adami, 2009; Bras et al., 2010) and (Sepulcre-Aguilar 2010), which describes the use of lime-based mortars with the addition of pozzolans), the effectiveness of grouting for earthen materials is limited by their shrinkage/swelling behaviour, (Silva et al., 2009). The quantity of water present in the grout may compromise the integrity and bond between the repair grout itself and the original material (Silva et al., 2009): during injection by causing the earthen crack surfaces to swell and/or shrink, resulting in cracking at the interface. Therefore, the determination of suitable water/binder ratio for lime and cementitious grouts and the water/soil ratio for earthen grouts are critical factors as far as shrinkage behaviour, bond to the crack surface and mechanical performance are concerned.

In the case of earthen materials, as is the case with other types of masonry (many authors as e.g. Vintzileou 2008 indicates that in the case of stone masonry an increase in stiffness is likely not to be desirable, especially if the grouting is applied only to some parts of a structure), the limited amount of literature on the subject (Langenbach, 2004a) suggests that stiffness discontinuity can cause global damage when the structure is subjected to seismic loading. The use of mortars or grouts resulting in rigid material, creating stiffness discontinuities in the masonry which, in case of seismic loading, would result in stress concentration and thus new cracking, is therefore not recommended in the repair or strengthening of earthen buildings, since low stress concentrations suffice to cause damage (Navarro Grau et al., 2006).

Foreseen difficulties in grouting are the achievement of adhesion, long term effectiveness, and more generally the restoration of structural integrity and continuity of a damaged structure in the long term. Thus, the correct grout selection should take into account characteristics such as chemical and mineral composition, porosity, water retention, shrinkage, durability, adhesion, mechanical strengths, etc. In addition, grout should be injectable and stable for a given time. Its consistency should allow the voids to be filled, which means that the rheological behaviour should be well understood and controlled so that the fluid may be pumped and flow correctly inside the porous environment, (Bras et al., 2009).

The main disadvantage of earthen grouts being high shrinkage (see Roselund 1990 and Silva et al. 2009), modified soil grouts have been developed, the composition of which usually includes lime, cement or organic binders. Flow and shrinkage properties can be optimised by organic additives such as dispersants. For the repair of the Las Flores Adobe, Barrow et al. (2006) developed a grout consisting of adobe soil, sand, a small amount of Portland cement, and additive (Sika GroutAid) to minimise shrinking during curing.

Grout injection as a means to repair structural cracks is a non-reversible technique, the use of which can result in durability and compatibility issues.

Figure 3.56 Alternative method for repair/strengthening of clay masonry, (by courtesy of Papayianni, 2006).

Another solution is proposed by Papayianni (2006) for repair or strengthening of adobe structures (Figure 3.56): Holes are drilled and a (clay-cement or a clay-lime-pozzolan) grout is introduced with the aim not to fill voids but to form “reinforcement bars”-like stronger zones within masonry. The up to now available results seem to be promising.

3.2.3 Bed Joint Reinforced Repointing

The reinforcement of the bed joint is commented in Section 3.1.4.

3.2.4 FRP/SRP/SRG application

The application of FRP/SRP/SRG is commented in Section 3.1.7.

The technique could be applied locally to repair cracked panel, even if the change of the property such as the increase of stiffness should be carefully taken into account.

3.2.5 Crack stitching and anchoring

In general, crack stitching consists in locally joining two independent wall sections, which can rock or overturn about their base, by means of “stitches”, i.e., joining built-in elements inserted at staggered points.

The effectiveness of the crack stitching in terms of restoring structural continuity or uniform load distribution of horizontal loads on a wall is related to the diffusion of the intervention, since force transfer is limited to the points where the stitches are introduced.

Figure 3.57 Crack stitching my metal bars, (Source: POLIMI).

Frequently proposed in the past, the use of reinforced grouting is often ineffective in stone masonry, (Figure 3.58); the failure is related to the difficulty of creating an effective anchoring in the stone masonry by the insertion of a metal tie in a drilled core and then injected.

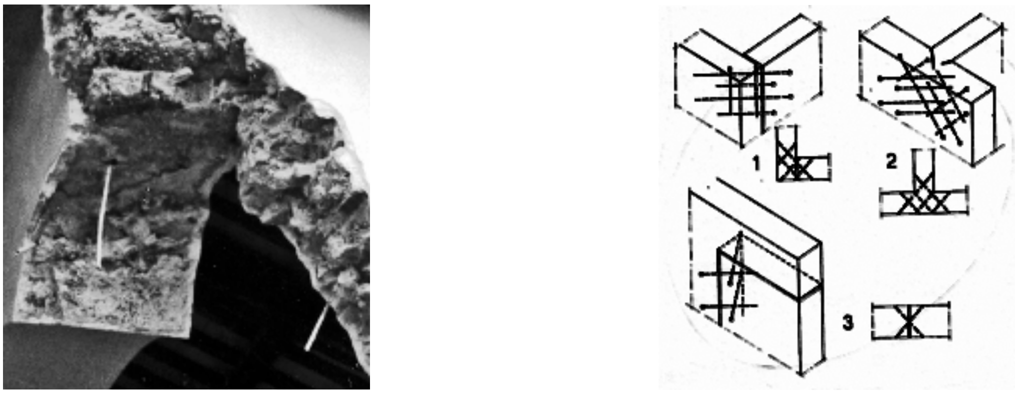


Figure 3.58 Ineffective stitching by reinforced injection.

Reinforced bed joints could act as stitching elements in regular masonry. The technique is extensively described in Section 3.1.4. The technique should be coupled with other intervention technique, as e.g. grouting in order to infill the crack and prevent the water penetration.

3.2.5.1 Crack stitching for earthen buildings

Stitching requires chases or grooves with two returns, i.e. deeper cuts, for each stitching end to be cut out of the wall. These grooves are then filled with tightly packed clay and/or timber and/or fibres: removal of historic material is therefore involved. Crack stitching does not re-establish complete structural continuity nor does it allow uniform load distribution of horizontal loads on a wall since force transfer is limited to the points where the stitches are introduced. In addition, no comprehensive review on seismic performance on site or after testing is available in the literature.

3.2.5.1.1 Stitching and anchoring

<u>Timber stitching</u>

Traditional timber stitching, (Fodde, 2007a,b), consists of filling grooves cut out from the walls by inserting timber ties or hooks in order to establish structural continuity between two wall sections. Timber, thanks to its flexibility and vapour-permeability, is described by the author as being compatible with earthen buildings. Timber ties are fixed at both ends with two timber wedges, one end being fixed first and the other extremity being fixed and tightened with a cross-tie end-securing method, and are then bedded in and packed with mud mortar and tile wedges. The technique has the advantage that it avoids typical shrinking which can occur if grouting is used. Seismic performance is not documented.

Steel nail stitching

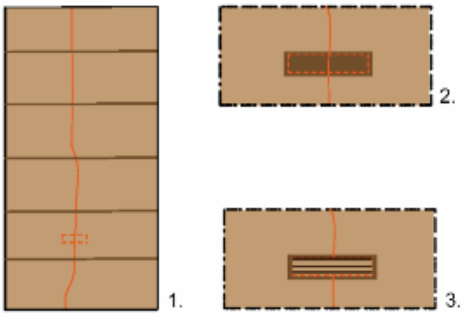
Stitching by means of steel nails, (Schröder, 2010), is carried out by using steel rods in plastic sockets supposedly fixed to the earthen material with cement slurry. The effectiveness of this technique has not been assessed.

Soft stitching

Soft stitching is carried out by cutting out chases and filling them with pre-shrunk earth tiles called clay bats, at alternate points on the interior and the exterior of earthen walls. Clay bats are used as they have been already pre-shrunk, and their application should follow the moistening of the inner part of the chase. Clay bats are then bedded with mud mortar alternated with steel, (Fodde, 2007), or fibre (Hurd, 2006) laths which allow flexibility. The disadvantages of employing steel are that metal is not a traditional building material and that it may rust, burst or expand.

This disadvantage is overcome by the technique described by Hurd (2006), which makes use of various hemp or fibre mats instead of steel.

Though Hurd (2006) claims to have observed “the use of still-functional stitches” after an earthquake of unknown magnitude, the effectiveness of soft stitching has not yet been confirmed by testing.



1. A groove as deep as half the wall thickness is cut over the crack. At its extremities, the groove is deeper, so as to form a „staple“
2. The groove is wetted
3. The groove is dry packed with alternate layers of tiles, loose adobe material and fibres.

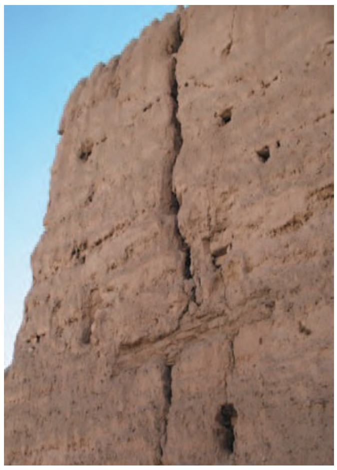


Figure 3.59 “Soft” stitching carried out by Hurd in 2004, (Photo: Jaquin, 2008).

3.2.5.2 Anchoring

Anchoring as a means to repair cracks is documented for adobe and rammed earth structures (Schröder, 2010; Jaquin, 2008). Anchors used are lightly or non- post-tensioned through-wall anchors fixed with end steel-plates or ties bolted internally to timber roof trusses and fixed externally also by means of steel plates. In one case cited, (Jaquin, 2008), the rate of crack growth was not decreased by the introduction of anchoring. The effectiveness of anchoring as crack repair for earthen buildings has not yet been thoroughly investigated. In order to minimise localised stress damage, wide hidden anchor plates separated from the wall by means of cement grout were used by ZRS to reduce anchor-induced stress (see Figure 3.60) for the Al Muwaiji Fort Project. The effectiveness of this technique has not been assessed.

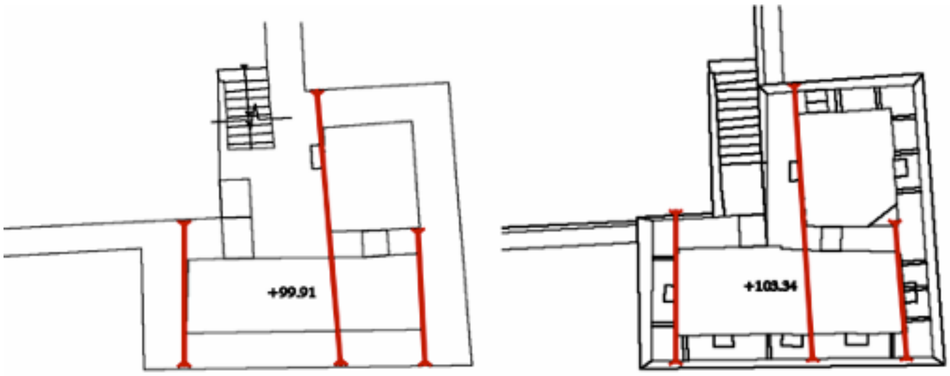


Figure 3.60 Schematic plans of anchors in South East Tower, Al Muwaiji Fort, UAE.

3.2.6 Others

3.2.6.1 Improvement of weathering performance of earthen materials

A number of studies have been published on the treatment of earthen surfaces (Coffman et al., 1990; Selwitz et al., 1990) to increase weathering performance, i.e. reduce erosion. Their use has been indicated in the literature to be particularly useful in the protection of earthen archaeological sites and ruins, where roofs, foundations, drainage systems, plasters, and renders may be compromised or absent, as they prevent granular disintegration whenever the impact of water runoff cannot be reduced by other means, such as appropriate roofing, foundations, and drainage systems nor by renewing protective plasters or renders.

The consolidants types are: inorganics such as alkaline silicates, natural organics such as plant mucilage and synthetic organics such as alkoxysilanes, acrylic resins, vinyl acetate polymers, epoxy resins and polyurethanes. Although alkoxysilanes and diisocyanates are frequently cited in the literature as being the most effective to protect earthen surfaces, (Ferron, 2007), chemical consolidants, when used as a protection to stone, have mostly failed due to overconsolidation and appearance of material incompatibility.

However, most of these studies are limited to laboratory tests on new samples and controlled environmental conditions and the real effectiveness of the previous treatment is highly debatable.

It is therefore believed that an increase in durability of earthen structure is most effectively achieved by exclusion of moisture, e.g. by sheltering, wall capping, roofing or plastering with compatible materials. A good example describing the latter approach is presented by Fodde (2008). In most case, traditional earth plasters, e.g. earth stabilized with lime and with natural fibers as straw, have the double effectiveness of wall confining and environmental protections, mechanical and hygrothermal compatibility.

Particularly counter-productive is the use of cement plasters, since cementitious materials are chemically, mechanically and physically incompatible with earth due to its low strength, high porosity and different hygrothermal behaviour. When cement plasters are applied as means to protect the walls from moisture, they can cause swelling and loss of strength, in some cases due to the higher capillary action described by Minke´s (2006) studies, based on German DIN Norm 52617, which suggest that the addition of cement to earth can increase its water absorption w (kg/m^2) by 100-400%), resulting in a decrease in mechanical strength. Figure 3.61 shows damaged earthen materials under cement stucco introduced as repair, the addition of which not only increases capillary action, but also prevents the release of moisture from the earthen material when they are vapour tight, resulting in strength loss. Excessive moisture in walls is one of the conditions described by Tolles (2006) as being the most critical within the scope of the seismic retrofitting of adobe heritage buildings, particularly in the case when they are covered by hard, impervious cement-type renderings.

(a)

(b)

Figure 3.61 (a) Cement stucco on an adobe masonry, San Andres, El Salvador (left, source: BAM) (b) Damage underneath cement stucco in cob house, Germany (right, source: ZRS).

3.3 STABILITY IMPROVEMENT

3.3.1 Enlargement

Masonry enlargement has been used traditionally to increase the load bearing capacity of walls, by applying it on the intrados, to increase the capacity of vaults or to increase wall stability. In this last cases, frequently the thickness of the new leaf can change in the height, working as a sort of buttress, (Figure 3.62a,b)

Enlargement refers to the addition of new material (such as an additional leaf of new masonry) to an already existing member, with adequate connection or interlocking, in order to increase its section and hence mechanical capacity.

Mechanical compatibility requires (1) the use of a not too different material, regarding stiffness and strength, with respect to the original one, and (2) a good connection between the original member and the added material.

Frequently in modern intervention, the added leaves consisted in regular brick masonry, sometimes also reinforced, or cement units. In Italy, most of these interventions were related to the increasing of the overall shear capacity of the buildings according to the code requests.

The evident differences in stiffness, the lack of effective connections, the irregular layout (sometimes the intervention was limited to some panel or only to a level) or the unexpected behaviour of the new assemblage produced several documented damages after the recent earthquakes in Italy. The addition of a third leaf to the existing walls usually failed by separation of the new leaf especially when bad connections were realised between the new and the other leaves.

In turn, the removability of the enlargement depends on the possibility of dismantling the added parts without causing significant harm to the original material. Such removal will be normally possible (with limited damage on the original part) if stone or brick masonry, with limited interlocking, has been used for the enlargement.

(a) (b) (c)

Figure 3.62 Example of wall enlargements, (Source: POLIMI).

3.3.2 Buttresses

It consists of the addition of massive elements to laterally prop a structure. Buttresses resist lateral forces and deformations essentially thanks to their self weight. Buttresses contribute to prevent from failure mechanisms related with lateral deformation. Buttresses originally built as part of the entire original construction may be very efficient, as they are normally built in a homogeneous way with the rest of the structure (with the same type of masonry, well interconnected to the rest, while also sharing a unique foundation). Conversely, buttresses built as a later strengthening device, after the construction of the original parts, may show limited efficiency due to lack of satisfactory interlocking or differential settlements separating them from the rest. Furthermore, when the buttresses are built as later additions, the structure will need to deform to significant extent in order to mobilise the new buttress.

These problems may be overcome by building flying arches in between the structure and the new buttresses, as done in the case of some Byzantine churches and monasteries.

External buttresses are quite often used in stone masonry monuments. We have several examples in Greece, especially in Byzantine churches, where (due to the complex system of cupolae and domes and to the resulting horizontal thrust) there is a tendency of out of plane displacement of walls. Some examples are given in the pictures that follow.

Katholikon of Dafni Monastery, (Figure 3.63). Two stone buttresses were added (in the beginning of the 20th century) in the north façade of the church. Buttresses would be needed also in the south façade. However, due to the fact that this would harm the access to the church, buttresses were constructed only in the north façade.

Katholikon of Ossios Lucas Monastery-Added buttresses (south façade) (Figure 3.64). In the north façade, buttresses were not needed, because there, there is another church serving the same purpose.

Flying arches produce stabilizing forces, without the structure having to laterally deform, as soon as they are built. Moreover, these forces will persist in spite of possible differential soil settlements between the buttress and structure. External buttresses combined with flying arches may provide a very efficient way to increase the seismic capacity of the structure (Figure 3.65). Due to its external character, this solution will normally be fully non-invasive and removable.

When a frontage wall or a great wall presents a cant, the response of the Community to this problem is almost always the same one: the use of a buttress, a mass of masonry built against a wall to strengthen it. This system is a consolidating element to the existing structure and it is generally added to an older masonry building. Sometimes, the buttress is constructed at the same time as the building construction, a voluntary and premeditated act to reinforce this construction, generally at the corners of the structure. In the areas subjected to the seismic risk, the buttress frequently accompanies the stone frame and becomes an essential element to achieve the building stability.

Sometimes, the buttresses were used both as utilitarian and decorative forms. They can be also used as staircase ensuring the access to the dwelling, built to play the role of the confortement, a judicious way to associate reinforcement technique with comfort.

Flying arches between buildings were frequently built up in order to prevent the overturning of walls, Figure 3.66. In time, some of them were also transformed in passages or lodges.

Figure 3.63 Katholikon of Dafni Monastery, (Source: NTUA).

Figure 3.64 Katholikon of Ossios Lucas Monastery-Added buttresses (south façade), (Source: NTUA).

(a)

(b)

Figure 3.65 Buttress at S. Chiara Church in Assisi, (Source: POLIMI).

(a) (b)

Figure 3.66 Flying arches between walls in (a) Israel (Sorce: IAA) and (b) Morocco, (Source: ENA).

Mud brick buttresses, in the case of earthen building, are used not only to reinforce structures in seismic areas, but also to prevent the collapse of unstable walls, for instance when reduction of the wall cross section at the base of the wall can result in instability. Figure 3.68 shows some geometrical specifications according to Guidelines for Wall Construction with Buttresses and Pilasters (IAEE, 1986).

Fodde (2007) provides detailed description of strengthening of heritage earthen walls by means of earthen (adobe) buttressing, though not for seismic purposes.

The addition of buttresses to adobe and cob structures has in some of the literature been held responsible for the collapse of the parts of the structure adjacent to the newly built buttresses (Langenbach, 2004), though it is not always clear in the literature what materials have been used to construct the newly built buttresses. Photographic evidence, (Figure 3.67), of the collapsed Caravanseray in Bam (Langenbach, 2004) shows that the domed rooms behind the added buttresses collapsed, while the domes opposite, where there are no buttresses, still remain, possibly due to a difference in stiffness between the newly introduced buttresses and the historical material. When a compatible material is used for the purposes of strengthening, the introduction of buttresses is believed to aid structural behaviour under seismic loading.

Figure 3.67 Collapse of domes adjacent to newly built buttresses, (Source: Randolph Langenbach).

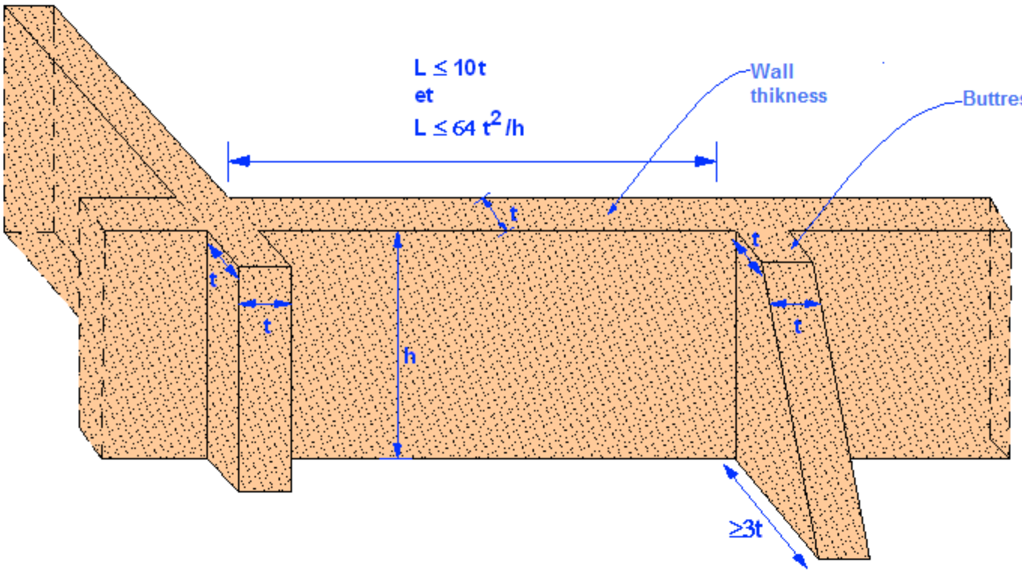


Figure 3.68 Some geometrical specifications according to Guidelines for Wall Construction with Buttresses and Pilasters, (IAEE, 1986).

4 IMPROVEMENT OF SUB-ASSEMBLAGE CONNECTIONS

Masonry building represents a box-type structural system composed of vertical structural elements - walls - and horizontal structural elements - floors and roofs. Vertical loads are transferred from the floors, acting as horizontal flexural members, to the bearing walls, and from the bearing walls, acting as vertical compression members, to the foundation system.

The observations of masonry buildings when subjected to earthquakes have shown that the behaviour is strongly dependent on how the walls are interconnected and anchored to floors and roofs. It is generally recognized that a satisfactory seismic behaviour is attained only if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited.

It has to be mentioned that masonry walls exhibit enhanced vulnerability to out-of-plane bending, (low bending moment capacity mobilized under limited imposed inflexion). This pronounced vulnerability is negatively affected by all the above mentioned conditions that limit the box action of buildings, as well as by the poor quality of construction type of masonry and the poor quality of building materials.

Old masonry structures seldom satisfy the conditions of ensuring box action: floors and roof are rarely well connected with the walls, floors and roof do not behave as diaphragms of limited deformability in their plane, the connections between walls is quite often defective, whereas large openings and openings located close to the corners of buildings lead to further weakening of the box action. In old structure the unfavourable effect of insufficient anchorage between walls and between walls and floors was often observed. Irregular structural layout in-plane, large openings and lack of bearing walls in both directions often caused severe damage or even collapse. A good quality of the connections between floors and walls, between roof and walls and between perpendicular walls is also crucial to reach a good global seismic behaviour of the building. Good quality connections will drive the collapse of the construction to a configuration that requires a stronger seismic action.

This structural behaviour is strictly connected to the presence and to the quality of the constraints between walls and floor/roof structures. The presence of ties is another relevant factor in the effectiveness of the constraints. The influence of the connection between orthogonal walls in the overturning mechanism has been studied by various authors such as (de Felice and Giannini, 2001) and (D'Ayala and Speranza, 2003).

As detailed in Figure 4.1, unstrained wall could globally overturn, while the presence of the floor constraint due to friction changes the kinematic mechanism triggered by a higher energy. Similar effects are due to the constraints of roofs structures and ties.

Within the project, the Deliverable 6.1 deeper explores this topic.

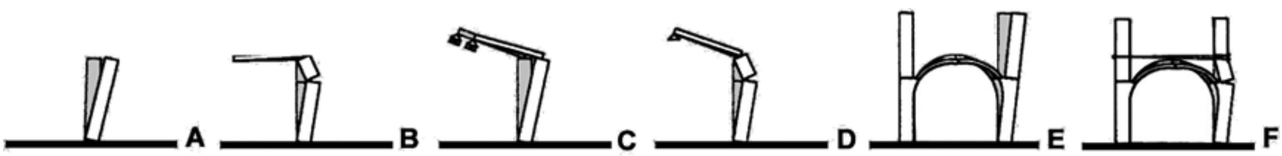


Figure 4.1 Effects of the constraints imposed by floors and vaults, (Carocci, 2004). A unconstrained wall, B floor constraint; C wall not restrained by roof structures; D roof restraining; E vault thrusting; F tied vault.

4.1 LACK OF CONNECTION BETWEEN WALLS

Besides global extensive interventions, like insertion of tie beams or tie-rods, local improvement of the connection can be carried out by the techniques described on Section 3.2.5 to stitch cracks.

4.1.1 Tie beams

The introduction of sub-assemblage connections, as an effective alternative to the introduction of independent structural frames to reinforce structures, is acknowledged by the most updates seismic codes, thanks to researches and post-earthquake documented surveys.

Traditional steel/iron ties or wooden ties are often present in traditional architecture all over the world.

Webster (2006) provides examples of earthen historical buildings in California where stability-based measures were used between 1992 and 2005. Such retrofitting is believed to have an effect only once ground shaking > 0.3 g (Webster, 2006), since it only comes into play once old cracks have reopened, new cracks have developed, and enough displacement has occurred to engage the stabilising measures, which, according to Webster (2006), reduce a building´s response in two ways: (1) by increasing the structural damping due to friction hysteresis across the cracks; and (2) by lowering the response frequency due to wall rocking.

Sub-assemblage connections used for the stability-based retrofitting of historic structures are the introduction of any of the following (which can be used in conjunction with one another): bond beam or equivalent (straps, cables, also intra-mural), top-of-wall pins (steel or fiberglass) or anchorage, diaphragm (partial or full).

4.1.1.1 R.C. tie beam

R.C. tie beams were used extensively as means to reinforce masonry/earthen structures by providing continuity and connection at the top of the walls and support the roof/floor structure. They were suggested by some code and often proposed in handbook both as wall/wall and wall/floor connection elements.

R.C. tie beams are usually inserted where timber floors and roofs are substituted by mixed concrete and clay block structures. In these cases a R.C. tie beam is built at every floor. The tie is positioned along the four sides of the structure to connect load bearing and shear walls in order to prevent out-of-plane failures.

In an existing building while the roof concrete tie can be positioned on the whole thickness of the top wall, the ties at each floor can only be inserted in part of the section after partial demolition of it, (Figure 4.2). Introduction of tie-beams in the masonry thickness at intermediate storeys should be definitely be avoided, due to their damaging effects on perimeter walls, often causing also uneven load redistribution among masonry leaves and/or pounding effects on the external masonry leaves in case of seismic excitation, (Figure 4.8). In this case it is very difficult to realise a stiff connection to the existing wall. In general this connection is very difficult when the wall is made of a multiple leaf irregular stone masonry. Recent seismic events in Italy widely documented the damages. The most frequently observed damages were the following: (i) partial eccentric loading of the walls, (Figure 4.3), (ii) lack or poor connection of the tie beam to the walls, (Figure 4.4).

The seismic events, then, showed that these elements cannot transmit the horizontal actions to the walls and neither can connect the two masonry leaves, of which one remains free and can rotate freely and overturn, (Figure 4.5).

The collapse mechanism of the masonry is not in-plane shear as expected after the floor substitution, but a partial overturning mechanism of the external leaf of the wall which starts for lower values of the expected collapse coefficient, (Figure 4.7).

Some details visible in the upper part of the Figure 4.3 and Figure 4.5 suggest that the intervention contributed to reduce the already weak connection between the leaves to the very critical section where the walls are connected to the floor. In fact, in those connections the confining actions of the floors are applied and most probably less uniformly distributed. Even the contribution of the new internal wall, perpendicular to the collapsed facade was completely missing in the collapse, possibly due to the restoration interventions.

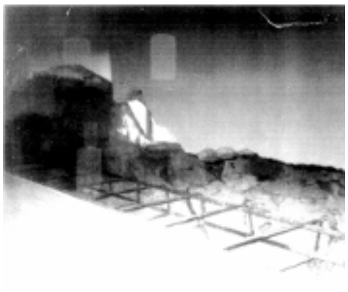
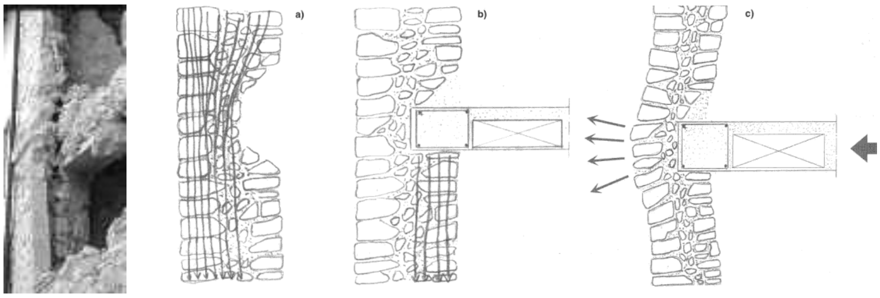


Figure 4.2 Building phase of a R.C. tie beam, (Source: POLIMI).



(a) (b)

Figure 4.3 Effect of eccentric loading due to R.C. tie beam positioning and failure of the tie beam insertion at each floor under vertical and horizontal actions (a) (Binda et al., 2003a); (b) (Gurrieri, 1999) and (Avorio, 2002).

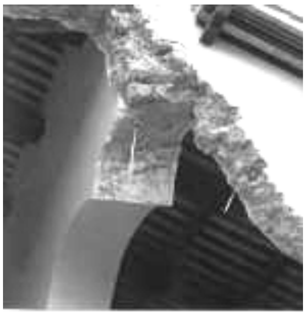
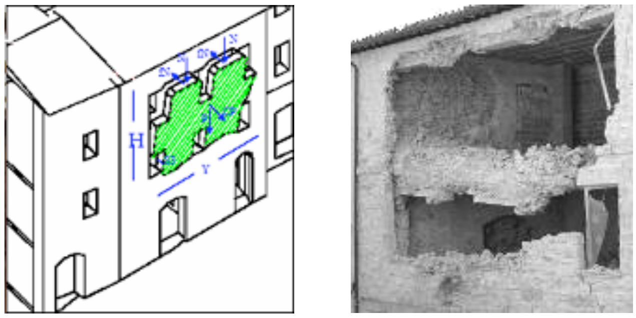


Figure 4.4 Difficult connection between the roof tie beam and the wall, (Binda et al., 2003a).



(a) (b)

Figure 4.5 Out-of-plane collapse of a wall with R.C. tie beams (a) (Gurrieri, 1999) and (Avorio, 2002); (b) (Binda et al., 2003a).

In this technique it is important to realise an effective connection between the tie beam and the masonry. In the case of the tie beam at the roof level, which can rest on the whole section of the wall, the connection is difficult because it should be realised by vertical metal connectors inserted in the wall from the top, (Figure 4.4). Once again, this connection is seldom possible in a multiple leaf stone masonry but also on brick masonry. The presence of the R.C. tie beams is frequently revealed by continuous cracks at the roof or floor level, (Figure 4.6, Figure 4.7 and Figure 4.8).

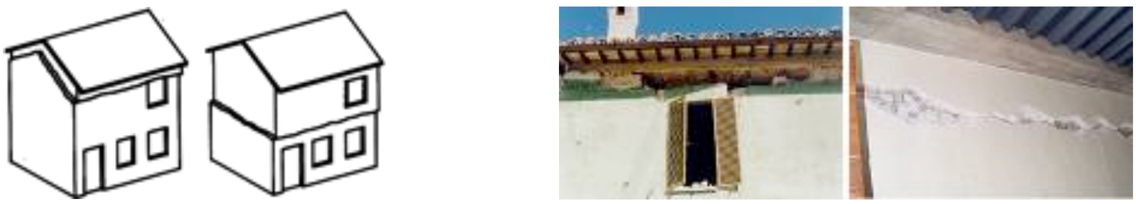


Figure 4.6 Difficult connection between the tie beams and the walls.

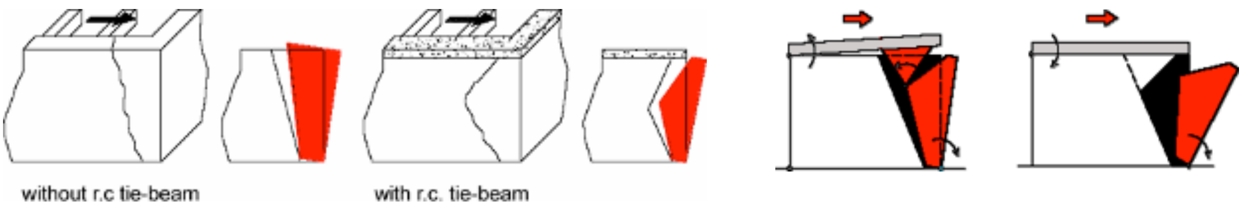


Figure 4.7 The presence of R.C. tie beams does not prevent the wall overturning but changes the shape of the damaged area, (Borri, 2004a).

Figure 4.8 A R.C. tie beam is constructed over the opening. Disintegration of the pier and horizontal sliding is observed, (Source: NTUA).

For earthen structures examples cited by Tolles et al. (1996) where concrete bond beams were used (in conjunction with other techniques, as specified below), are:

Reyes Adobe	Concrete bond beam, plywood diaphragm
Sepulveda Adobe	Concrete bond beam and corner columns, grade beams, steel rods cross ties at bond beam level
Roberto Adobe	Concrete bond beam, plywood diaphragm, earth anchors
Mission San Juan Batista	Concrete bond beam, plywood diaphragm, earth anchors

Prior to 2007, Reinforced concrete bond beams used to be recommended for the upgrading of existing adobe buildings in California, (California State Historical Building Code; see appendix C). The 2007 edition of the Code, which has specific recommendations for adobe buildings, instead, generally recommends the use of tie beams, but does not specify the use of reinforced concrete. (The code also specifies allowed strength values for adobe buildings and suggested design levels, and limits the wall height-to-thickness ratio to 6. Anchorage forces are not explicitly addressed in the SHBC.).

Similarly to masonry buildings, the disadvantages of this technique are its invasive and destructive nature, which usually requires removal of the roof system, and results in stiffness discontinuities: due to their design often being based on elastic design criteria, very high beam stiffness results, which once cracks have developed in the earthen structures may exceed the stiffness of the walls by two or three orders of magnitude, (Tolles et al., 1996). This difference and the lack of a positive connection between the bond beam and the adobe walls has caused adobe walls to pull out from underneath bond beams during earthquakes, (i.e. Northridge Earthquake 1994, see Tolles et al., 1996 and 2006).

Despite Tolles (2006) stating that if the roof system is already intended to be removed or replaced the installation of a bond beam if and only if it is properly anchored may well be an appropriate retrofit option, testing results found in the literature do not support this retrofitting technique. As in case of masonry structures the building of an effective anchoring is seldom possible.

Results from testing conducted at PUCP, (Torrealva et al., 2009a), included models with a reinforced concrete ring beam anchored to the walls with shear connectors at all corners, and showed that for strong motions, equivalent to intensity MMI = X, partial collapse and global instability are not avoided, since the reinforced mortar bands are much stiffer than the adobe walls and therefore tend to absorb most of the seismic forces until the elastic resistance is reached and a fragile rupture occurs.

In his evaluation of the retrofitting for the Soltani Mosque after Silakhor Plan Earthquake in Iran, Vosoughifar (2006) attributes the seismic destruction of the Mosque to the execution of a rigid concrete beam over the doorway, which aggravated the impact of the earthquake on the structure.

4.1.1.2 Timber tie beam

In the last two decades, many retrofitting design solutions have been directed toward methods that are less invasive than the use of reinforced concrete strength-based design. A detailed discussion in the context of recent seismic retrofitting designs for a number of historic masonry and adobe structures is given in (Thiel, 1991), (Gurrieri, 1999), (Doglioni, 1999) and (Doglioni and Mazzotti, 2007).

Within the scope of the Getty Seismic Adobe Project (see Tolles et al., 1996), retrofit measures were investigated that would have minimal impact on the historic fabric of the building, which included the use of timber bond beams as an alternative to reinforced concrete bond beams.

The beams were introduced to have a negligible effect on the dynamic behavior of the walls in the elastic domain, and a large impact on the out-of-plane behavior of the walls after cracking.

Results showed that the strength and the stiffness of the wood bond beam used in the tests were sufficient to transfer loads to the in-plane walls. An alternative to the standard timber bond beam is shown in Figure 4.9(b).

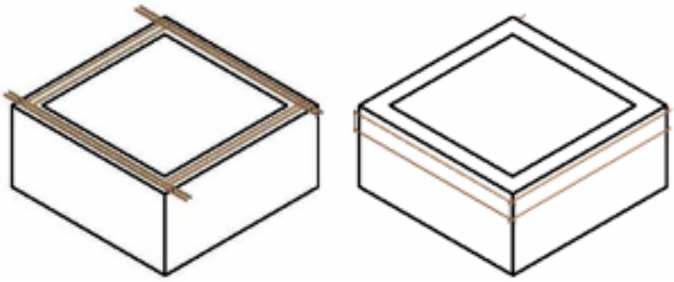


Figure 4.9 Timber Bond Beam (left) traditional and (right) a Nepali vernacular technique which includes vertical wooden ties at each corner, as described and reintroduced by Sikka and Chaudry (2006).

Untreated timber components, where termite infestation is known to occur in the region, should not be considered as effective unless they are treated.

4.1.1.3 Other

Reinforced masonry (Figure 4.10) or steel tackles could be effective alternatives (Figure 4.11); this last one is enough light and easy to anchor to the roof wooden structure. Figure 4.11 shows an example of steel confinement by tackles.

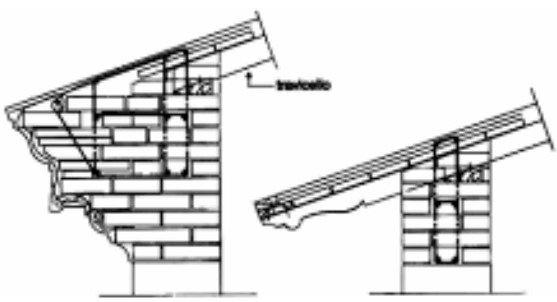


Figure 4.10 Reinforced masonry bond beam, (Gurrieri, 1999).

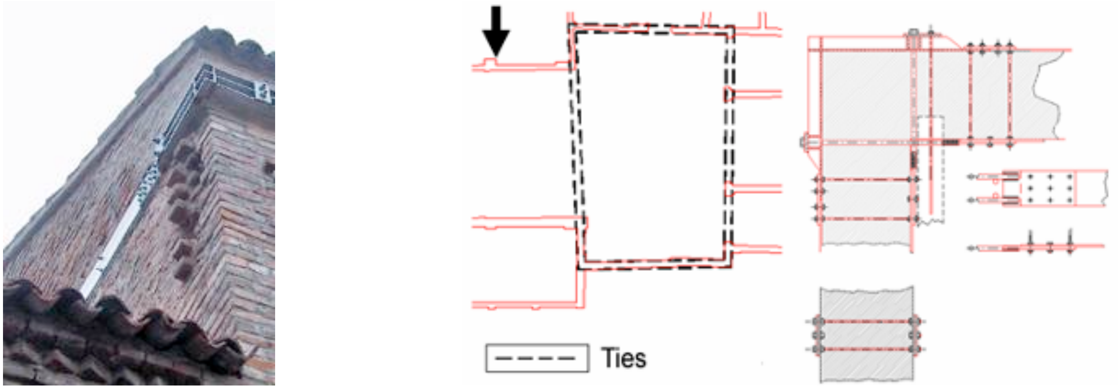


Figure 4.11 Steel tackling, (Source: UNIPD).

4.1.2 Tie-rods

Metal ties are widely used in traditional buildings as wall-to-wall and wall-to-floor connections aiming to improve the integrity of the structure. Knowing the concept and details of the original use is often a pre-requisite for making a right decision on the repair method. Low cost and easy installation, easy maintenance and repair are the great advantages of the techniques.

The use of steel or timber tie rods to connect wall to wall and wall to floor was known since the Byzantine times (St Demetrius in Thessaloniki, Aghia Sophia in Istanbul, 4th and 5th cent.). The use continued in the Gothic architecture not only for seismic protection but also to collect the thrust of arches and vaults. In the 15th century, systems of rods were applied in seismic areas to regain verticality for out of plane walls. Tie rods in seismic areas were suggested systematically through the 17th, 18th and 19th century, (Milizia, 1785) and (Rondelet, 1832). This technique was also applied in Calabria in 1878 after an earthquake. In Umbria it was prescribed by the Recommendations published by the Municipality of Norcia and in Sicily at the beginning of the 20th century (Archivio Storico, 1861), (Figure 4.12).

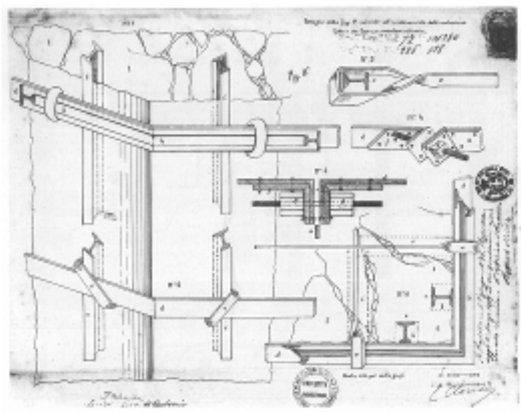


Figure 4.12 Connection wall to wall by tie rods (1909).

The ties are usually positioned at the roof/floor level. The layout should be as regular as possible (Figure 4.13).

The dimension of the anchoring system is related to the tie tension and masonry quality.

Ancient ties had a low stress according to the time technology by heating. Their anchoring system is generally only a bar or a decorative element. Recent ties, instead could be high tensioned, but require greater anchoring devices, (Figure 4.14 and Figure 4.15).

Excessive high tension ties could be too stiff during the earthquake, damaging the masonry at the anchoring level.

In general metal ties are very effective restrain systems. Failure is generally related to anomalous added masses as shown in Figure 4.16.

Besides ties the connection could be obtained by external confinement systems. In general the application is possible metal elements (iron, steel, SMA…) or more recently by FRP stripes. Emergency confinement could be done by polyester bands. Corners require special care to the stress distribution, (Figure 4.17).

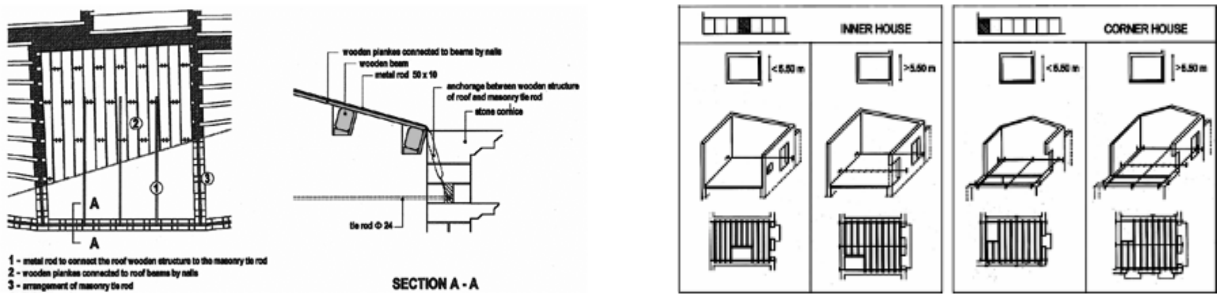
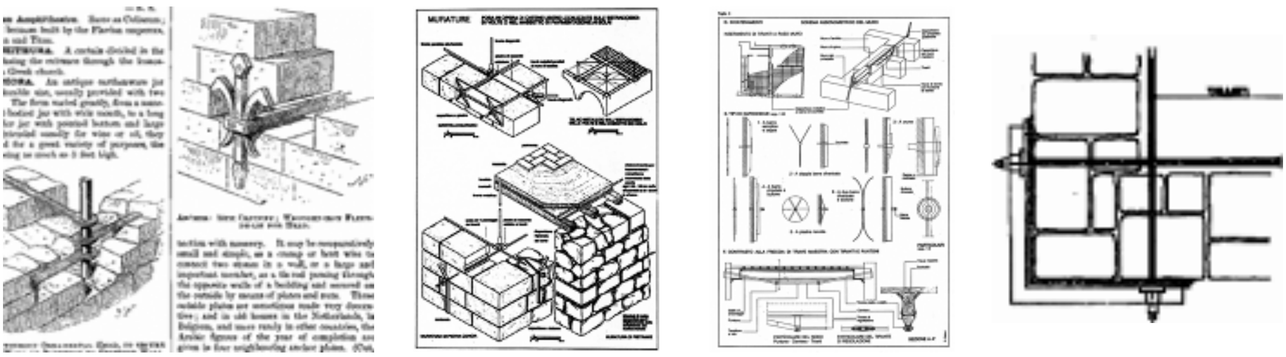


Figure 4.13 Layout of the tie rods in a Palermo's historical building (Giuffrè, 1999), (Carocci, 2001).

Figure 4.14 Example of tie anchors.



(a) (b) (c) (d)

Figure 4.15 Tie anchoring.

Figure 4.16 The tie was not able to restraint the wall overturning; the stiff and heavy R.C. tie beam pushed over the church façade, (Source: POLIMI).

Figure 4.17 External confinement by metal tie, (Source: UNIPD).

Examples of the traditional metal ties used at historical masonry buildings in Bosnia and Herzegovina dating from the end of the 19th and the beginning of 20th century are presented in Figure 4.18, Figure 4.19 and Figure 4.20. Metal ties, normally invisible are found exposed due partial or significant damage of the buildings as consequence of the recent war destruction and/or the long-term influence of weather conditions.

Figure 4.18 Residential Building Bulevar 25 in Mostar, Bosnia and Herzegovina. Position of visible metal ties marked A to D. Front facade wall demolished above the ground level. Photo Interprojekt, December 2009.

Figure 4.19 Residential Building Bulevar 25 in Mostar, Bosnia and Herzegovina. Detail A. Detail of the anchors on the facade. Photo Interprojekt, February 2009.

Figure 4.20 Residential Building Bulevar 25 in Mostar, Bosnia and Herzegovina.

4.1.2.1 Through-wall anchors or ties for earthen buildings

Retrofit measures to improve wall connection with minimal impact on the historic fabric of earthen buildings include the use of steel or polypropylene

- upper- and lower-wall horizontal cables/ties;
- exterior horizontal cables/ties;
- through-wall anchors.

The following buildings were retrofitted in California as part of a retrofitting scheme for seismic strengthening:

Casa Primera	Partial plywood diaphragm, steel straps and anchors.
Pico Pico Adobe	Steel Straps and fiberglass earth anchors.
Parra Adobe	Independent timber frame with steel tie rods.

Evidence of failure at anchorage points due to stress failure of earthen material in recently installed retrofit systems were observed after the 1994 Northridge earthquake at the Pio Pico Mansion (See Tolles et al. 1996) due to crack propagation starting at stresses anchorage points: due to the weakness of earth as a structural material, low stress concentrations at these locations, which can hardly be avoided, generally lead to cracks and crushing of material, and can pull into retrofitted walls thus being ineffective in adequately restraining out-of-plane motion. Wall anchors (or tie rods) retrofitted with the intention of holding walls together with perpendicular walls or diaphragms are therefore considered by Tolles (1996) to be difficult to apply to adobe successfully.

4.1.3 Hysteretic dissipation anchor

In the last decades, drawing on the experience derived from earthquakes such as Northbridge, California, USA, 1994, and Kobe, Japan, 1995, structural engineers have progressively abandoned capacity methods and moved on to performance-based techniques, (Priestley, 2000), which focus on the enhancement of the ductility and the use of additional dissipative elements rather than rely on stiffness and strength for the purpose of improving the seismic behaviour of structures.

Nevertheless, current codes still acquiesce to the use of traditional stiffness-based systems for the retrofit of historic buildings (EN 1998 Eurocode 8; Italian Ministry of Cultural Heritage and Activities, 2006), whilst the application of techniques involving ductility and energy dissipation, despite being allowed in principle, is limited since innovative systems rarely meet some of the requirements – reversibility, low impact – required for interventions on historic structures. Few high-profile case studies indeed appear in literature, (Indirli and Castellano, 2008; Mandara and Mazzolani, 1994).

Considering that a substantial number of European and Mediterranean historic centres are located in earthquake prone areas and that generally the whole urban fabric should indeed be considered as a heritage asset, the state of the art raises concerns. The drawbacks of strength-based systems were brought once more to the fore by last earthquakes in Italy (Umbria-Marche 1997, L'Aquila earthquake, Italy, April 2009): inadequate compatibility of materials in terms of mechanical properties – stiffness, weight, and connections - can indeed be highly detrimental (Binda et al., 2003a), (D'Ayala and Paganoni, 2010).

However, while in the case of elements such as concrete ring beams the elevate mass and stiffness, often aggravated by inadequate connections, concur to cause tragic collapses and make this system unsuitable for historic low shear capacity masonry walls, cross-ties, which have been and are still commonly applied in rehabilitation practice all over Europe, (Tomaževič, 1999), meet the requirement of restoring the box-like behaviour, allowing for the redistribution of horizontal load in sets of perpendicular walls without substantially increase the mass. Thus, traditional cross ties are able to provide a connection at the joints of perpendicular sets of walls, where poor quality, previous damage, or general wear and tear facilitate crack onset and eventually out-of-plane failure. Nonetheless anchors can also cause pull-out damage at the head of the anchorage and increase in-plane diagonal cracking because of the different deformability of steel and masonry. This might become a major problem when damage limitation should be pursued also avoiding cracking in precious plasters, frescoes, or other culturally valuable finishes.

Drawing on the above observations, Paganoni and D'Ayala (2009) developed, within the framework of a Knowledge Transfer Partnership (KTP) between the University of Bath and Cintec International Ltd, a dissipative device specifically designed to address the lack of passive systems for the seismic protection of heritage buildings.

The device is conceived as add-on for stainless steel ties. Thanks to either the hysteretic properties of a stainless steel element, shaped to optimise its post-elastic behaviour, or a friction mechanism set to be triggered for a certain level of pulling force, the device allows small relative displacements, dissipating energy and hence reducing the impact of seismic force on the walls, and controlling damage. The system has the considerable advantage of being compatible with existing as well as post-installed anchors.

Initial experimental work (Paganoni and D'Ayala, 2009) included tests in pseudo-static and dynamic regime of the dissipating device on its own. A target displacement of ± 10 mm, comparable to the allowable inter-storey drift required by current guidelines, was achieved (Figure 4.21) as well as the control over the localisation of deformations within the anchor assembly, (Figure 4.22).

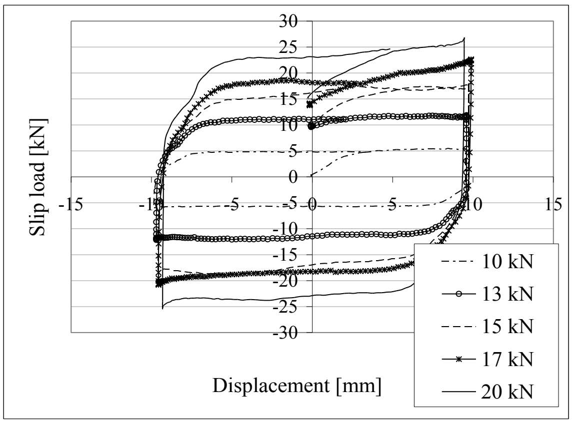


Figure 4.21 Load deflection curve of frictional anchor working at different level of F⊥ (Paganoni, 2009).

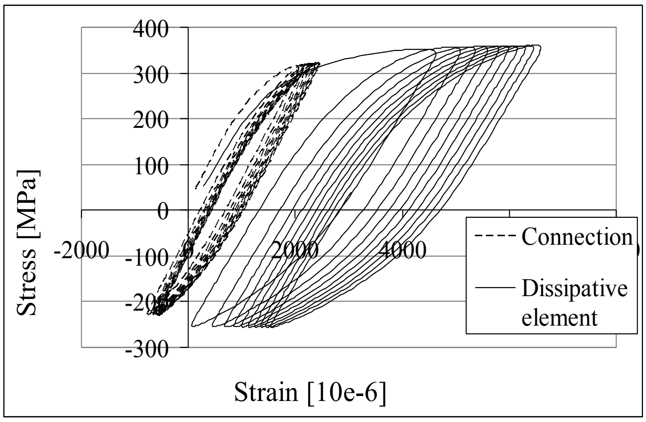


Figure 4.22 Distribution of deformation in yielding anchor undergoing dynamic sinusoidal loading (Paganoni, 2009).

A campaign of pull-outs tests aimed at characterise the behaviour of strength-only and dissipative anchors in low shear capacity walls (Paganoni and D'Ayala, 2010) followed.

Experimental results confirmed that both the yielding and frictional element are able to provide the anchorage with ductility, (Figure 4.23 and Figure 4.24), and prevent damage to occur in either the grouted sleeve or the masonry, while traditional anchors display a high stiffness and fail at the interface between grout and parent material or in the masonry units. In the case of friction devices, it was observed that, due to tolerances of the pieces composing the assembly, a locking phenomenon occurred for the higher levels of perpendicular pressures; consequently, load-deflection curves feature additional stiffness in respect to the flat branch typical of a friction mechanism.

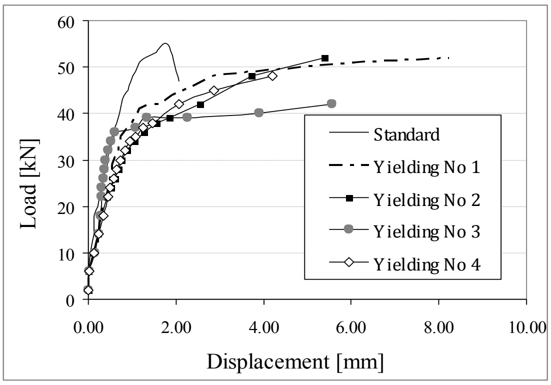


Figure 4.23 Comparison between traditional anchor (continuous line) and frictional anchors working at different levels of F⊥ (Paganoni, 2010).

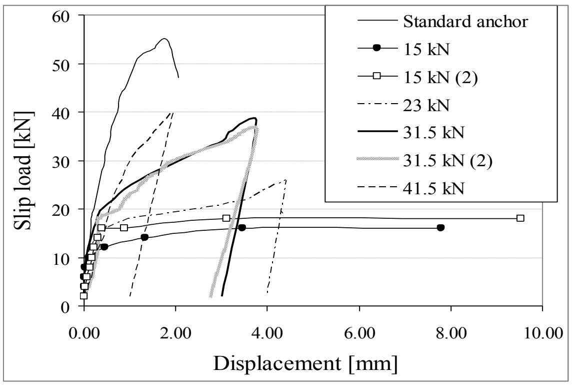


Figure 4.24 Comparison between traditional anchor (continuous line) and frictional anchors working at different levels of F⊥ (Paganoni, 2010).

The authors (Paganoni, 2010) also developed a set of Finite Element models that were calibrated with experimental data through monotonic linear and non-linear analysis, showing good agreement and confirming the values derived for mechanical parameters such as the coefficient of friction. For high levels of perpendicular forces, the FE model of the friction device conserved a behaviour consistent with the Coulomb friction, this representing the target performance of the prototypes. The influence of the parent material was investigated by modelling both isolated devices and devices embedded in a parent material; while a difference can be observed for hysteretic devices, the change in stiffness of the overall load-deflection curve of friction devices is hardly detectable. Models confirmed that deformations in the parent material and in the joint with the grouted part of the anchor are negligible, as observed from experimental results.

4.2 LACK OF CONNECTIONS BETWEEN WALLS AND FLOORS/ROOF

Traditional intervention concerning the connection between walls and floors/roofs concerned the tying of the main beams by metals anchoring elements. In this way the wooden beams were slightly tensioned. Wooden tie beam was another effective way to connect as commented in Section 4.1.1.

Modern interventions were mainly addressed to replacing of timber structures with R.C. or mix clay/R.C. floors/roofs. The connection were realised through the R.C. tie beams, with the effects already commented in Section 4.1.1.

The anchoring of the floor beams to the walls is aimed at prevent the beam slipping and hammering. Furthermore, the effectiveness of the connection is important to distribute shear action and restrain the walls preventing possible overturning.

The wooden structures are anchored by metal elements nailed on the beams and restrained to the wall, (Figure 4.25). Several layouts and configurations are available, some of them used in the past as well, (Figure 4.25).

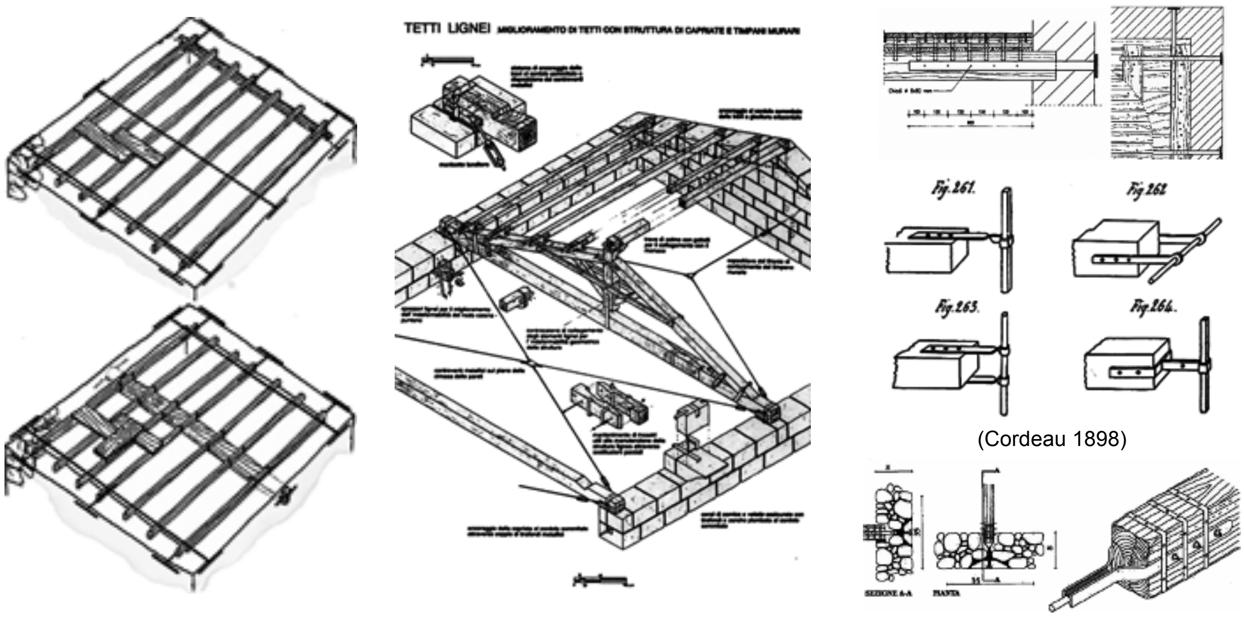
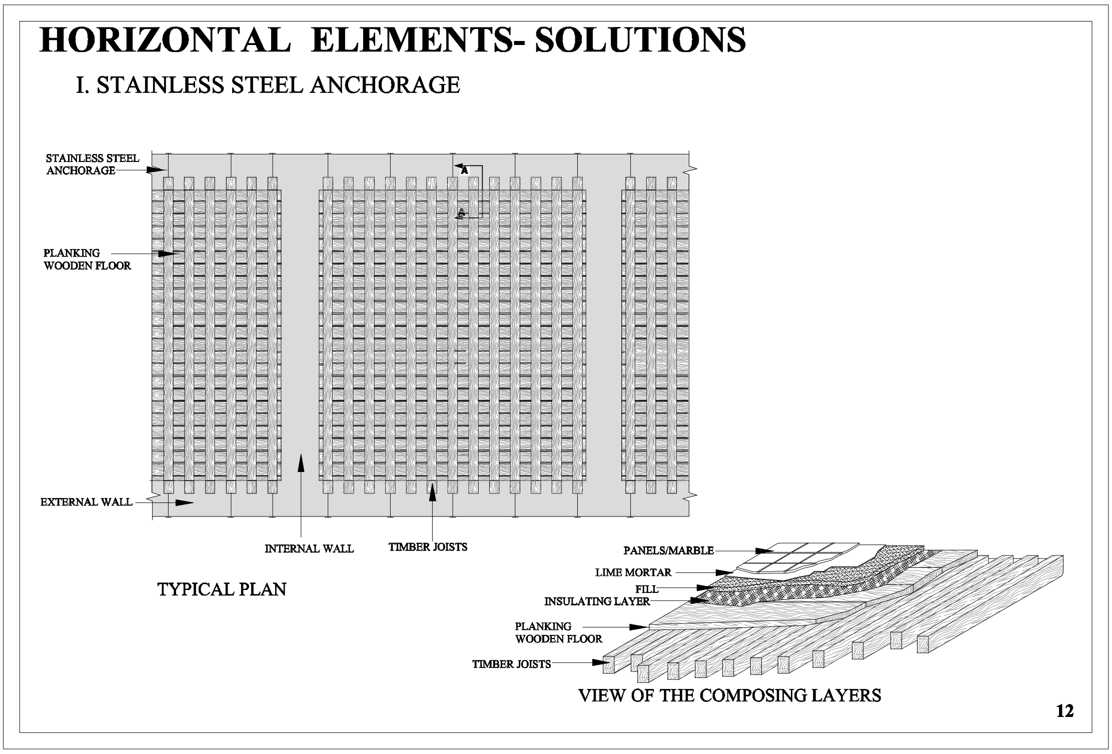


Figure 4.25 Connection between timber floor and roof and walls.

Figure 4.26 shows a floor strengthening method developed by IAA. The strengthening of the wooden floors is not different from similar systems used in other countries, with the exception of the floor's joints anchors. This method of anchors was used in conservation projects in Acre since 1995.



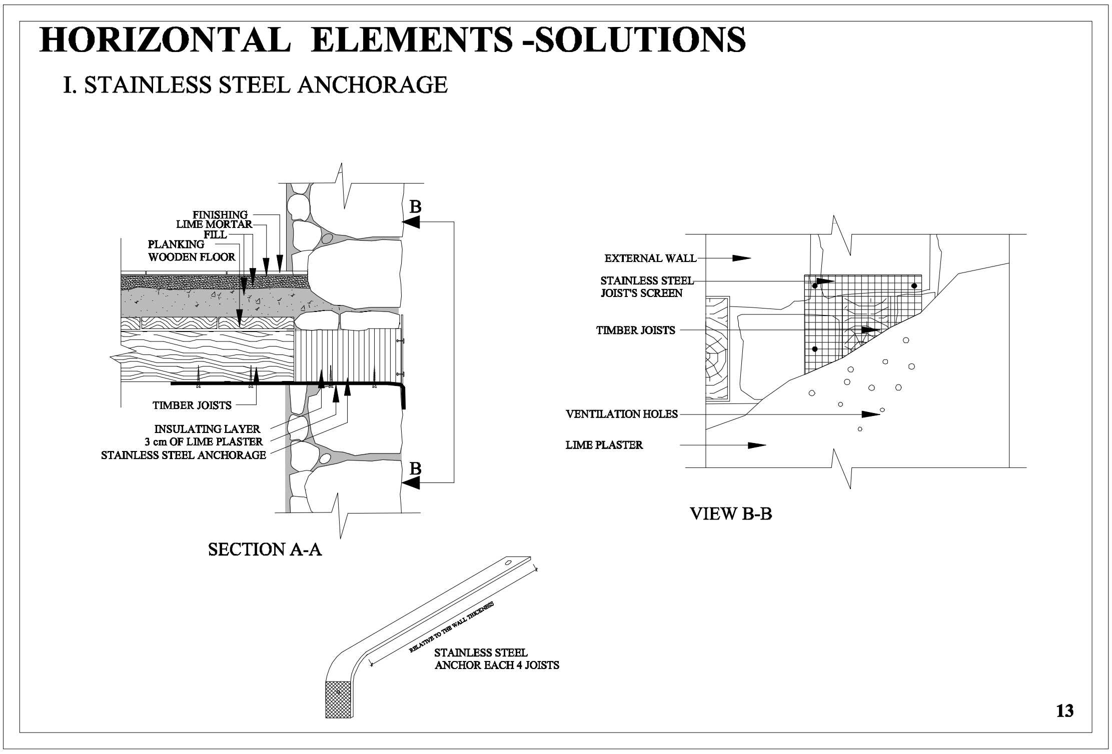


Figure 4.26 Strengthening of wooden floors and joints anchors developed by IAA. This method of anchors was used in conservation projects in Acre since 1995, (Source: IAA).

4.2.1.1 Earthen materials

Due to the weakness of earthen materials, connection between walls and floors should be carried out by means which avoid tensile stresses in the earthen material. In order to minimise localised stress damage, Tolles (2006) discourages the use of connections such as those shown in Figure 4.27 (left) and advocates the use of connections such as those shown in Figure 4.23 (right).

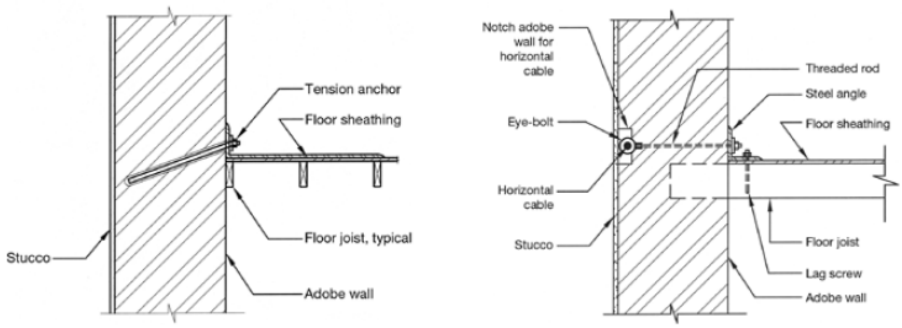


Figure 4.27 Wall-floor connection for earthen walls. To the left, technique discouraged by Tolles (2006). To the right, wall-floor connection proposal by Tolles (2006) to avoid localized stress damage to earthen walls.

5 OPTIMIZATION OF VAULT PERFORMANCE

It is possible to classify the vaults collapse mechanisms based on its causes. Among the possible sources of damage it is possible to distinguish:

- The relative displacement of the supports;
    a. Displacement of the supports on the orthogonal direction to the generatrix line of the vaults;
    b. Differential settlement of the piers;
    c. Longitudinal sliding.
- The variation of the load to which the vaults and piers are subjected;
- The decay of masonry.

The first mode mechanisms involve the rotation of the supporting elements (walls, columns, pillars, etc…) (Figure 5.8), while the second mode mechanisms involve in plane loading, then a relative sliding and deformation due to shear stresses, (Figure 5.8).

5.1 DIRECT INTERVENTIONS (APPLIED TO VAULTS)

5.1.1 Local Dismantling and Reconstruction ("scuci-cuci")

The technique is commented in Section 3.2.1 related to masonry repair.

5.1.2 Grout injection of the cracks

The technique is commented in Section 3.1.1 related to masonry repair.

5.1.3 Structural Repointing (Deep repointing)

The technique is commented in Section 3.1.3 related to masonry repair.

5.1.4 Bed Joint Reinforced Repointing

The technique is commented in Section 3.1.4 related to masonry repair.

5.1.5 FRP/SRP/SRG application

The main aim of this technique is resisting to tensile stress, alters the brittle collapse mechanism of the unreinforced structure, preventing the formation of some hinges, because the edge of two adjacent faces are constrained not to split by the reinforcement's presence on that surface.

Therefore, the resistance of the interface between masonry and FRP is crucial to ensure good cooperation between the two materials. So in most cases, less stiff FRP material appeared to be more effective both in terms of ultimate strength and stiffness increase of the panels. That is due to the particular design criterion (weaker material has a larger adhesion area), and also to the fact that stiffer material is more vulnerable to de-bonding, especially when the number of plies increase, (Valluzzi et al., 2002).

The technique is commented in Section 3.1.7 related to masonry repair, as well.

The application procedure requires: shore up the vault at the intrados before removal of filling at the extrados, then clean the surface where the FRP will be applied. The next step applies first layer of polymer resin, the fibres and then the second layer. The application must be made with a temperature greater than 5°C and low humidity, because low temperature retard the polymerization and humidity decrease the characteristic of polymer resin.

The intervention is minimally invasive and easily removable, however, can be largely ineffective. For example in Figure 5.1 the hinge on keystone can still be as it coincides with the fibre.

Figure 5.1 FRP at extrados, (Gurrieri, 1999).

In Figure 5.2 the reinforcement arch or “frenelli”, which increases the thickness of the vaulted structure, thus increasing the effective thickness and stability of the element, but do not change the mode of collapse.

Figure 5.2 Reinforcement arch at intrados, (Gurrieri, 1999).

In Figure 5.3, strengthening combines the previous two: FRP are included between vaults and reinforcement arch; this technique changes the mode of collapse, but the springing sliding, is no real way of collapsing.

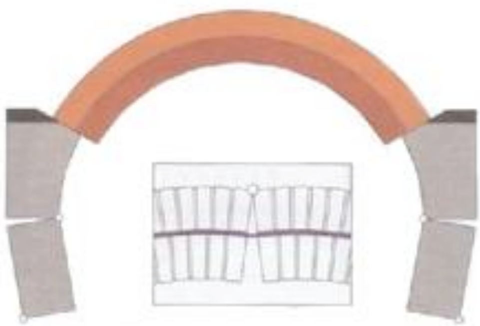


Figure 5.3 Reinforcement arch and FRP, (Gurrieri, 1999).

The construction of reinforcement arch at the extrados is certainly advisable, it is designed to increase the thickness of the vault, thus allowing a greater variation of thrusts line and increase the stability. In addition the reinforcement arch confines the fibres to the extrados of the vault.

If the FRP is positioned at the intrados, the adherence is only ensured by adherence between the resin layer and the surface of the vault. In other words, between FRP and vault are created tensile stresses perpendicular to the surface. This does not happen with the extrados application: where the tension on the FRP causes a compressive stress on the vault.

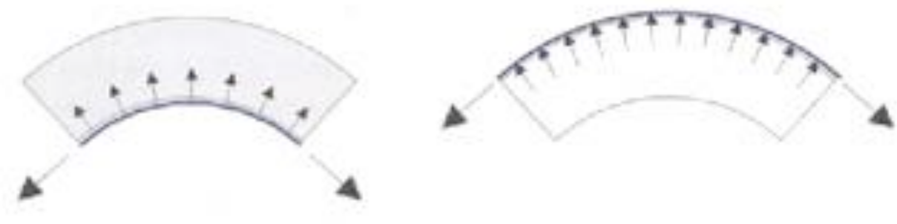


Figure 5.4 Behaviour of FRP at intrados or extrados, (Gurrieri, 1999).

5.1.5.1 Geo-textiles for earthen vaults

Adobe vaults fully reinforced with polymer mesh on both sides have been tested on a shaking table at PUPC (Lima, see Torrealva et al., 2009a,b). Results showed that unlike the unreinforced adobe vaults, the retrofitted vaults performed well under simulated seismic loading, despite extensive cracking.

5.1.6 Use of extrados R.C. jacketing

One of the main problems of vaults may be due to the load on it.

The aim of this technique is to remove the extrados heavy filling and replacing them with other lighter structures for decrease the loads on the vaults.

The advantages of this technique are: reduction of compressive stresses in the vault, introduces the tensile strength with the new structure, (Ferrari et al., 1999). Furthermore, the technique can deeply change the thermo hygrometric conditions, and start moisture problems which could decay masonry but also the vault reinforcement.

Comments relate to the wall jacketing are referred in Section 3.1.2 related to masonry repair.

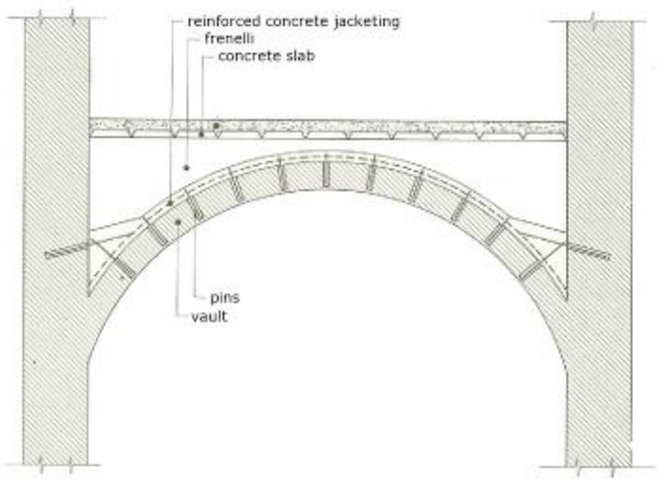


Figure 5.5 Extrados R.C. jacketing, (Caleca et al., 1983).

The phases of construction is: hold up the vault with steel elements, remove the filling, clear the extrados of the vault, drilling holes for inserting the steel bars, tends a layer of electrically welded and cast concrete. The concrete slab increases in thickness near the abutments, so that the line of thrust results inside the core of section, (Guerrieri, 1999).

In case of earthquake, the vibration of R.C. slab can be producing the collapse of part of masonry and reinforced concrete generated significant deterioration due to incompatibility (mechanical, physical, chemical, rheological and thermal) with the original materials.

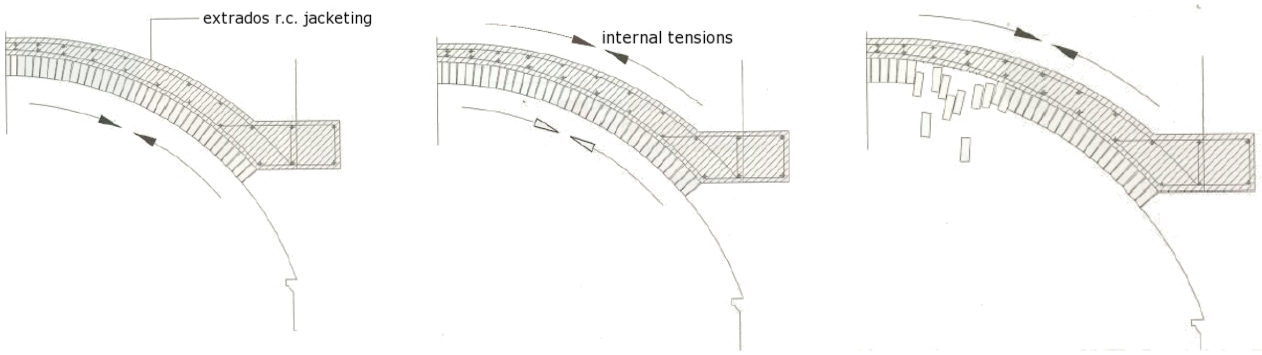


Figure 5.6 Limits and restrictions of R.C. jacketing, (Gurrieri, 1999).

5.1.7 Reducing the loads from extrados infilling

In some situation the reducing of the thrust could be necessary. In this case, the infilling could be slightly and slowly reduces symmetrically. As know, in common situation infilling has an equilibrating action.

An alternative strengthening system should be built up, particularly during the working phases.

5.1.8 Extrados stiffening elements, mainly in barrel vaults (“frenelli”)

Stiffening masonry elements were frequently added in the past. They could be built with the vault or added at the extrados in a second time, (Figure 5.7).

Figure 5.7 Layout of stiffening elements in barrel vaults.

5.2 INDIRECT INTERVENTIONS (SUPPORTING MASONRY STABILITY)

5.2.1 Insertion of the tie-rods and confinement

The technique is commented in Section 4.1.2 related to the wall connections.

Figure 5.8 shows an example of tie rods application to strengthen a crossing vault.

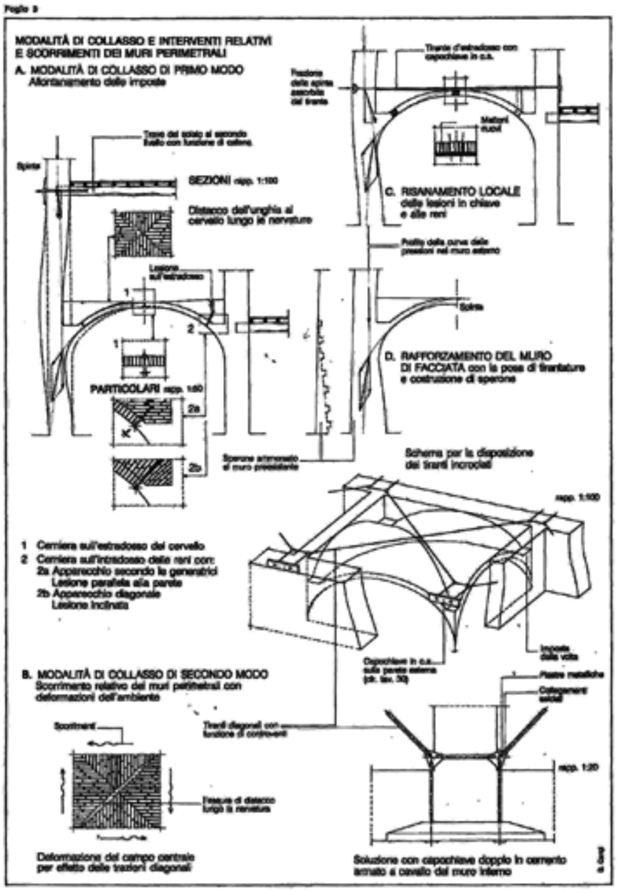


Figure 5.8 I and II mechanisms in vaults and strengthening intervention, from (Giovanetti, 1998).

Confinement is particularly suitable for domes due to its geometry. Frequently used in the past during the building of the dome itself at the drum level. It were mainly in metal but it is possible to find wooden confinement elements in the masonry or large stone blocks connected between them, as in case of the S. Maria del Fiore in Florence by Brunelleschi. Sinan, an ottoman architect of XVIth cent., systematically inserted metal tie in his domes. After the famous Poleni's research in XVIII, the dome of St. Peter in Rome was confined.

The intervention is very effective as documented by the behaviour in past time by several structures.

5.2.2 Buttresses

When the masonry walls strengthening, in foundation and along the height, are not able to counterbalance the thrust of arches or vaults, can increase their effectiveness by creating buttresses, placed at the centres of thrust, (Figure 5.9 and Figure 5.10). Their characteristics and weight lead the result of the inclined and vertical forces inside the central core of sections.

The buttresses can be constructed in various materials such as stone, brick, concrete and reinforced concrete, stone masonry.

This solution is often not feasible for aesthetic reasons, and economic dimensions, yet it is used for provisional structures, (Caleca et al., 1983). However in traditional architecture is often present, particularly at the end of sequences of arches, (Figure 5.10).

The main problems connected this technique affect the interface between abutment and buttress: for example, the new element can have differential settlement of the foundation or the transmission of the thrusts is of poor quality, (Guerrieri, 1999). Other problems could concern the breaking of the tie, which can change dangerously the equilibrium conditions, (Figure 5.11).

Further comments are in Section 3.3.2.

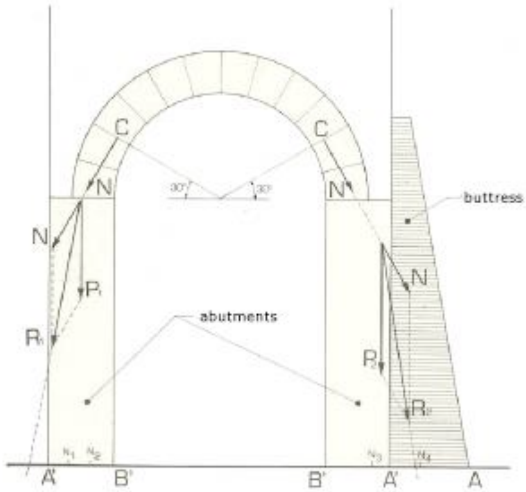


Figure 5.9 Buttress and forced, (Caleca et al., 1983).

Figure 5.10 Buttressed vaults, (Source: POLIMI).

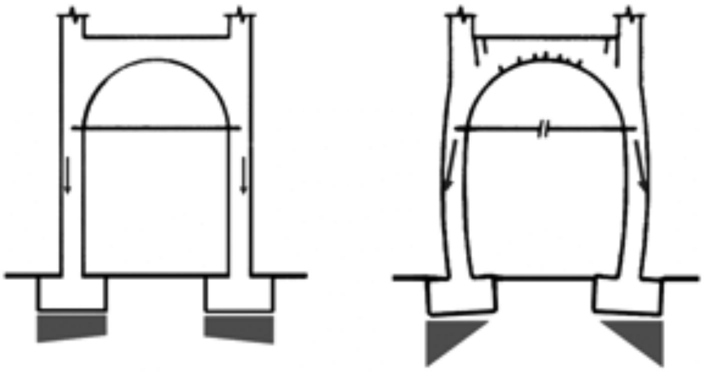


Figure 5.11 Effects of the tie breaking, (Source: POLIMI).

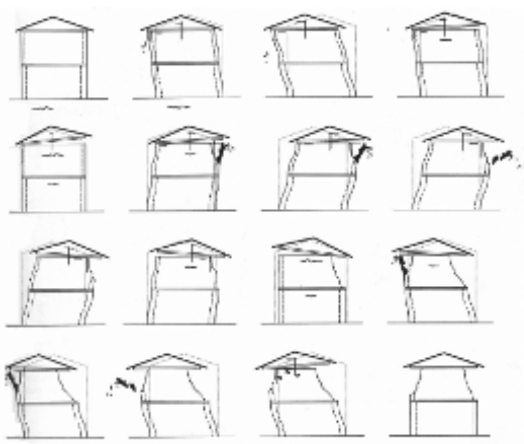
6 SEISMIC IMPROVEMENT OF WOODEN FLOORS

The post-earthquake surveys after the recent earthquake have shown poor performance of heavily reinforced buildings that often made them more vulnerable. It appears that a large number of the damaged buildings had been modified over the years to meet the needs of their users also following in Italy the seismic code, but that such measures had rarely improved their resistance against seismic events. As a matter of fact, these functional modifications, (remodelling, raising, enlargement, opening or closing of new windows or doors) frequently added vulnerability. A common form of upgrade has been the replacement of the old wooden floors with new floors of heavy reinforced concrete beams supporting hollow clay tiles. This was often done without upgrading the strength of the masonry bearing walls. Sometimes bearing walls were even removed to open up spaces, resulting in beams located where shear walls had once existed. The post-earthquake survey found that alterations and remodelling was rarely done with any heed of regulations or seismic design criteria, (Mufanò, 1990).

The compatibility or incompatibility of materials and systems is a good indicator of seismic vulnerability. Recent earthquakes were particularly ruthless on masonry buildings that had been retrofitted relatively recently with reinforced concrete floors (usually with recast beams with large hollow clay tile elements spanning between the beams). One might have thought the stronger and stiffer diaphragms would have improved performance, but these floors were not properly tied through the masonry walls, and thus did not contribute to holding them together. A significant problem with this kind of construction is that the absence of wall ties is not easy to verify, while the presence of the new floors may give one the false sense of security that the structure has been improved.

In the highly seismic Umbria-Marche region, seismic retrofitting may have been the principal intention in replacing roofs. The added masses and the stiffness of the new roof cracked the supporting walls and frequently causing their total collapse; or damaged the external leaves of multiple leaves masonry walls, (Figure 6.1).

The increase in the weight and the stiffness of the roof, whose retrofit usually involves a reinforced slab insertion over the wooden ledger board, can lead to an increase in the horizontal seismic forces that induce collapse of the masonry walls; moreover, the stiffness of the top most tie-beam can obstruct the natural vibration mode of the masonry, thus inducing local high stresses in the masonry. In most cases the stiffness of the concrete roof or floors was too high compared to the one of the existing wall and the roof can hammer the wall and cause a partial collapse.



(a)

(b)

Figure 6.1 Structural failure due to the insertion of a stiff R.C. roof (a) (Avorio, 2002), (b) (Binda et al., 2006).

In most cases, the wooden floors present in historical constructions, are considered deformable until some elements are added to increase the in-plane stiffness (ex: reinforced concrete slabs), and until appropriate connections with the perimetral walls are established. Indeed, in the historical constructions, the floor can hardly be considered as an in-plane bracing element, but rather as a dead load that unloads horizontally, during a seismic event, on the masonry, (Barbisan and Laner, 1997).

The in-plane stiffening of the floors, even limited, allows to distribute diversely the seismic actions among the vertical elements, and implies, generally, an increased of resistance, which improves the structure robustness, (Barbisan and Laner, 1997).

The need to stiffen the floors in their plane, ensuring the connection to the walls, is almost exclusive of wooden floors, because in the floor solutions in mixed steel-tile or in reinforced concrete-tile, the in-plane stiffness is usually always assured by the concrete layer, while the connection to the perimetral walls is ensured by a R.C. beam, (Bazzana, 1999).

However, it should be considered that the transformation of flexible floor into in-plane rigid floors leads to a redistribution of the horizontal loads over the walls that can have positive or negative effects, depending on the geometry of the structure and on the mechanical characteristics of the materials that compose it. Indeed, the stiffening of wooden floors, unless accompanied by adequate monitoring and improvement of the mechanical characteristics of the supporting walls that result more loaded (in-plane) with this redistribution, can even be harmful. In this regard, it is known how important is for the global behaviour of a structure to consider the stiffness of each material that composes it, because the combination of materials with very different stiffness causes abnormal and fragile behaviours if the structure is subjected to dynamic actions, as in the case of seismic actions.

Next are summarized the main stiffening techniques used, that range from the execution of a reinforced concrete layer, to the use of steel ties, beams or plates, and reinforcement techniques according to the principles of sustainability, with the use of wood elements. The types of intervention are presented according to the type of material used.

6.1 IMPROVEMENT INTERVENTIONS THROUGH THE USE OF WOOD (IPM)

The idea of reinforcing the wooden floors with wood has been widely applied in the past. Already, Rondelet in one of its treaties, anticipated today's floors reinforced with plywood panels, suggesting a coherent system of intervention that required a greater stiffness, (Barbisan and Laner, 1997). The in-plane stiffening of these structural elements may be performed by working in the extrados on the planking.

6.1.1 Orthogonal or diagonal planking

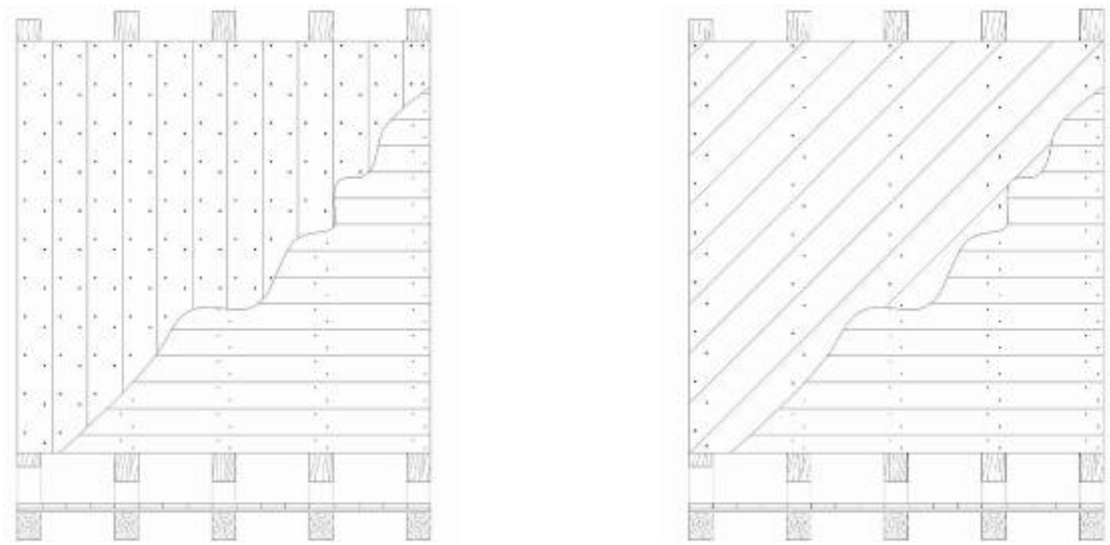
This stiffening technique consists on attaching to the floor’s extrados a second deck rotated of 45° or 90° in relation to the existing one and connected to it using screws or nails, Figure 6.2. Generally the boards should have a thickness not inferior than 4.0cm. It’s also desirable the use of interlocking boards and especially well-seasoned, for a proper interface with the existing beams that can be seasoned due to years of installation or, on the contrary, impregnated with moisture, (Barbisan, 1995).

This intervention improves the in-plane behaviour of the floors without introducing excessive weight. However, it should be paid particular attention to the connections with the side walls.

The application of reinforcement at 45° offers considerable resistance to compression, the strength of the connections such as bearing resistance, doesn’t vary between the tension and compression state of the boards; remains the instability and equilibrium problem of the boards, which can be solved using screws that force the floor boards to remain in-plane. Where the table is tight, the behaviour is similar to that of the metallic diagonals with a more widespread distribution of connections.

The connection of the reinforcement planking with nails in two layers, allows a doubling of the number of connections and guarantees, through the use of boards with tongue and groove joints, an increase of stiffness due to static friction between the edges of the joints.

Figure 4.26 shows a strengthening of wooden floors and joints anchors developed by IAA by planking. This method, particularly interesting for the anchoring system, was used in conservation projects in Acre since 1995.



(a) (b)

Figure 6.2 In-plane stiffening example using (a) orthogonal boards and (b) diagonal boards.

6.1.2 Only wood technique - Timber flange connected by dowels to main beams, (Modena et al., 1997c)

This technique uses only wooden components for the reinforcement of the floor, Figure 6.3. The reinforcing technique consists of placing a new board connected by means of dowels above each beam of the existing frame, (Modena et al., 1997c, 1998). To facilitate execution and to ensure compatibility with existing structures and materials, in the original conception of the intervention the dowels were made of hardwood and driven into the preferably 'dry'-boards, that is, without any material to improve adhesion inside the hole, (Figure 6.3a). Moreover, with proper connection of main beams to load-bearing walls, and possible additional boards in the upper surface, the system can efficiently work even in seismic zones, (Figure 6.3b), (Modena et al., 2004). A T-beam compound section, whose web (original beam) and flange (new board) are made of wood, with deformable connections between the flange and web, is thus obtained. Web and flange are separated by the existing planking, having a thickness of 2.0 to 2.5cm. Figure 6.3c shows the typical unequal spacing of the dowels along a beam in the prevalent shear zone, and Figure 6.3d shows the different mechanical behaviour of compound sections with deformable connections. The operation is executed from the underside of the floor, thus preserving possible valuable floor surfaces and enables the instalment of systems (electrical, water, etc…) without impairing the walls at their base.

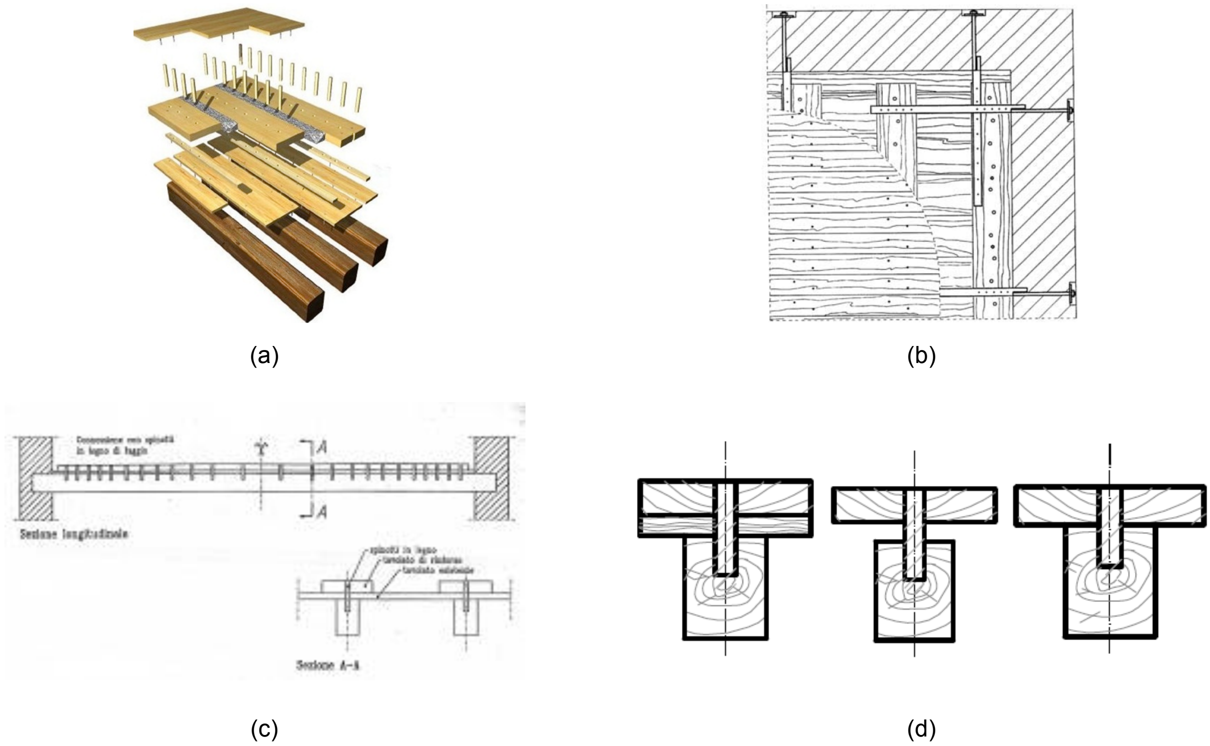


Figure 6.3 (a) Wood-wood floor: prospective view of the composing elements, (www.unibs.it/centrosismo/schede). (b) Detail of the connection to the walls. (c) Qualitative design example, the variable step (spacing) of the connectors, (Modena et al., 1997c). (d) combined flexural and shear (left), bending (centre), pure shear (right).

The stiffening boards must be sawn before being placed on the planking and their undersides levelled to allow perfect adhesion with the existing surface. The boards can be fixed to the existing planking by means of screws to facilitate subsequent intervention phases and, when the floor beams are not propped, to provide adhesion between the straight boards and the permanently deflected floor beams. The screws can be placed every four to six dowels, in pre-drilled countersunk holes. The boards must be placed with the pith upward, in order to maintain the screws in tension even after wood shrinkage. Subsequently, the position where the dowels were inserted can be drawn into the board by means of a template.

Dowel positions are staggered by about half a diameter from the longitudinal beam axis, in order to increase the resistance of the system to longitudinal splitting. The pre-bored holes are about 1 mm smaller than the dowel diameter. They are cleaned with compressed air before forcing the dowels by hammering, (Modena et al., 2004).

6.2 IMPROVEMENT INTERVENTIONS THROUGH THE USE OF A REINFORCED CONCRETE COOPERATING SLAB

It is a technique well spread for the in-plane reinforcement of wooden floors, consists on placing over the existing deck a concrete slab, usually reinforced with a metallic net, and anchored to the existent floor by pins or connectors fix on the top edge of the beams, which cross the planking and are embedded in the concrete slab and connected to the metallic net, Figure 6.4.

The overlap of the reinforced concrete layer changes the wooden floor structural concept, transforming it from a beam solution into a floor with a partial T section. This implies a displacement of the neutral axis to the point in which the timber beam works essentially under a tensile state, (Barbisan and Laner, 1997). For this reason, the actual behaviour of these connections is intermediate between the two limit cases of rigid connection and no connection, (Capretti and Ceccotti, 1992).

The structural particularity of this type of section is the connection between wood and concrete, designed to transmit shear forces parallel to the structure, between the beams and slab.

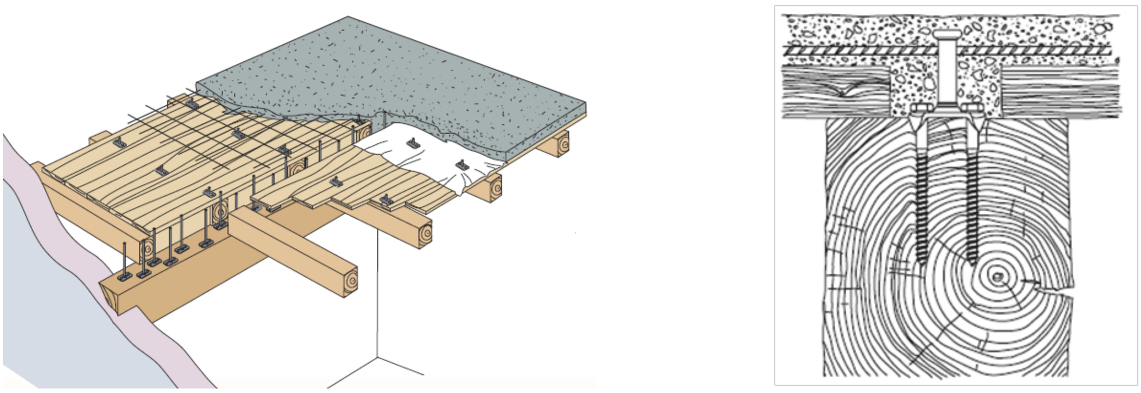
There's no advantage in overlaying the slab without linking it to the pre-existent structure, because the two structures would work independently, the friction caused by the roughness of the existing boards wouldn't sufficient to prevent sliding in the horizontal plane, (Tampone, 1996). However, any type of connection between these two structures can't be considered completely rigid and a slip between the two parts of the floor should be accepted.

Moreover, the unsuccessful anchorage of the concrete slab to the wooden beams and to the walls, in case of a high intensity earthquake, may even be counterproductive, due to the different behaviour of the two layers and due to the increase of the floor mass.

The connections are often made with rebar with improved adherence, cemented with epoxy mortar in holes drilled in the beams, the fixation length of the bars must be of at least 10ø or 2/3 of the wood height, the space between connection (step) should be between 8ø ≤ i ≤ 30ø.

In recent years many solutions for the connection between wood and concrete have been tested. The following solutions can be distinguished: punctiform union (connectors with steel bars for reinforced concrete, nails, pins in concrete, dry metallic pins) and continuous (metallic blade, truss beam, perforated corrugated steel sheet), (Laner, 1995).

Finally it should be noted that this reinforcement technique, developed in the last 20 years, allows to significantly increase the floor's in-plane bending stiffness, however, it leads to a weight gain for the deck, resulting in increase of the seismic actions, and creating an extra thickness, sometimes incompatible with the local dimensions, such as the inter-storey height, (Gattesco et al., 2007).



(a) (b)

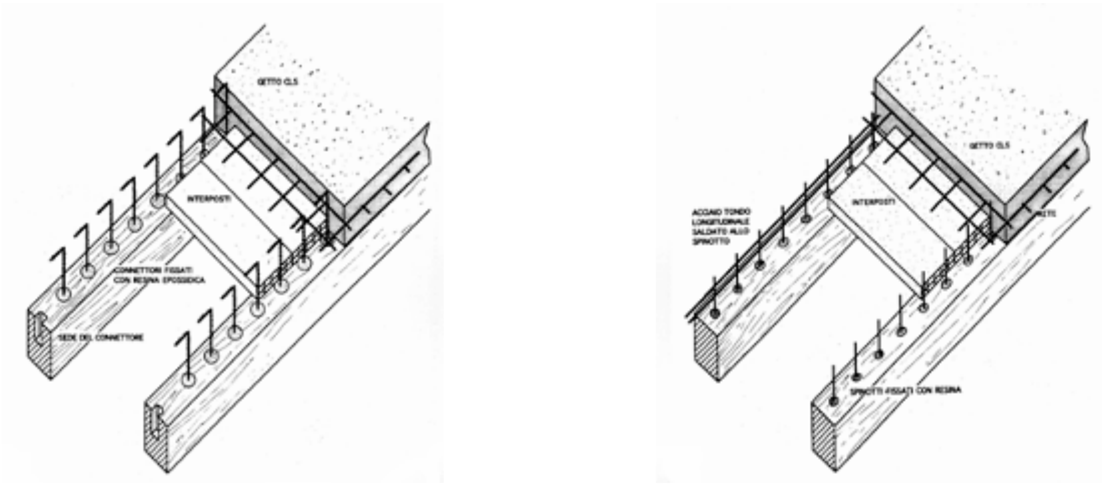
Figure 6.4 (a) Example of the reinforcement of a wooden floor with a cooperating reinforced-concrete slab, (www.tecnaria.com). (b) Basic connectors Tecnaria (www.tecnaria.com).

6.2.1 The Turrini - Piazza (1983) method

It's one of the most used methods, following an experimental campaign the authors coded connector's method, to overcome the uncertainties resulting from the simple nailing, allowing its correct design. These steel connectors with improved adherence are inserted in suitable holes, with a higher step in the area near the supports, and attached to the wood with epoxy mortar, (Turrini-Piazza, 1983), Figure 6.5a.

6.2.2 The Alessi, Lamborghini, Raffagli (1989) system

It is a variant of the Turrini-Piazza method, considers the use of connectors to which are welded two longitudinal rods to increase anchorage and the pins union, in the free stripe of the planking, (Barbisan and Laner, 1997), Figure 6.5b.

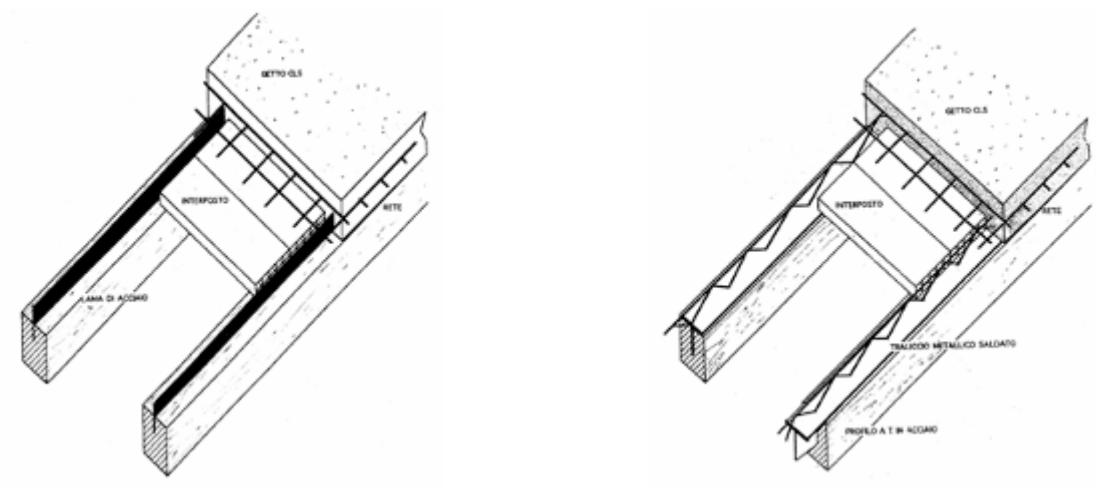


(a) (b)

Figure 6.5 (a) The Turrini-Piazza method. (b) The Alessi, Lamborghini, Raffagli system, (Barbisan and Laner, 1997).

6.2.3 The Tampone (1992) system

Consists on inserting on the wooden beam one or more metal blades, dry linked (bolts) or linked with epoxy resin. This way the fiber continuity of the wood is maintained. The steel blades can be welded also in the case of the connectors, (Figure 6.6a). The inclusion of a T-profile, attached with resin on the top part of the wooden beam, as can be observed in Figure 6.6b, it is a variant of the Tampone system. To the superior flange it’s welded a truss for the connection of the concrete, (Barbisan and Laner 1997).



(a) (b)

Figure 6.6 The Tampone system, (Barbisan and Laner, 1997).

The major issues concerning the implementation of interventions with reinforced concrete slabs are due to the cast of concrete. It is very difficult to prevent the contact between the deck and the still fluid concrete, the leaching through the cracks between boards and beams can ruin any valuable soffit located at the intrados of the floors. Also the weight increase of the floor, although small when compared with the complete replacement of the existent floor with a new on in concrete-tile, can affect the stability of the entire building due to possible failure of the foundations and of the walls that weren’t properly consolidated.

The problematic of load increase its connected not only with the inclusion of heavy materials on the structure but it's inherent in the problem of reuse of the historical structure, sometimes the change in the use, implies a considerable increase on the live loads like for example from the 0.75kN/m² typical of houses from the beginning of last century to the currently used 2.0kN/m², arriving at 5.0kN/m² for crowded rooms or libraries, (De Micheli, 1993).

However, the slab guarantees a noticeable stiffness, also in-plane, and even the possibility of connecting the floor to the walls through R.C. beams. The effects of this intervention are assessed in terms of load distribution between the vertical elements and in terms of mass increase.

6.3 IMPROVEMENT INTERVENTIONS THROUGH THE USE OF STEEL ELEMENTS

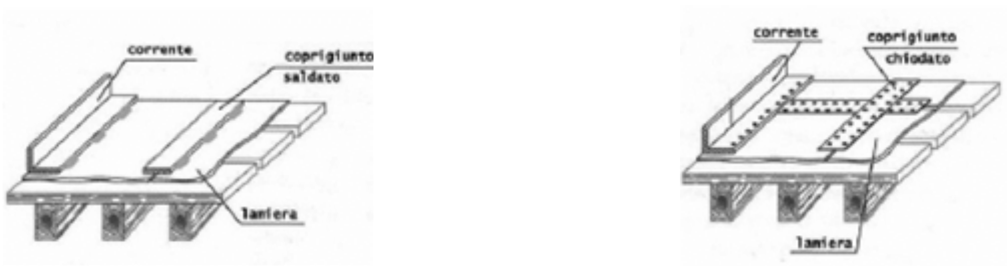
6.3.1 Metallic plates

The technique proposed by Giuriani and Plizzari in 2003, provides the in-plane stiffness of the floor with steel plates bolted to the existing planking, Figure 6.7.

This technique is used under totally dry conditions what excludes the possibility of damaging existing valuable ceilings (intrados) on the floors or walls with frescos. Moreover, the nailed connections between the plates avoid further damage risks from fire caused by the welding process.

However, this reinforcement technique, as well as presenting higher costs than Others solutions, doesn't increase the floor's bending stiffness and is ill-suited to the geometry, often irregular, of historical constructions.

The weak point of this intervention it's the joining of several elements, (Giuriani and Plizzari, 2003). Moreover, this reinforcement technique is applicable almost exclusively in the case of simple geometries; the problematic related to the weight increase and to the economic factor are particularly restrictive for the execution of this type of intervention, (Borri, 2004b).



(a) (b)

Figure 6.7 In-plane stiffening of floors with metallic plates: (a) welded plates, (b) nailed plates, (Giuriani and Plizzari, 2003).

6.3.2 Metallic diagonals

The idea of including on the floor's intrados a horizontal bracing composed of steel ties and arranged in S. Andrea Cross is present on one of the most noticeable anti-seismic construction guides from the beginning of the century, "*Masciari Genoese Costruzioni Antisismiche*".

This simple and rapidly applied solution consists on attaching to the floor two metal diagonals ties using screws or nails, Figure 6.8.

Since the steel elastic modulus is much higher than that of wood, the tensioned metal strip will undergo negligible deformation when compared to those of wood, behaving rigidly. For this reason it is important the choice of connections, which are responsible for absorbing the forces generated by the deformation of the diagonal, and unload it on the wood causing in the typical staving plastic deformation near the connection.

However, this reinforcement technique, doesn't increase the out-of-plane bending stiffness of the floor and, given the length of the connections and the reduced thickness of metal diagonals, has the disadvantage of having to fix the reinforcements at the points of overlap with the floor's main beams.

The increase in the number of connections is not so much a function of the stiffness increased as of an improvement of the stiffening side effects such as, ductility or energy absorption. Indeed, in terms of ductility the behaviour of a system with high hyperstaticity allows to dissipate a large amount of energy without significant loss of resistance, (Ceccotti and Vignoli, 2002).

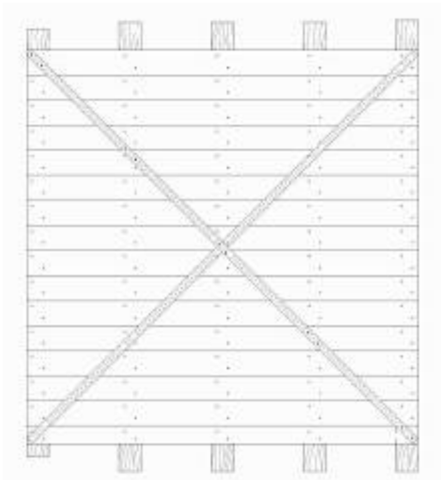


Figure 6.8 In-plane stiffening with metallic diagonals.

6.3.3 Intervention using metallic plates and diagonals (Gattesco et al., 2007)

This intervention technique is of dry application and has a high reversibility index. It consists on placing over the deck of the existing floor, in correspondence to the beams, a steel plate with holes attached to the beams with steel pins. These are forced into holes drilled in the main beams, with a fixing length of at least seven times the diameter and are then welded to the steel plates.

This technique creates an increase of the bending stiffness of the main beams of the floors creating a T shape composed beam with deformable connection, where the web it's composed by the existing wooden beam and the flange by the steel plate.

This intervention is of fast to implementation, if it doesn't involve the removal of the existing planking. The planking as a great influence on the strength and stiffness of the connection but does not offer a noticeable contribution to the flexural stiffness nor to the load capacity of the floor, as so, from a static point of view it's considered only as a spacer between the wooden beam and the steel plate.

To ensure horizontal diaphragm behaviour the intervention comes integrated through a series of steel plates arranged diagonally and attached to an L-shaped steel element anchored to the wall through steel bars inserted into holes that have a length slightly inferior than the thickness of the wall and injected with cement mortar with compensated shrinkage. In this way, it's ensured an effective distribution of horizontal forces between the shear walls and the out-of-plane deformations of the walls placed in the orthogonally to the seismic action are contrasted, Figure 6.9.

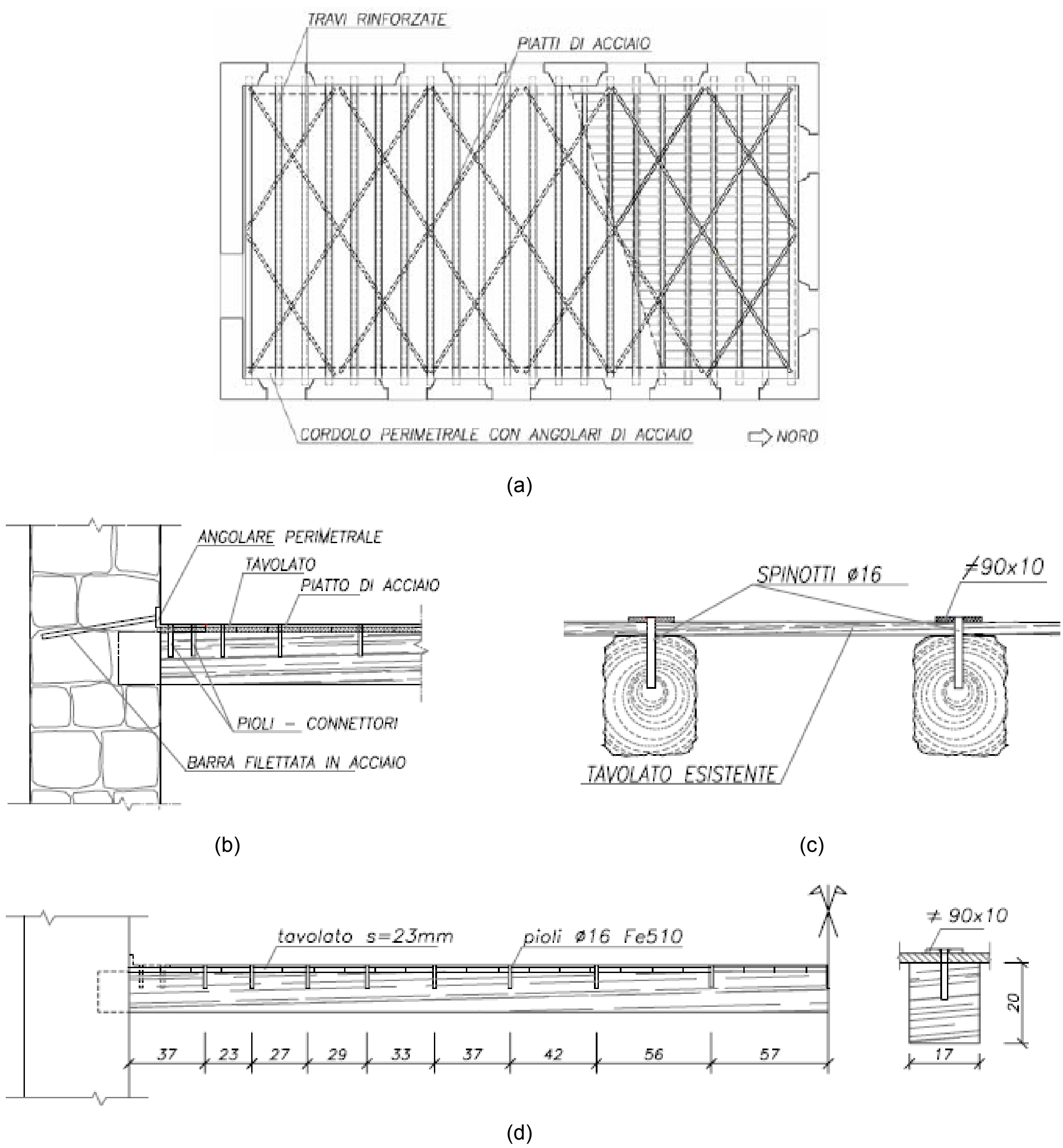


Figure 6.9 Intervention using metallic plates and diagonals, (Gattesco et al., 2007). (a) Technical prospect of the reinforcement. (b) Detail of the anchorage to the wall. (c) Transversal section. (d) Pins distribution along a main beam and correspondent transversal section.

6.4 IMPROVEMENT INTERVENTIONS THROUGH THE USE FIBRO-REINFORCED COMPOSITE MATERIALS

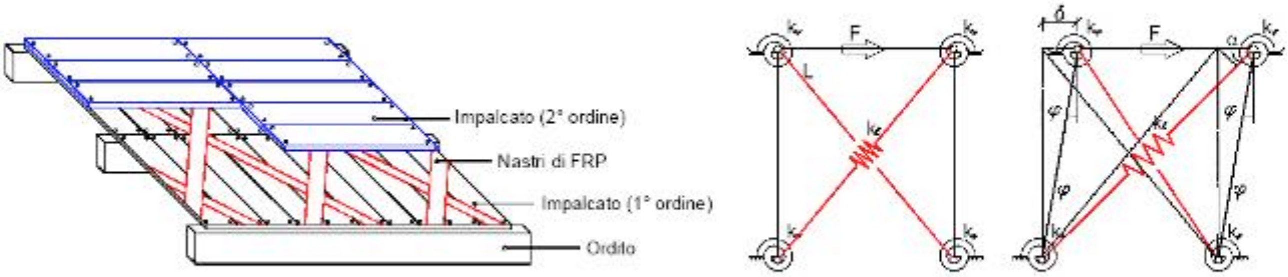
Among the improvement techniques there are those that consist on introducing strengthening elements in fibro-reinforced composite materials.

The contribution of FRP strips, (Gentile et al., 2002), (Borri 2004b) and (Angotti et al., 2005), applied at the floor underside can increase beam structural performance, with or without specific strengthening at the upper side. Mechanical behaviour strongly depends on the local bond at the interface between wood and fibres subjected to various types of action (debonding, peeling), and may be particularly sensitive to the environmental conditions of the wood, mainly the relative humidity of the substrate. Italian standards recommend a limit of 10.0% relative humidity to ensure proper adhesion of fibres to porous materials like masonry, (CNR-DT200, 2004), and the use of polymer resins able to ensure durability, in cases of mechanical and dimensional variations in timber materials (CNR-DT201, 2005). In wood, relative humidity of the substrate of 12.0% seems to be adequate to allow proper bond behaviour of glued elements, (Piazza et al., 2005).

6.4.1 Intervention using FRP, (Angotti et al., 2005)

The type of in-plane floor stiffening, described below consists on the application over the planking, of bracing crosses in fiber-reinforced composite strips, as shown in Figure 6.10a.

It is possible to analyze the effectiveness of the strengthening solution by using the simplified diagram shown in Figure 6.10b.



(a) (b)

Figure 6.10 Scheme (a) and calculation model (b) of a typical floor reinforced with FRP, (Angotti et al., 2005).

Despite its simplicity, the described model provides important information on the reinforcement characteristics in order to ensure the required in-plane stiffness of the deck planking.

The main features are presented next, (Angotti et al., 2005):

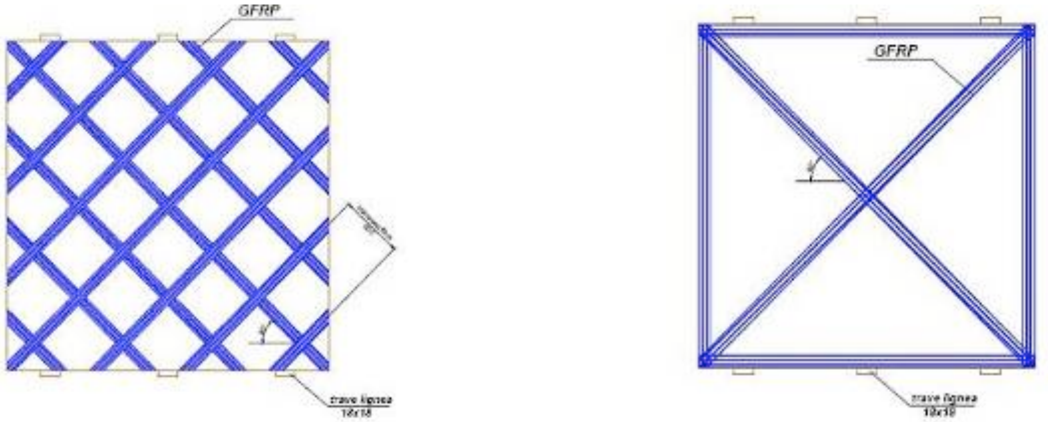
- The reinforcement can be placed diagonally, with a behaviour similar to that one presented by the metallic braces, using only the tensile strength characteristics.
- The composite material must be unidirectional, with adequate axial stiffness sufficiently thin and deformable for out-of-plane actions.

Others considerations may arise from practical aspects related to the implementation of the intervention. Some of them are listed next, (Angotti et al., 2005).

- The most qualified materials to provide reinforcement are the ones with glass-fiber, for their modest elastic modulus, that allows them to accommodate not only the wood natural movements but also the inevitable movements of the floor, without causing excessive stresses on the bonded interface.
- The type of product more adapted to the mechanical characteristics are certainly the ones available in the form of fabrics, pre-impregnated on site, available in convenient rolls, from which can be obtained with ease strips of suitable length.
- With the objective of preventing the instability phenomena of the compressed reinforcements, of yielding more uniform the behaviour of the floor in the two main directions, of protecting the reinforcement intervention and improving the adherence it is important that at the extrados the intervention is completed by a second deck. The boards of this last one are arranged perpendicular to those of the first deck, to which must be attached using nails or screws, (Figure 6.10a). Before the installation of the second deck, the upper surface of FRP reinforcements can be further covered with glue, so as to improve the attachment between reinforcement and floor and between the two decks. When finished, the surface of the extrados reinforced floor will be very similar to that attained in the absence of reinforcement, making possible the common surface treatments.

6.4.2 Intervention using GFRP, (Borri 2004b)

This reinforcements presented by Borri in (Borri 2004b) consist on the application of strips of composite materials made of unidirectional glass fibers (GFRP) according to the layouts presented in Figure 6.11. The bonding of the fibers to the pre-existent structure is executed using bi-component epoxy resin.



(a) (b)

Figure 6.11 Layout of the GFRP reinforcements, (Borri 2004b).

6.5 FINAL REMARKS

6.5.1 Available experimental procedures and analytical and numerical models

In the last decade, many research works have been focused on the characterization of compatible techniques, able to strengthen and rehabilitate original timber floors, (Modena et al., 1998), (Modena et al., 2004), (Giuriani, 2004), (Liberatore et al., 2004), (Piazza et al., 2005), (Corradi et al., 2006), (Gattesco et al., 2007), and to provide a suitable “box” behaviour, (Tomaževič, 1999), (OPCM 3431 2005).

In this context, the study of the influence of deformable floors on the seismic behaviour of existing masonry buildings has not been sufficiently investigated up to now, both at experimental, (Tomaževic, 1991), (Mezzina, 1997), (Galasco et al., 2001), (Liberatore et al., 2004), (Abrams and Tena-Colunga, 1994), (Corradi et al., 2006) and numerical level, (Moon and Lee, 1994), (Cohen et al., 2001), (Lee et al., 2002) and (Kim and White, 2004).

Moreover, only few works are available on the role of the connections between vertical and horizontal structural elements in the dynamic behaviour of masonry buildings, (Tomaževič, 1999), (Giuriani, 2004), (Foraboschi, 2005) and (Sorrentino et al., 2007).

In particular, some laboratory experimental campaigns have been performed on full scale floor samples or elements, subjected to in-plane monotonic or cyclic loads, (Modena et al., 1998), (Giuriani and Plizzari, 2003), (Peralta et al., 2004) and (Corradi et al., 2006). They propose various setup, related to different basis conception. In Modena et al. (1998), a simple diagonal test on square samples of timber floor portions (125.0x125.0cm), rotated of 45° and positioned vertically under a press, was used. This test, usually standardized for masonry walls, is very easy to perform and avoids undesirable confining and frictional effects; however, it is not completely representative of the real floor shear behaviour, due to the limited geometrical dimensions of the sample, the way how the load is applied, and the probable instability of the planking. Moreover, only monotonic test can be easily performed. Alternatively, test set-up using full-scale specimens lying horizontally were developed by Giuriani and Plizzari (2003). and Corradi et al. (2006). These shear testing systems allow monotonic and cyclic tests without instability of the planking and by minimizing frictional effects. Anyhow, they seem providing a not negligible confining effect, due to the presence of active steel frames, that force the shear behaviour all around the floor specimen. In Peralta et al. (2004), a monotonic and cyclic bending test setup on horizontal floor samples, treated as simple supported or double fixed ends beam, was presented. This procedure enables the investigation of the shear behaviour of the floor as a part of the flexural deformation, as in the beam model of Timoshenko (1934). By this system it is possible to perform tests without the disadvantage above, but it requires specimens in real scale with large dimensions (about 4.0x8.0m, as in Peralta et al. 2004).

7 IMPROVEMENT OF THE GLOBAL STRUCTURAL BEHAVIOUR

In general, the interventions should be aimed at improving the structural connections, and reducing horizontal diaphragm deformability, at increasing masonry strength; furthermore they should improve the behaviour of vaults, arches, pillars, etc. Nevertheless, in numerous cases very invasive structural modifications have been applied, probably as a result of the assumption that they should provide a higher safety level, but without any definite proof of their effectiveness.

There are many cases in which roof or floor reconstruction - during which original timber trusses or floors were replaced by new elements of reinforced concrete or steel - were probably the main cause of damage. Substitution of wooden roofs with reinforced concrete slabs is a very common rehabilitation technique in most recent intervention.

In most cases it is difficult to categorise the intervention in good and bad. Their seismic performance is, in fact, referred to the weak compatibility with the old structure or the poor application/workmanship.

The use of r.c. structures to retrofit historic masonry buildings was strongly supported by the changes in the building process in Europe, excluding masonry and turning to the extensive use of r.c. structures and cement mortars. Furthermore, the calibration of numerical procedures and research developed national codes mainly addresses to R.C. which increased the neglecting of masonry building tradition but also of the repair materials.

The post-earthquake surveys after the recent earthquake have shown poor performance of heavily reinforced buildings that often made them more vulnerable. It appears that a large number of the damaged buildings had been modified over the years to meet the needs of their users also following in Italy the seismic code, but that such measures had rarely improved their resistance against seismic events. As a matter of fact, these functional modifications, (remodelling, raising, enlargement, opening or closing of new windows or doors) frequently added vulnerability. A common form of upgrade has been the replacement of the old wooden floors with new floors of heavy reinforced concrete beams supporting hollow clay tiles. This was often done without upgrading the strength of the masonry bearing walls. Sometimes bearing walls were even removed to open up spaces, resulting in beams located where shear walls had once existed. The post-earthquake survey found that alterations and remodelling was rarely done with any heed of regulations or seismic design criteria.

The compatibility or incompatibility of materials and systems is a good indicator of seismic vulnerability. Recent earthquakes were particularly ruthless on masonry buildings that had been retrofitted relatively recently with reinforced concrete floors (usually with recast beams with large hollow clay tile elements spanning between the beams). One might have thought the stronger and stiffer diaphragms would have improved performance, but these floors were not properly tied through the masonry walls, and thus did not contribute to holding them together. A significant problem with this kind of construction is that the absence of wall ties is not easy to verify, while the presence of the new floors may give one the false sense of security that the structure has been improved.

The use of inadequate intervention techniques that alter significantly the original bearing system of the structure and/or are based on concepts that were proven to be reliable in case of modern structures and are applied (without prior validation) to historic structures as well. To mention an example, measures to ensure infinitely rigid diaphragms at floor and roof levels or to significantly enhance the ductility of an historic structure do not necessarily affect in a positive way the seismic behaviour of the building.

The above risks may be dramatically increased by inadequate analysis of historic structures.

One of the first aspects to be taken into account when dealing with the seismic behaviour of existing masonry buildings is the lack of good connections between structural elements.

Hence, to allow the structure to manifest a satisfactory global behaviour, it is necessary to improve the connections between masonry walls, and between walls and floors and walls and roofs (Tomaževič and Weiss, 1994), (Tomaževič and Lutman, 1996a,b), (Tomaževič, 1991 and 1999). The observations of masonry buildings when subjected to earthquakes have shown that the behaviour is strongly dependent on how the walls are interconnected and anchored and to floors and roofs. In old structure the unfavourable effect of insufficient anchorage between walls and between walls and floors was often observed. Irregular structural layout in plan, large openings and lack of bearing walls in both directions often caused severe damage or even collapse. A good quality of the connections between floors and walls, between roof and walls and between perpendicular walls is also crucial to reach a good global seismic behaviour of the building. Good quality connections will drive the collapse of the construction to a configuration that requires a stronger seismic action (Borri, 2009).

This goal was in the past achieved by inserting ties or confining timber rings at the top of the building or at the floor level, with a regular layout. An effective connection between floors and walls is useful since it allows a better load redistribution and applies a restraining action towards the walls overturning. In the case of timber floors, a satisfactory connection was provided by fasteners anchored on the external face of the wall.

The use of this linking system shows again the manner of the earthquake loads resistance.

7.1 INDEPENDENT STEEL OR CONCRETE FRAME

The introduction of reinforced concrete frames, cages or cores (Tolles, 2006) is well documented both masonry buildings in Italy and for adobe buildings in California, such as the Sonoma Barracks, Sonoma State Historic Park, Petaluma Adobe State Historic Park in Sonoma Country, Plaza Hotel in San Juan Bautista, the Cooper-Molera Adobe, Monterey, and Mission La Purissima Conception, Lompoc.

Several collapses or heavy damages of such hybrid structures were extensively documented in post-earthquakes surveys in Italy since the 1997 earthquake (see the proceedings of the several Italian Earthquake Engineering Conferences – ANIDIS). Damages were mainly related to the stiffness changes or to hammering effects of the tow structures.

At present the tendencies are to remove such intervention.

For adobe buildings, the effectiveness of this technique is not supported by testing nor by post-seismic on site investigation. One example was the damage suffered by the Pisco Cathedral during the 2007 Pisco Earthquake in Peru. Cancino (2009) observed that the retrofitted concrete dome and columns of the cathedral were the only elements to survive the earthquake, and attributed the collapse of the side walls and vaulted roof to the combination of reinforced concrete elements and earthen materials, by explaining that the concrete probably pounded the earthen elements to collapse.

Due to the invasiveness of this technique and the incompatibility resulting from the use of elements much stiffer than the earthen elements, this technique is not considered within the scope of the NIKER project.

8 FINAL REMARKS

The general overview of the main repair techniques shows several criticisms in the decision process, lacking Standards/Recommendations but also guidelines which could help in the choice.

Also most of research is mainly aimed at the study of mechanical aspects of the techniques, without an addressed deepening of the laying down procedures and possible problems, the durability, the maintenance aspects, the on site controls etc...

It is worth to remind, that several handbook are available in literature, but most of them describe shortly the single techniques. Up to now, the following aspects are not yet considered:

- Maintenance suggestions and periodic controls/monitoring;
- Long term performance / durability;
- Life time;
- Standards and/or Recommendations;
- Parameters to take into account in analytic procedures;
- Laboratory controls and on site controls;
- Control parameters of the effectiveness of the intervention;
- Analytic procedures and structural modelling.

In case of interventions, recently elaborated and proposed, developed with the purpose of deeply increasing ductility, the implications of ductile behaviour in terms of conservation of CH assets are not clearly evidenced. It has to be noted that 'good' ductile performance means that large deformations and very severe damage occur, not only in the new added materials and/or components, but also in the original materials/components to which they are structurally connected. This usually corresponds to loss of historic and artistic value, and even loss of any residual life after the earthquake.

Some intervention techniques lead to significant modification of the original structural behaviour. Even though those intervention techniques may be considered "in principle correct", as they contribute to significant resistance improvement and/or redistribution of seismic loads and/or ductility enhancement, due to the limitations related with the theoretical/numerical models, the real behaviour of the strengthened historic assets cannot be accurately predicted. As a result, some intervention techniques of this type may represent a severe threat for CH structures (see for examples the effect of substituting floors and roofs with reinforced concrete diaphragms.

Proposed solutions often claim to include such critical issues as 'removability/reversibility', 'compatibility', 'low intrusiveness', but lack to demonstrate these issues in practice. It has to be admitted that those concepts are defined in a rather ambiguous way, thus leaving room for arbitrarily declaring that those concepts are satisfied by the proposed solutions, rather than demonstrating the efficiency of the proposed intervention schemes.

A summary of repair techniques for masonry and earthen structures with respective advantages and disadvantages is presented in the following tables.

Location / Structural element	Damage /Problem	Possible causes	Parameters indicating level of damage	Possible strengthening actions
Mortar Joints	Deterioration of mortar Sanding Push out Loss in adhesion Other	Environmental agents Biological agents Other	Depth of deteriorated mortar Cause of damage Loss in strength properties/ functioning Other	Repointing Reinforced repointing Reconstruction of the section with new binder Other
Wall-section	Seperation of leaves Loosening of infill Bulging Spalling Crushing of units Other	Deterioration of infill Overloading Eccentring loading Differential settlement Other	Width of bulging Cause of damage State of infill Location of spalling and Crushing Other	Grout injection Tying Prestressing Deep repointing Jacketing Other
Wall- plane	Cracks (vert./hor./dia*.) Crushing of units Displacement Leaning Settlement Other	Change in load transfer mechanism Overloading Differential settlement Environmental agents Overturning Other	Width/pattern of cracks Location of cracks/crushing Amount of displacement (Vertical/horizontal) Degree of tilting Cause of damage Condition of foundations Other	Grout injection Tying Prestressing Anchoring Strutting Buttressing Confinement External reinforcement Stitching Repointing/Reinf. Repointing Substitution Enlargement Other
Columns and Piers	Cracks (vert./hor./dia*.) Displacement of units Tilting Bulging Crushing Other	Overloading Change in load transfer mechanism Differential settlement Seperation of leaves (if any) Environmental agents Other	Width/pattern of cracks Location of cracks/crushing Amount of displacement (Vertical/horizontal) Degree of tilting Cause of damage Condition of foundations Other	Confinement External reinforcement Substitution Enlargement Tying Prestressing Other
Arch, Vaults and Domes	Cracks (vert./hor./dia*.) Widening of abutments Slipping of voussoirs Distortion of shape Other	Inadequate lateral support Overloading Differential settlement Other	Width/pattern of cracks Location of cracks Cause of damage Condition of foundations Other	Tying External reinforcement Prestressing Buttressing Enlargement Confinement Other
Foundations	Cracks (vert./hor./dia*.) Deterioration Crushing Overturning Other	Differential settlement Alteration in water table level Eccentricity in loading due to settlement Inadequate design Alteration of surrounding site conditions (constr. of new buildings around etc.) Other	Degree of out of plumb of vertical elements Well patterned cracks on the façade/upper structure Cause of damage Other	Enlargement Underpinning Soil improvement Replacement External reinforcement Other
Towers, Bell towers and Minarets	Cracks (vert./hor./dia*.) Crushing Leaning Creep Seperation of leaves Other	Overloading Differential settlement Ageing Inadequate horizontal Confinement Deterioration of infill Other	Degree of out of plumb of structure Well patterned cracks on the structure Cause of damage Other	Grouting Confinement Anchoring Tying Other

** Vertical / Horizontal / Diagonal*

Table 8.1 Damages at different structural elements with possible causes, indicators and possible strengthening actions.

	Considerations	Pros	Cons
Repointing Reinforced repointing	-Compatibility of mortar with original mortar and unit -Deep removal of external deteriorated mortar is necessary -Size and corrosion resistance of reinforcing bars -Use of sacrificial mortar -Other	-Increase in strength of masonry component -Effective in prevention of water penetration through joints -Confining effect in the masonry component especially in reinf. repointing - Can be reversible depending on the mortar used -Other	-May accelerate the decay of masony units unless the mortar is compatible -May not be effective in irregular masonry -May lead corrosion if not proper reinforcing bars and enough cover is provided -Other
Grout Injection	-Compatibility of injected material with original mortar and unit -Suitability of wall section for injection -Decision on injection pressure and preperation of masonry for injection -Determination of specific locations for grouting -Other	-Can restore the uniformity homogenity of strength in section -Effective in prevention of water penetration through cracks and voids -Can restore the continuity of multi leaf wall sections as a result increasing the strength of the element -Other	-Irreversible action - Not suitable for walls with low percentage of voids and loose infill masonry -Grout may cause segredation and shrinkage -May not be effective due to lack of precise knowledge on the distribution of voids and their sizes -Other
Substitution	-Must be differentiable from the original fabric but not obtrusive -Compatibility of new material with the original material -Must be preferred after all methods proved to be ineffective -Other	-Recover the original function of structural element -Increase the stability ad integrity of structure -Improve the global behavior of the structure -Might be reversible depending on the size of substitution -Other	-Mostly irreversible -Obtrusive especially when the structural system is substituted -May violate the minumum intervention principle -Other
Stitching	-Compatibility of material used for stitching with original material -Must be prefered after other methods like tying, grouting, confinement, anchoring etc proved to be ineffective -Other	-Improve the connections of adjacent walls - Add extra strength and ductility to the wall -Can control the crack propagation -Other	-Might be reversible by harming the original fabric -Can be obtrusive depending on the type shape of material used -May need intense intervention for the placement of plates etc -Other
Tying	-Compatibility of material used for tying with original material -May necessitate anchorage -Other	-Effective in compensating the weakness of masonry in tension -Generally necessitate minor alteration /reversible -Can ensure stability and integrity of the element -Other	-May lead corrosion problems if tying bars not selected properly -May be obtrusive in some cases -Other

Table 8.2 Some basic strengthening methods with their advantages and limits.

Confinement	-Compatibility of confining material with original material -Determination of locations for confining rings/bars/tis -Other	-Effective in increasing the capacity of structural element it is applied -Effective in prevention of seperation of external leaves in mutli-leaf walls -Improve the stability and stiffness of whole structure if the structure is confined -Can be reversible -Other	-Depending on the material used it can be obtrusive -May lead deterioration problems if not protected against/selected accordingly when exposed to environmental effects -Other
Jacketing	-Transversal connections of jackets and masonry walls -Bonding between nets and masonry walls -Compatibility of new material with the old one -Other	-Increase strength and ductility of the wall -Improve the global behavior of the structure -Other	-Irreversible and highly invasive -May lead eccentricity under dynamic loading since it increases stiffness at particular element where applied -Other
External Reinforcement	-Location of application -Determination of reinforcing material -Compatibility of material with original material -Other	-Increase the strength and ductility of structure -Improve the global behavior of structure -Can be effectively used in curved elements, walls etc -Can be reversible depending on the material used -Other	-Heat and radiation sensitivity of FRPs -Loosening of the bond due to moisture for FRP strengthening -Might be obtrusive depending on the material used -Other
Anchoring	-Location and type of anchoring -Other	-Can increase the stability of structure or element -Can be non obtrusive depending on the place of application -Other	-May be invasive the historic fabric -May change the appearance especially when applied externally -Other
Prestressing	-Amount of compression applied to the member or structure -Stability of anchorages at the ends -Other	-Increase the capacity of member -Improve the global behavior and integrity -Can be applied externally and internally on element or structural level -Other	-May lose the effectiveness in time -Other
Buttressing	-Location of application -Connection of new construction with the old part -Other	-Increase the stiffness and integrity of the structure -Improve the global behavior -Confine the structure horizontally -Relieve stresses in certain places	-Alteration in appearance -Might not be reversible depending on size and form of the buttressing Other
Strutting	-Selection of strutting -Other	-Can provide an immediate and reliable supporting -Can be easily replaced -Other	-Mainly temporary solution -Not improve but maintain the current condition of structure -Other

Table 8.3 Some basic strengthening methods with their advantages and limits.

Version		Advantages	Disadvantages
1.	Masonry Bond	• Simple installation technique • No special equipment needed • No areas/parts possessing physical and structural-physical material properties other than earth block masonry	• Some historical earth blocks to be removed • Historic rendering damaged by masonry bond
2a.	Stainless Steel Reinforcing Bars	• Following field-proven repair methods • No corrosion (except in chloride-containing environment)	• Loss of historical earth block and joint material in slit areas • Areas/parts possessing material properties different from earth block masonry; particularly problematic: thermal length variation • Installation by briefed professionals only • Unprofessional installation will limit effectiveness
2b.	Glass-fibre Reinforcing Bars	• If trass-lime mortar is used physical and structural-physical properties near-similar to earth block masonry • Simple installation • Lightweight • Little thermal length variation • No corrosion	• Loss of historical earth block and joint material in slit areas • Repair method not field-proven • Installation by briefed professionals only • Unprofessional installation will limit effectiveness
3.	Geogrid	• Little, isolated interference with historical structure necessary due to injection anchors • Simple installation of geogrid due to grid's light weight • High tensile strength, little elongation • Flexible structure • Final rendering with good adhesion to grid	• Irreversible bond between injection mortar and earth block masonry • Little elongation may lead to (limited) crack formation • Geogrid without effect if installed unprofessionally

Table 8.4 Comparison of repair options for earthen structures developed by ZRS.

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