Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 49, No. 2, June 2016

SEISMIC ASSESSMENT AND IMPROVEMENT OF UNREINFORCED STONE MASONRY BUILDINGS: LITERATURE REVIEW AND APPLICATION TO NEW ZEALAND

Marta Giaretton[1], Dmytro Dizhur[2], Francesca da Porto[3] and Jason M. Ingham[4]

(Submitted *December 2015*; Reviewed *March 2016*; Accepted *March 2016*)

ABSTRACT

Following the 2010/2011 Canterbury earthquakes considerable effort was applied to the task of developing industry guidance for the seismic assessment, repair and strengthening of unreinforced masonry buildings. The recently updated "Section 10" of NZSEE 2006 is one of the primary outputs from these efforts, in which a minor amount of information is introduced regarding vintage stone unreinforced masonry (URM) buildings. Further information is presented herein to extend the resources readily available to New Zealand practitioners regarding load-bearing stone URM buildings via a literature review of the traditional European approach to this topic and its applicability to the New Zealand stone URM building stock.

An informative background to typical stone URM construction is presented, including population, geometric, structural and material characteristics. The European seismic vulnerability assessment procedure is then reported, explaining each step in sequence of assessment by means of preliminary inspection (photographic, geometric, structural and crack pattern surveys) and investigation techniques, concluding with details of seismic improvement interventions. The challenge in selecting the appropriate intervention for each existing URM structure is associated with reconciling the differences between heritage conservation and engineering perspectives to reinstating the original structural strength. Traditional and modern techniques are discussed herein with the goal of preserving heritage values and ensuring occupant safety. A collection of Annexes are provided that summarise the presented information in terms of on-site testing, failure mechanisms and seismic improvement.

INTRODUCTION

Existing load-bearing stone unreinforced masonry (hereafter referred to URM) buildings in New Zealand were constructed in the second half of the 19th century by early European settlers, in particular between 1860 and 1890. Local geology and the extent of access to a supply of local natural stone largely determined the number of URM buildings and the type of stone used in construction in a given region. An inventory of the load-bearing URM buildings of New Zealand was previously compiled, listing location, construction details and architectural configuration in order to identify traditional construction practices and seismic vulnerabilities [1]. The authors identified 668 stone URM buildings that are currently in existence in New Zealand.

As in many other countries, the construction of URM buildings in New Zealand commonly did not take into consideration building stability and safety when subjected to earthquake induced lateral loads. The inherent vulnerability of New Zealand URM buildings was once again highlighted during the damage assessment inspections following the 2010/2011 Canterbury earthquake [1], [2], [3] with damage being partly attributed to the poor quality of constituent materials (particularly weak mortar), water-induced deterioration, the lack of connectivity between leaves in the wall cross section and among various structural components, and inadequate maintenance. Vulnerability and failures were also observed as a result of inappropriate repairs and additions to URM structures using incompatible materials and construction techniques that adversely impact structural integrity [2]. Excessive strengthening may cause unintended catastrophic brittle behaviour which counters the capacity of more resilient URM structures to absorb and dissipate forces and can lead to immediate collapse. Resilient structures sustain damage during an earthquake but allow time for people to evacuate.

The information presented herein is intended to provide New Zealand professional structural engineers with a general guidance resource and to provide references to applicable literature on the seismic assessment and improvement of URM buildings, focusing specifically on natural stone structures. The majority of the advice and recommendations presented herein are based on experience and observations made following previous earthquakes that occurred in Europe and the latest research that is being conducted at universities in the European Union. A brief summary of the seismic vulnerability assessment procedure for masonry buildings is shown in Figure 1 and is discussed in the following sections. Da Porto et al [4] present an example of assessment and repair of an unreinforced stone masonry building damaged during the 2009 L'Aquila earthquake (Italy), using the procedure herein discussed.

[1] *Corresponding Author, PhD Candidate, University of Padova, Padova - Italy, marta.giaretton@dicea.unipd.it*
[2] *Lecturer, The University of Auckland, Auckland - New Zealand, ddiz001@aucklanduni.ac.nz (Member)*
[3] *Associate Professor, University of Padova, Padova,- Italy, francesca.daporto@dicea.unipd.it*
[4] *Professor, The University of Auckland, Auckland - New Zealand, j.ingham@auckland.ac.nz (Member)*

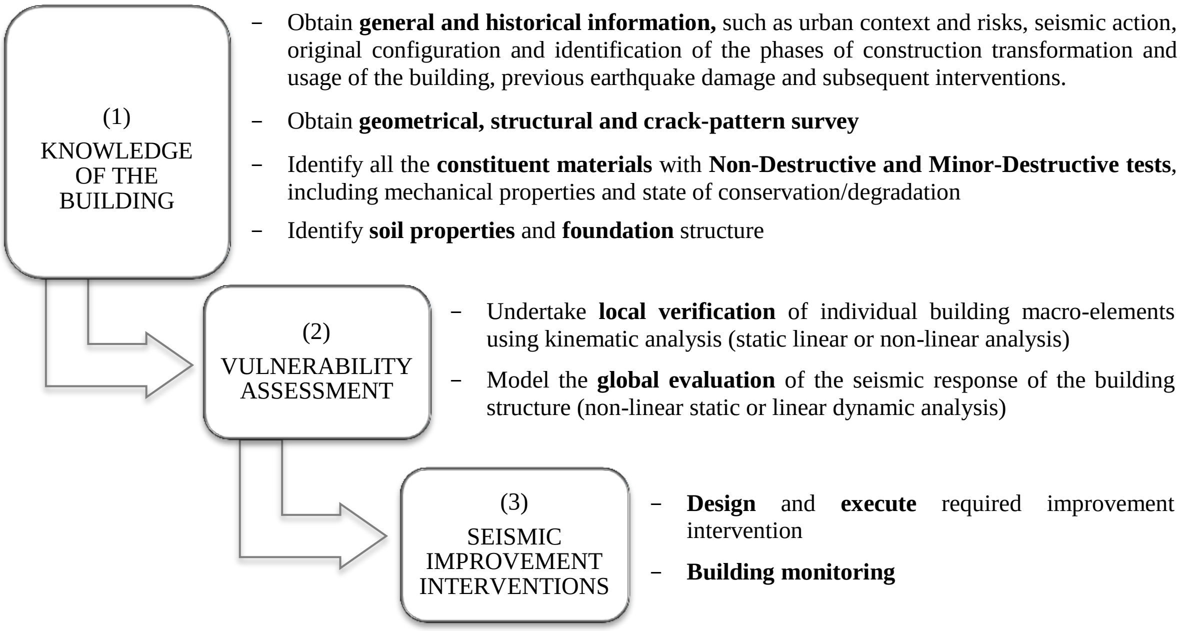


Figure 1: Seismic vulnerability assessment procedure for URM buildings.

URM BUILDINGS IN NEW ZEALAND

Load-bearing stone URM buildings in New Zealand are typically stand-alone and regular in shape, both in plan and elevation, having typically one or two storeys and a foot-print area of less than 200 m^2 [1]. Floors and roofs are typically made with timber and the connections between walls and floors/roof or between load-bearing walls are commonly insufficient when subjected to horizontal seismic loads. Rubble stonework was often chosen when using local schist or basalt in construction, while ashlar stonework was preferred when using limestone [5], [6], [7]. Nevertheless, during the survey of stone URM buildings [1] it was observed that even if the outer leaf appears to be well constructed using regularly arranged stones and strong mortar, an in-depth assessment may show the widespread use of an irregular rubble core and surface repointing of mortar joints. Structural mortar is likely to be weak or locally absent and the wall cross-section is often composed using different materials, including stone, clay bricks and concrete. Several public URM buildings in city centres were identified to be of stone facing with a clay brick back leaf or a cast in-situ no-fines concrete core [1], whereas full cross-sections constructed of natural stone were mainly used in rural buildings. Both stone/clay/concrete composite construction and stone-only construction usually lack connections between leaves, resulting in increased earthquake vulnerability.

The inspection of representative case study buildings allowed a thorough understanding of the mechanical and physical material properties to be acquired [8]. Original structural mortar used in the construction of New Zealand stone URM buildings is typically lime based with a high percentage of sand aggregate, resulting in low compressive strength and locally eroded and deteriorated mortar joints (f'_m = 0.2-2.8 MPa), with pointing mortar identified to be 3 to 7 times stronger than structural mortar. Regarding the natural stones used, the compressive strength is greatly influenced by extraction site, crystal grain size, mineral composition, bedding planes and foliation. Igneous rocks such as phonolite, Otago trachyte, basalt, hyperite, tonalite and granite have high compressive strength, ranging from 140 to 300 MPa, compared with other construction stones that have a compressive strength of less than 100 MPa. Limestone and tuff (red scoria and vitric tuff) are the softer types of natural stones, with tested compressive strengths ranging from 3-35 MPa [8]. Annex 2 provides default probable material properties for stone masonry, but it is strongly recommended that an appropriate material testing campaign be performed whenever possible.

KNOWLEDGE OF THE BUILDING

Evaluation of the safety of the building should be based on both qualitative methods (documentation and observation) and quantitative methods (experimental and analytical) that take into account the effect of seismic activity on the structural behaviour of the building. The common problem for existing buildings is the lack of knowledge regarding the original composition, geometrical layout and material properties of the structure. This lack of knowledge can be overcome with various degrees of surveying operation, historical research and experimental investigation that can be divided into the following steps [9], [10], [11], [12]:

1. *Collect general and historical information regarding the building*, including its location in relation to particular risks (such as proximity to slopes, or possibility of liquefaction, flood, tsunami, and other factors that could increase the risk of damage and deterioration), the seismic hazard, and the connection of the building within its surrounding urban context (particularly in the case of row buildings or clusters of buildings). During this stage it is necessary to refer to existing documents, such as original plans, proposals for structural interventions, previous earthquake damage reports and subsequent repair actions, and to hypothesize the construction hierarchy in order to understand and identify structural and geometric modifications that could be the cause of local vulnerabilities.

2. *Geometrical, structural and damage pattern survey* of the building in its current state, well-documented by a complete *photographic survey*. The geometric survey is intended as the accurate measuring of all the parts of the building (structural, non-structural and ornamental) identifying shape, dimensions, and locations. The structural survey should focus on the identification of the load-bearing walls and their cross-section, constituent materials of the structural elements, type of floor

with dimension of beams and junctions, construction details, and possible previous interventions. It is also important to identify any existing crack-patterns and deformations as one of the main sources of the building's existing vulnerability. A complete photographic documentation of the state of the building is also necessary.

3. *Characterise all the constituent materials* (stone, clay bricks, mortar, concrete, timber, steel, and others) focusing on the type, state of decay [13], [14], and material properties. The investigation should firstly consist of a visual inspection identifying as much information as possible and the type of tests required. Testing locations should be carefully selected to be representative of all the structural elements of the building and to ensure that sufficient data is collected to enable assessment of the entire structure. Guidance on site testing is provide in Appendix 10A of the revised NZSEE Section 10 [15] and in Annex 1 herein.

4. *Identify the soil properties and foundation structure*. Soil properties include physical and mechanical characterisation of the soil, definition of underground waterways and their seasonal variation, and susceptibility to liquefaction or to instability due to natural slopes. Geometry, depth and material of the foundations are also required.

Investigation Techniques

The reliability of the seismic assessment, the capacity to design a retrofit intervention with high performance and low cost, and the effectiveness and durability of the intervention are greatly influenced by a suitable preliminary investigation campaign. Tests are classified into three main groups, in relation to their invasiveness and to the required accuracy of the results: (i) Non-Destructive Tests (NDT), (ii) Minor-Destructive Tests (MDT), and (iii) Destructive Tests (DT), [16], [17]. A brief summary of the required tests versus obtained parameters is presented in Annex 1.

NDT tests such as sonic pulse velocity, radar tests and infrared thermography allow hidden characteristics of the masonry to be found, including voids, construction flaws, the presence of different materials, the number and depth of each leaf in the wall cross-section, and the effectiveness of grout injections. With *sonic pulse velocity*, a signal is generated by percussion with an instrumented hammer and is received by means of an accelerometer, see Figure 2. The data processing consists of measuring the transit time between the transmitter and the receiver. Recommendation references and examples are available in [18], [19], [20].

Figure 2: Example of direct sonic pulse velocity test [Source: University of Padova].

Radar tests entail the propagation of short electromagnetic impulses, which are transmitted into the building material using a dipole antenna [21], [22]. Using *thermographic analysis*, thermal radiation is collected by a camera that is sensitive to infrared radiation. Each material emits energy as electromagnetic radiation, which is characterised by thermal conductivity (the capacity of the material itself to transmit heat). Masonry, as a nonhomogeneous material, presents different components and each component emits different temperatures that are shown in the final image [19], [23].

The *characterisation of the existing mortar* is typically conducted in a laboratory and includes compression tests, thin section petrographic examination using optical microscopy (OM), scanning electron microscopy with X-ray microanalysis (SEM/EDS), and X-ray powder diffraction (XRPD), see [16], [24], [25]. Mortar samples must be collected and stored carefully, selecting representative areas for the whole structure and, in the case of heritage buildings, considering the cultural importance of the original construction. The quantity of samples must be consistent with the scope and the requirements of the test procedures but be sufficient to statistically represent the material. On site the measurement of the surface hardness can be evaluated by measuring the rebound of a hammer on the surface. This test is referred to as a pendulum hammer test [26].

The state of the *stone elements* is assessed firstly by on-site mapping of the extent and type of damage and the degradation and causes (presence of salts, frost or hygrothermal action). Stone samples are then analysed in a laboratory using thin section petrographic examination (optical microscopy, OM) and compressive tests [8], [14].

MDT tests include direct inspection, boroscopy, drill-energy (penetration test), pull-out and flat-jack tests that are used to provide qualitative information on materials and structures. *Direct inspection* consists of removing a plaster area of approximately 1 x 1 m, and where possible also removing a few stones, in order to survey the wall in detail. A small camera is inserted into a borehole (diameter 20-30 mm) during boroscopy *inspection*, allowing a detailed analysis of the wall cross-section, see Figure 3.

Figure 3: Execution of boroscopy test [www.expin.it].

With a *pull out test* the mortar joint resistance is determined by introducing a tapping screw and measuring the force required to pull it out. The quality of the mortar joints can also be evaluated using the *drill-energy test* (penetration test) which registers the energy absorbed by an instrumented drill (hole diameter 4 mm and depth 5 mm). The drill-energy test can also be used on stones, recording the stress released by drilling a hole in the surface. In this case, three electrical resistance strain gauges are placed in a circumference around the hole [17], [27].

A *Flat-jack test* is performed using a steel pad which is inflated (into the mortar layer) with oil until the slot is tightly filled, i.e. the original vertical load condition is restored and the relative strength can be reconstructed. The reliable determination of the equilibrium pressure is the fundamental requirement for the test. A single flat-jack (Figure 4) allows determination of the state of stress, the compressive strength and the elastic modulus of the masonry, while the double flat-jack test (Figure 5) permits the deformability and stress-strain behaviour, the stiffness degradation, and the effectiveness of grout injection to be identified. Recommended references are [28], [29].

Figure 4: Execution of single flat-jack test [www.expin.it]

Figure 5: Execution of double flat-jack test [www.expin.it].

Destructive tests allow investigation of the main mechanical characteristics of the materials and the structure and can be performed both in-situ and in the laboratory. Static tests on structural elements, such as simple compression tests, diagonal compression and shear compression experiments [30], [31], can be performed on-site and in the laboratory, while dynamic tests can only be executed in the laboratory using reproduced models and a shaking table [32], [33].

The *compression test* consists of apply a vertical (axial) stress state to a masonry panel [20], [34]. The stress can be monotonic or cyclic, where in the first case the load is applied up to collapse while in the second case the test involves several loading and unloading cycles before failure. The specimen deformations are recorded by displacement transducers. The compressive test allows the compressive strength, elastic modulus and Poisson's ratio, stiffness degradation, tensile stress and deformation capability in compression to be determined. The laboratory experiments are carried out using a test frame and a hydraulic jack acting in either displacement or load control mode, see Figure 6. During in-situ testing a panel is isolated from the whole structure and a distribution beam is installed to diffuse the load applied using actuators.

Often a *cyclic compression test* constitutes the preliminary phase of a shear compression test. In this case the specimen remains in the elastic range and the elastic properties are studied prior to applying the lateral load.

With the *shear-compression test* the specimen is subjected to a constant vertical (axial) load, simulating the existing pre-compression acting on the building. With this set-up a monotonic or cyclic lateral displacement is applied with step-wise increasing displacement amplitudes, see Figure 7. The aim of the test is to determine the shear and tensile strength, shear modulus, ultimate tensile stress and the evaluation of ductility, energy absorption capacity, stiffness degradation and effectiveness of grout injection. The test can be performed either in the laboratory on extracted samples or on-site, depending upon the portability of testing apparatus.

Figure 6: Compression test and failure mode [30]

Figure 7: Shear-compression test and failure mode [30]**.**

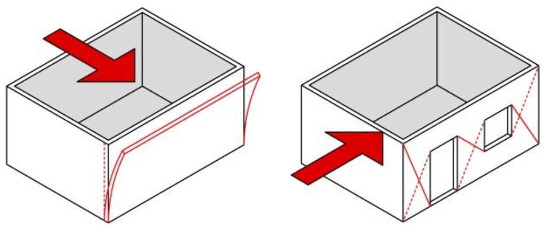
When the *diagonal compression test* [35] is undertaken on-site the specimen is isolated from the remaining structure and is compressed along its diagonal, see Figure 8. When the test is executed in a laboratory, the extracted or replicated specimen can either be rotated by 45 degrees or kept horizontal. The diagonal compression test allows the shear strength and shear modulus, the tensile strength, stiffness degradation, and the effectiveness of a retrofit solution such as grout injection to be evaluated.

Figure 8: In-situ and laboratory diagonal compressive test undertaken using two different set-up configurations [Source: University of Padova].

EARTHQUAKE BEHAVIOUR OF EXISTING URM BUILDINGS

URM load-bearing buildings were commonly constructed following traditional techniques that did not consider stability and safety for earthquake-induced loads. The general conditions which allow the application of common equivalent static procedures, such as "box-like behaviour" and elasto-plastic behaviour, are typically not satisfied for existing URM buildings. A structure presents a "box-like behaviour" when it contains sound wall-to-wall and wall-to-diaphragm connections, and an adequate horizontal diaphragm stiffness. In addition, the response of unreinforced stone masonry when subjected to earthquake-induced loads is not exclusively governed by the mechanical properties of the constituent materials, but also by the geometric and physical characteristics that permit monolithic behaviour of each structural component. Construction type, quality, and state of preservation of the masonry [1], [36], play a fundamental role in determining the capacity of a URM building to sustain seismic actions.

The ultimate capacity of URM buildings depends on the stability of its macro-elements when a kinematic mechanism is activated [11], [37]. Macro-elements are defined by single or combined structural components (walls and diaphragms), considering their connections and restraints, construction deficiencies and the characteristics of the constituent materials. According to Giuffrè's definition [37], damage to masonry buildings can be interpreted on the basis of two fundamental failure mechanisms. The out-of-plane failure mode (also called the first damage mode) is produced by seismic actions oriented perpendicular to the wall that cause overturning of the entire wall or a significant portion of it (Figure 9a). The out-of-plane mechanism represents the highest vulnerability for the structure. Once this mechanism is precluded the seismic actions are transmitted to the in-plane lateral load resisting walls. The in-plane failure mode (called the second damage mode) is caused by forces acting parallel to the wall and is usually marked by inclined cracks associated with shear forces that often result in an "X" pattern and seldom result in total collapse (Figure 9b). Depending on the quality of masonry units and mortar, diagonally oriented cracks may either follow the bed-joints and head-joints or pass through the units. When the vertical compressive stress is low and the quality of the mortar is poor, seismic forces may cause a sliding failure along one of the bed-joints. Sliding shear failure usually takes place in the upper parts of a URM building below rigid roof structures, where the compressive stresses are low and the response accelerations are high [38]. Once the macro-elements are identified, a detailed static linear or non-linear analysis considering all possible failure mechanisms is required to assess the building. As a second step a non-linear static (push-over) or linear dynamic analysis of the global seismic response of the building is recommended [11], [16].



(a) Out-of-plane (b) In plane

Figure 9: Kinematic mechanisms applied to masonry buildings.

A catalogue of all the possible failure mechanisms in URM buildings is presented in the Italian damage survey forms for cultural heritage, prepared for churches (A-DC model) and palaces (B-DP model) [39] and is reported in [40] with a summary presented in Annexes 3 to 12.

SEISMIC IMPROVEMENT INTERVENTIONS

Interventions in heritage buildings, involving refurbishment, repair or seismic improvement, should be subjected to a series of criteria to ascertain the efficiency of the solution together with its compliance with conservation principles. Recognising that conventional techniques and legal codes are oriented to the design of new buildings and may be difficult to apply, or inapplicable, to heritage buildings, the ICOMOS Committee issued the recommendations [41] summarised below.

Respect for structural authenticity: the original features of the structure, including materials, geometry and structural behaviour should be preserved.

Minimal intervention and non-intrusiveness: Among all the possible solutions providing the required level of safety, the intervention causing minimal alteration and minimal intrusiveness on the original structure should be preferred. The impact of an intervention on the building must be investigated and quantified, in terms of loss or alteration of the original material and structural features.

Compatibility: The ICOMOS/ISCARSAH recommendations [41] 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 (related to chemical, physical, mechanical, thermal and rheological phenomena). 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 demonstration must include long-term effects, so that undesirable side effects are avoided". For example, the use of Portland cements may free soluble salts which may experience expansive crystallization (efflorescence) causing detachments due to chemical incompatibility. The shrinkage or thermal deformation of Portland cement or concrete may cause cracks to stone or brick masonry due to rheological or thermal incompatibly. A mass of very stiff repair material inserted within the existing material may cause the latter to crack or crush due to additional loading or mechanical incompatibility. Useful information regarding the characterisation of the old mortar, the identification of the factors that influence its degradation, and the selection of the repair mixture are available in [24], [42], [43].

Durability: The selected repair materials or mechanical devices used must be sufficiently durable. The overall safety of the structure and the durability of the original parts can be compromised by the decay of the new intervention material.

Non-intrusiveness and non-obtrusiveness: Among all the possible alternatives, preference should be given to the least invasive one. Regarding obtrusiveness, the Venice Charter [44] 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."

Removability: Any intervention adopted should be reversible, or at least removable, so that the intervention can be replaced with a more suitable solution if new technology is acquired.

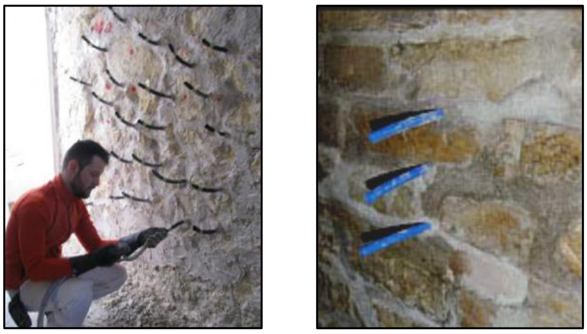
Monitorability and controllability. Repair or strengthening measures whose performance and effect on the building are impossible to control should not be allowed. In order to carry out such control, monitoring should be applied during and after the execution of the intervention [16].

The following sections contain a short description of the common improvement interventions for stone unreinforced masonry buildings. Schematics and details are summarised in Annex 13.

Global Interventions of URM Walls

The choice of the most appropriate seismic improvement technique for URM walls is connected to the type and quality of masonry, the type, extent and causes of existing damage, and the possible preservation requirements [12]. The wall cross-section is commonly retrofitted using grout injection and the installation of elements that link the exterior leaves to imitate the function of header units, while on the wall-surface the technique of bed joint repointing and the application of fibre-reinforced polymers (FRP) or textile reinforced mortars (TRM) are used to increase the strength.

Grout injection consists of injecting binder mixtures into the masonry in order to fill voids and cracks, enhance the masonry homogeneity, and improve the bond between leaves and the inner core, see Figure 10. The technique is appropriate for stone masonry, and in particular for multi-leaf walls that have not less than 10% of the entire element cross-section as internal voids. Grouts are made of binder, water and possible additives such as superplasticizer (maximum suggested 0.25%, [45]). Inorganic binders, such as hydraulic-lime based or ternary grouts, are suggested by previous studies, [31], [46], [47], [48], [49], considering compatibility and durability issues. The characteristics required for the injection mixture are injectability (including penetrability and fluidity), stability, low hydration heat, hygroscopic properties, adhesion, salt resistance, and chemical, physical and mechanical compatibility with existing mortars and stone units. Some advice about the chemical composition, the mechanical characteristics and the rheological aspects were refined and calibrated by Valluzzi et al. [45]. Fluidity, segregation and injectability of the mixture can be evaluated using the tests described in [45], [50], [51], and [52] respectively. Recommendations on the application of grout injection are provided in [53].



a) Injection of the inner core b) Crack injection

Figure 10: Grout injection [Source: University of Padova].

The *insertion of tie-rods* in the cross-section of multi-leaf masonry walls replicates the function of header units and limits out-of-plane deformation of the wall. Ties are simple steel bars with improved adherence that are fixed by bending or injection [32], [54], and [55]. Threaded bars with a bolted head can also be used and it is suggested to warm and tighten the bar upon insertion in order to place the bar in tension.

Bed joint repointing consists of removing a few centimetres of bed joint mortar and replacing with better quality mortar, in order to improve the masonry mechanical characteristics, particularly in terms of deformation. *Bed joint structural repointing* also includes the placing of reinforcement into the bed joints, such as small bars, strips or plates made of steel or carbon [56], [57], [58]. This intervention is suitable for masonry having a regular textured surface, such as brick masonry, and leads to a reduction of the transverse expansion and the formation of cracks and to repair existing cracks. An interesting case study is available in [59].

Application of *fibre-reinforced polymers* (FRP) involves the use of different fibres such as carbon, glass, steel, or hemp that are impregnated in a polymeric matrix [60], [61], [62]. Recently several studies have also focused on the use of an inorganic matrix known as Textile-Reinforced Mortar (TRM), that includes Steel-Reinforced Grout (SRG) and Basalt Textile-Reinforced Mortar (BTRM) [63], see Figure 11. For both FRP and TRM applications, the fibres (grids, strips or cords) are immersed into the matrix and bonded directly to the masonry surface. Useful standards, recommendations and examples on the application of FRP are [64], [65], [66]. This intervention can be considered an extension of *jacketing*, typically using hand applied RC plaster. RC jacketing consists of the use of a reinforcing steel net placed on both faces of a wall, connected with frequently spaced steel connectors and covered with cement mortar based rendering [47], [67]. This technique is particularly invasive and considerably increases the mass and stiffness of the wall. In addition, the presence of the RC jacketing often produces an increase in moisture within the wall, generating a barrier that hinders evaporation. Recently it was observed that RC jacketing has insufficient transverse ties to provide confining action and low durability due to insufficient steel mesh cover, resulting in steel corrosion and separation of the jacket [68], [69], see Figure 12.

a) Application of FRP on masonry vaults b) Application of TRM on masonry wall

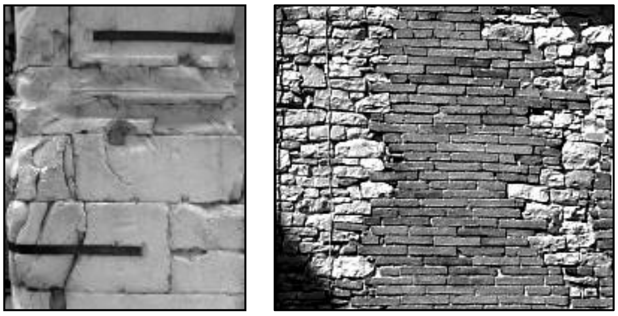
Figure 11: Example of FRP and TRM application [Source: University of Padova].

a) Application b) Loss of plaster and bond following an earthquake

Figure 12: Jacketing using RC plaster [68].

Local Repair of URM Walls

Local repairs include grout injection of cracks, stitching and anchoring, dismantling and reconstruction of small portions of masonry, and local application of FRP. Cracks can be repaired and stitched by means of *grout injection of the cracks* (see Figure 10b, [54]) or by providing *anchors* (see Figure 13a, [68]) respectively, i.e. joining built-in elements inserted at staggered points. The effectiveness of crack stitching in terms of restoring structural continuity or uniform load distribution on a wall is related to the diffusion of the intervention, because force transfer is limited to the points where the stitches are introduced. Reinforced injections by means of insertion of numerous and inclined steel bars drilled and grouted in the wall cross-section is a technique that results in extensive damage to the masonry during execution, and therefore should be avoided [67].



a) Crack stitching and anchoring   b) Dismantling and reconstruction using clay bricks

Figure 13: Local repairing [68].

Local dismantling and rebuilding (referred to as “scuci-cuci” in Italian, which means “unstitch-stitch”) is a technique that consist of the substitution of damaged elements in order to restore the wall continuity, Figure 13b [54]. Both new and original units can be used, preferably compatible and similar in terms of form, dimensions, stiffness and resistance. Adequate connection has to be provided between original and new units, and headers have to be used to connect the leaves.

Global Stability of Structure

Confinement of columns and pillars that suffer from cracks caused by compression is made using metal elements, see Figure 14a, or FRP strips. Polyester bands are also used in the case of emergency interventions. The confinement allows an increase in the compression strength capacity and stability of structural elements and improvement of the global behaviour of the structure.

Masonry enlargement has traditionally been used to increase the load bearing capacity of walls and vaults or to improve wall stability. Enlargement refers to the addition of new material (such as an additional leaf of masonry) to an existing element, with adequate connection, in order to increase its cross-sectional geometry and hence mechanical capacity.

Buttresses resist lateral forces and lateral deformations, essentially due to their self-weight. Large elements or flying buttresses are added laterally to the structure to improve out-of-plane stability, see Figure 14b.

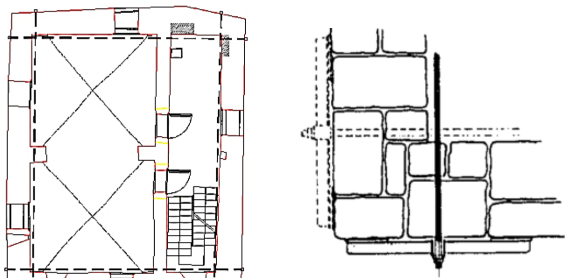
a) Addition of column confinement   b) Presence of buttresses

Figure 14: Stability improvements.

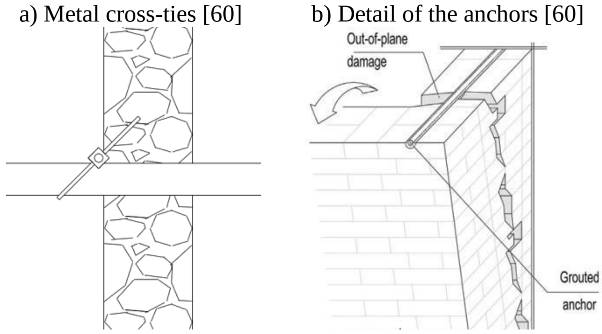
Wall-to-Wall and Wall-to-Floor Connections

Cross-ties are widely used in traditional buildings as wall-to-wall and wall-to-floor connections, aiming to improve the integrity of the structure. The ties are usually positioned at the roof/floor level and the dimensions of the anchors are related to the tie tension and masonry quality [54], [60], see Figure 15a,b. When the external anchor is a simple steel bar, it should be positioned at 45 degrees with the higher arm against the orthogonal wall in order to distribute the load to both the diaphragm and the wall, see Figure 15c. Low cost, easy installation, and easy maintenance and repair are distinct advantages of this technique. Other similar interventions are the use of anchors grouted into the masonry, see Figure 15d [70], and the use of confining rings or tie-beams (reinforced masonry or steel), recognising that corners require special care due to the stress distribution [60], see Figure 15e,f.

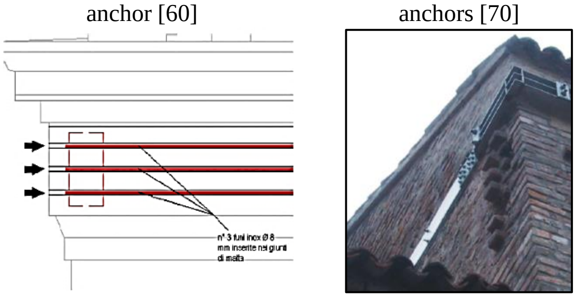
Metal or wooden anchors can be applied to beams to improve the connection between walls and floor, see Figure 16a. The anchors increase wall stability preventing failures due to sliding and hammering of the beam and acting as a cross-tie to reduce the possibility of developing a wall overturning mechanism [67], see Annex 4.



a) Metal cross-ties [60]   b) Detail of the anchors [60]



c) Correct positioning of the anchor [60]   d) Example of wall-to-wall anchors [70]



e) Confining with stainless steel cables [60]   f) Stainless steel tie-beam [60]

Figure 15: Wall-to-wall connections.

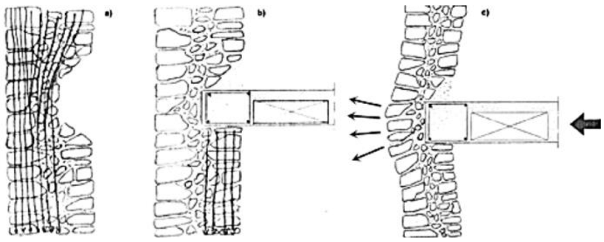
Over the last few decades seismic interventions applied to diaphragms mainly consisted of the *replacing of timber structures with RC or mixed clay/RC floors*. This intervention entailed the insertion of RC tie beams in the masonry wall cross-section after partial demolition of the masonry portion. Such intervention led to eccentricity and redistribution of the load in the existing masonry walls, causing out-of-plane failures due to pounding effects, as documented following recent earthquakes in Italy [60], [68], [69], see Figure 16b and Figure 17.

a) Insertion of metal anchors

b) Earthquake damage due to roof replacement using RC

Figure 16: Wall-to-floor connections [60].

The recommendations for *rehabilitation of existing timber floors* include the use of both traditional and modern materials and techniques. Some examples of diaphragm interventions are the application above a floor of single and double planking or plywood overlay, steel plates or bars, or externally bonded (EB) composite materials such as carbon fibre reinforced polymer (CFRP), steel reinforced polymer (SRP) or wooden diagonal natural fibres (hemp) [71], [72], [73], [74].



a) Effect of the eccentric loading and failure when subjected to vertical and horizontal loads

b) Out-of-plane failure due to pounding effect of the diaphragms

Figure 17: Eccentricity and redistribution of load in existing masonry walls due to RC beam positioning [75].

CONCLUSIONS

Recent earthquakes, and in particular the 2010/2011 Canterbury earthquakes, showed once again the high seismic hazard of stone unreinforced masonry buildings, reminding buildings owners and occupants of the level of risk associated with this construction. Because little guidance was available in New Zealand pertaining to the seismic assessment and improvement of stone URM buildings, a detailed survey was undertaken by the authors to identify the characteristics and location of the majority of existing stone load-bearing URM buildings in New Zealand [1] and useful information on international assessment and refurbishment techniques that were relevant to the New Zealand stone URM stock was compiled.

When assessing existing load-bearing URM buildings the information, construction details and design parameters required are often unknown. Moreover, stone masonry is a non-homogeneous material whose characterisation is of fundamental importance in order to identify the seismic performance of the building and the most compatible materials to be used during the strengthening intervention. An appropriate level of knowledge of the building is obtained by referring to the documentation in existence (original plans and reports, historical photos, information from council archives, literature) and planning a complete experimental campaign that is focused in particular on identification of the materials used and their mechanical parameters [19], [54], [76]. The professional engineer in charge will have to plan the experimental campaign on a case-by-case basis in relation to the available original documentation, the first complete inspection of the building, and the information required for the complete assessment. The more common tests for stone masonry buildings have been described, presenting their grade of invasiveness, the procedure and the type of results that can be obtained. Annex 1 provides a brief summary of the required on-site tests versus obtained parameters, while default probable material properties for stone masonry are presented in Annex 2.

Using the collected information, macro-elements are identified and all possible failure mechanisms are considered using a detailed static linear or non-linear analysis. As a second step a non-linear static (push over) or linear dynamic analysis of the global seismic response of the building is recommended [11], [16]. A catalogue of all the possible failure mechanisms in masonry buildings is presented in Annexes 3 to 12, referring to the Italian damage survey forms for cultural heritage [39].

Once information from investigations and assessment is collected, a suitable retrofit intervention can be designed and compatible materials can be selected. Some intervention techniques or the introduction of new remediation materials lead to significant modification of the original structural behaviour, which may contribute or directly cause structural failures, with many such examples having been observed during post-earthquake investigation [60], [67], [68]. Repair materials must be compatible with the original materials in hardness, density, porosity, permeability, elastic modulus, and moisture expansion. Strengthening systems must be compatible in terms of stiffness and deformability. Seismic improvement techniques include: (i) global or local interventions on stone URM walls, such as grout injection, insertion of tie-rods, bed joint structural repointing, FRP and TRM application, local dismantling and rebuilding; and (ii) enhancement of wall-to-wall and wall-to-floor connections, i.e. insertion of cross-ties, confining rings, tie-beams, anchors, and strengthening of timber floors. Traditional interventions have been discussed and references were provided focusing on the task of preserving cultural heritage against the effects of earthquakes. Schematics and details for seismic improvement interventions are summarised in Annex 13. Once the intervention is designed and applied, it is suggested that the performance of the intervention be monitored in order to validate the method adopted and to verify the behaviour of the structure.

ACKNOWLEDGMENTS

The authors acknowledge the NZ Natural Hazards Research Platform and the Italian project ProVaCI for their financial support, E. Cescatti, E. Garbin, M. Munari, P. Girardello and F. Casarin are thanked for their help in collecting references.

REFERENCES

1 Giaretton M, Dizhur D, da Porto F and Ingham JM (2014). “An inventory of unreinforced load-bearing stone masonry buildings in New Zealand”. *Bulletin of the New Zealand Society for Earthquake Engineering*, **47**(2): 57–74.

2 Dizhur D, Ingham J, Moon L, Griffith M, Schultz A, Senaldi I, Magenes G, Dickie J, Lissel S, Centeno J, Ventura C, Leite J and Lourenco P (2011). “Performance of masonry buildings and churches in the 22 February 2011 Christchurch Earthquake”. *Bulletin of the New Zealand Society for Earthquake Engineering*, **44**(4): 279-296.

3 Senaldi I, Magenes G and Ingham JM (2011). “Performance of unreinforced stone masonry buildings during the 2010/2011 Canterbury earthquake swarm and retrofit techniques for their seismic improvement”. *Journal of the Structural Engineering Society New Zealand*, **24**(2): 45–57.

4 da Porto F, Munari M, Prota A and Modena C (2013). “Analysis and repair of clustered buildings: Case study of a block in the historic city centre of L’Aquila (Central Italy)”. *Construction and Building Materials*, **38**: 1221–1237.

5 Hayward BW (1987). "*Granite and Marble: A Guide to Building Stones in New Zealand*". Geological Society of New Zealand.

6 Lowndes WS (1994). "*Stone Masonry : Prepared Especially for Home Study*". 3rd ed. International Textbook Company, Pennsylvania, USA.

7 Marshall P (1929). "*Building-Stones of New Zealand*". Wellington: NZ Department of Scientific and Industrial Research.

8 Giaretton M, Dizhur D, da Porto F and Ingham J (2015). “Constituent material properties of New Zealand unreinforced stone masonry buildings”. *Journal of Building Engineering*, **4**: 75–85.

9 DPCM 09-02-2011 (2011). "*Guidelines for Evaluation and Mitigation of Seismic Risk of Cultural Heritage*". Ministry for Cultural Heritage and Activities, Italy. G.U. n.47 del 26/02/2011 - Suppl. ord. n.54.

10 Binda L, Cardani G, Saisi A, Valluzzi MR, Munari M and Modena C (2007). “Multilevel approach to the vulnerability analysis of historic buildings in seismic areas - Part 1: Detection of parameters for vulnerability analysis through on site and laboratory investigations”. *Restoration of Buildings and Monuments*, **13**(6): 413–426.

11 Valluzzi MR, Munari M, Modena C, Binda L, Cardani G and Saisi A (2007). “Multilevel approach to the vulnerability analysis of historic buildings in seismic areas - Part 2: Analytical interpretation of mechanisms for vulnerability analysis and structural improvement". *Restoration of Buildings and Monuments*, **13**(6): 427–442.

12 Lorenco PB (2015). “Reducing seismic vulnerability: Retrofitting historic buildings”. *Conservation Perspectives. The GCI Newsletter*, **30**(1): 16–18.

13 NorMaL 1/88 (1990). "*Raccomandazioni NorMaL-1/88. Alterazioni macroscopiche dei materiali lapidei: lessico*". Consiglio Nazionale delle Ricerche - Istituto Centrale del Restauro (CNR_ICR), 1–6 [in Italian].

14 Bowman I (1992). “*Historic Stone Structures*” in *Conservation Bulletin 3*. New Zealand Historic Places Trust.

15 NZSEE (2015). "*Assessment and Improvement of the Structural Performance of Buildings in Earthquakes - Section 10 Revision. Seismic Assessment of Unreinforced Masonry Buildings*". New Zealand Society for Earthquake Engineering, Wellington, New Zealand.

16 Niker Project (2010). “*WP 3.3: Critical Review of Methodologies and Tools for Assessment of Failure Mechanisms and Interventions*”. New Integrated Knowledge based approaches to the protection of cultural heritage from Earthquake induced Risk. Project founded within the 7th EU Framework Programme (2007-2013) - n.244123.

17 OnSiteForMasonry Project (2005). “*WP 11: On-Site Investigation Techniques for the Structural Evaluation of Historic Masonry Buildings*”. Project founded within the 5th EU Framework Programme (1998-2002) - EVK4-2001-00091.

18 RILEM TC 127-MS (1996). “In-situ and non-destructive test proposed test method. MS.D.1 Measurement of mechanical pulse velocity for masonry”. *Materials and Structures*, **29**: 463–466.

19 Binda L, Saisi A and Tiraboschi C (2000). “Investigation procedures for the diagnosis of historic masonries”. *Construction and Building Materials*, **14**: 199–233.

20 Silva BLQ, Dalla Benetta M, da Porto F and Valluzzi MR (2013). “Compression and sonic tests to assess effectiveness of grout injection on three-leaf stone masonry walls”. *International Journal of Architectural Heritage*, **8**: 408–435.

21 RILEM TC 127-MS (2001). “In-situ and non-destructive test proposed test method. MS.D.3: Radar investigation of masonry”. *Materials and Structures*, **34**: 134–143.

22 Binda L, Lenzi G and Saisi A (1998). “NDE of masonry structures: use of radar tests for the characterisation of stone masonries”. *NDT & E International*, **31**(6): 411-419.

23 Tavukcuoglu A, Akevren S and Grinzato E (2010). “In situ examination of structural cracks at historic masonry structures by quantitative infrared thermography and ultrasonic testing”. *Journal of Modern Optics*: 1–11.

24 Groot C, Ashall G and Hughes J (2007). “*Characterisation of old mortars with respect to their repair*”. Report 28 by RILEM TC 167–COM, RILEM.

25 Secco M, Artioli G, Dalla Benetta M, da Porto F and Colletti C (2011). “Restoration of cultural heritage masonry structures damaged by the 2009 Abruzzo earthquake: materials and methods”. *97th Congresso Nazionale della Società Italiana di Fisica*, September 26–30, L'Aquilla, Italy.

26 RILEM TC 167-MS (1998). “In-situ and non-destructive test proposed test method. MS.D.2: Determination of masonry rebound hardness”. *Materials and Structures*, **31**: 375–377.

27 ASTM E837 (2013). "*Standard Test Method for Determining Residual Stresses by the Hole-Drilling Strain-Gage Method*". American Society for Testing and Materials, USA.

28 ASTM C1196 (2014). "*Standard Test Method for In Situ Compressive Stress within Solid Unit Masonry Estimated Using Flatjack Measurements*". American Society for Testing and Materials, USA.

29 RILEM TC 177-MDT (2004). “Masonry durability and on-site testing. MDT.D.4: In-situ stress tests based on the flat jack”. *Materials and Structures*, **37**: 491–496.

30 Niker Project (2011). “*WP 4.3: Optimization of Design for Vertical Elements. Technical Report with the Experimental Results*”. New Integrated Knowledge based Approaches to the Protection of Cultural Heritage from Earthquake Induced Risk. Project Founded within the 7th EU Framework Programme (2007-2013) - n.244123.

31 Silva B, Dalla Benetta M, da Porto F and Modena C (2014). “Experimental assessment of in-plane behaviour of three-leaf stone masonry walls”. *Construction and*

Building Materials, **53**: 149–161.

32 Valluzzi MR, Mazzon N, Garbin E and Modena C (2013). "Experimental characterization of out-of-plane seismic response of strengthened three-leaf stone masonry walls by shaking table tests". *XV Convegno Nazionale "L'Ingegneria Sismica in Italia" - ANIDIS*, 30 June- 4 July, Padova, Italy.

33 Mazzon N, Valluzzi MR, Giaretton M and Modena C (2013). "Dynamic modal identification of strengthened three-leaf stone masonry walls subjected to out-of-plane shaking table tests". *XV Convegno Nazionale "L'Ingegneria Sismica in Italia" - ANIDIS*, 30 June- 4 July, Padova, Italy.

34 UNI EN 1052-1 (2001). "*Methods of tests for masonry. Determination of compressive strength*". Ente Nazionale Italiano di Unificazione: **44**(0), Italy.

35 ASTM E519 (2002). "*Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages*". American Society for Testing and Materials, USA, 1–5.

36 Binda L and Penazzi D (2000). "*Classification of Masonry Cross Sections and of Typologies of Historic Buildings*". Technical Report - RILEM MMM Committee.

37 Giuffrè A (1993). "*Sicurezza e conservazione dei centri storici: il caso Ortigia*." Bari: Laterza [in Italian].

38 Tomaževič M, Lutman M and Petković L (1996). "Seismic behavior of masonry walls: Experimental simulation." *Journal of Structural Engineering*, **122**(9): 1040–1047.

39 DPCM (2006). "*Approvazione dei modelli per il rilevamento dei danni a seguito di eventi calamitosi, ai beni appartenenti al patrimonio culturale – scheda "Chiese" e scheda "Palazzi"*". Ministry for Cultural Heritage and Activities, Italy. G.U. n.55 del 07-03-2006 [in Italian].

40 Senaldi I, Magenes G and Ingham JM (2015). "Damage assessment of unreinforced stone masonry buildings after the 2010-2011 Canterbury earthquakes". *International Journal of Architectural Heritage*, **9**(5): 605–627.

41 ICOMOS/ISCARSAH Committee (2005). "*Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage*". International Scientific Committee for Analysis and Restoration of Structures of Architectural Heritage, Approved in the Committee Meeting in Barcelona, Spain, 1–37.

42 RILEM TC 203-RHM (2012). "Repair mortars for historic masonry". *Materials and Structures*, **45**(9): 1287–1294.

43 van Hees RPJ, Binda L, Papayianni I and Toumbakari E (2007). "Damage analysis as a step towards compatible repair mortars". *Report 28 by RILEM TC 167–COM*, 107–152.

44 Venice Charter (1964). "The Venice Charter for the Conservation and Restoration of Monuments and Sites". *Second International Congress of Architects and Technicians of Historic Monuments*, 25-31 May, Venice, Italy.

45 Valluzzi MR, da Porto F and Modena C (2003). "Admixtures requirements for the injection of stone masonry walls". *International Conference on Performance of Construction Materials in the New Millennium - ICPCM*, pp. 393–402, 18-20 February, Cairo, Egypt.

46 Kalagri A, Miltiadou-Fezans A and Vintzileou E (2010). "Design and evaluation of hydraulic lime grouts for the strengthening of stone masonry historic structures". *Materials and Structures*, **43**(8): 1135–1146.

47 Modena C and Bettio C (1994). "Experimental characterisation and modelling of injected and jacketed masonry walls". *Italian-French Symposyum Strengthening and Repair of Structures in Seismic Area*, pp. 273–282, 17-19 October, Nice, France.

48 Valluzzi MR, da Porto F and Modena C (2004). "Behavior and modeling of strengthened three-leaf stone masonry walls". *Materials and Structures*, **37**(267): 184–192.

49 Vintzileou E and Miltiadou-Fezans A (2008). "Mechanical properties of three-leaf stone masonry grouted with ternary or hydraulic lime-based grouts". *Engineering Structures*, **30**(8): 2265–2276.

50 ASTM C939 (2010). "*Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)*". American Society for Testing and Materials, 9–11.

51 ASTM C940 (2010). "*Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory*". American Society for Testing and Materials, 1–3.

52 ASTM C943 (2010). "*Standard Practice for Making Test Cylinders and Prisms for Determining Strength and Density of Preplaced-Aggregate Concrete in the Laboratory*". American Society for Testing and Materials, 8–11.

53 Niker Project (2012). "*WP 10.1: Guidelines for Design and Execution of Optimum Interventions for End-Users*". New Integrated Knowledge based approaches to the protection of cultural heritage from Earthquake induced Risk. Project founded within the 7th EU Framework Programme (2007-2013) - n.244123.

54 Modena C, Casarin F, Valluzzi MR, da Porto F and Munari M (2007). "Codes of practice for architectural heritage in seismic zones". *5th International Conference on Structural Analysis of Historical Constructions (SAHC)*, 6-8 November, New Delhi, India.

55 Elmenshawi A, Sorour M, Mufti A, Jaeger LG and Shrive N (2010). "Damping mechanisms and damping ratios in vibrating unreinforced stone masonry". *Engineering Structures*, **32**(10): 3269–3278.

56 Valluzzi MR, Binda L and Modena C (2005). "Mechanical behaviour of historic masonry structures strengthened by bed joints structural repointing". *Construction and Building Materials*, **19**: 63–73.

57 Valluzzi MR, Tinazzi D and Modena C (2005). "Strengthening of masonry structures under compressive loads by FRP strips: local-global mechanical behaviour". *Science and Engineering of Composites Materials*, **12**: 203–212.

58 Modena C and Valluzzi MR (2008). "*Repair techniques and long-term damage of massive structures*" in *Learning from Failure: Long-term Behaviour of Heavy Masonry Structures*, L. Binda, Ed. WIT press, 175–204.

59 Modena C, Valluzzi MR, Tongini Folli R and Binda L (2002). "Design choices and intervention techniques for repairing and strengthening of the Monza cathedral bell-tower". *Construction and Building Materials*, **16**(7): 385–395.

60 Modena C, Valluzzi MR, da Porto F and Casarin F (2011). "Structural aspects of the conservation of historic masonry constructions in seismic areas: Remedial measures and emergency actions". *International Journal of Architectural Heritage*, **5**: 539–558.

61 Valluzzi MR, da Porto F, Garbin E and Panizza M (2014). "Out-of-plane behaviour of infill masonry panels strengthened with composite materials". *Materials and Structures*, **47**(12): 2131–2145.

62 Calderini C, Abbati SD, Cotič P, Kržan M and Bosiljkov V (2015). "In-plane shear tests on masonry panels with

plaster: correlation of structural damage and damage on artistic assets". *Bulletin of Earthquake Engineering*, **13**(1): 237–256.

63 Garmendia L, Marcos I, Garbin E and Valluzzi MR (2014). "Strengthening of masonry arches with Textile-Reinforced mortar: experimental behaviour and analytical approaches". *Materials and Structures*, **47**(12): 2067-2080.

64 CNR-DT 200 R1 (2013). "*Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures*". National Research Council, Italy.

65 ACI 440M (2004). "*Guide Draft-1 Guide for the Design and Construction of Externally Bonded FRP System for Strengthening Unreinforced Masonry Structures*". ACI Committee 440. American Concrete Institute, Farmington Hills, USA.

66 Valluzzi MR, Modena C and de Felice G (2014). "Current practice and open issues in strengthening historical buildings with composites". *Materials and Structures*, **47**(12): 1971–1985.

67 Modena C (1994). "Repair and upgrading techniques of unreinforced masonry structures utilized after the Friuli and Campania/Basilicata Earthquakes". *Earthquake Spectra*, **10**(1): 171–185.

68 Binda L, Cardani G, Penazzi D and Saisi A (2003). "Performance of some repair and strengthening techniques applied to historical stone masonries is seismic areas". *International Conference on Performance of Construction Materials in the New Millennium - ICPCM*, pp. 1195–1204, 18-20 February, Cairo, Egypt.

69 Penazzi D, Valluzzi MR, Saisi A, Binda L and Modena C (2001). "Repair and strengthening of historic masonry buildings in seismic areas". *International Congress on "More than two thousand years in the history of architecture safeguarding the structure of our architectural heritage"* Vol. **2**, Bethlehem, Palestine.

70 Paganoni S and D'Ayala D (2014). "Testing and design procedure for corner connections of masonry heritage buildings strengthened by metallic grouted anchors". *Engineering Structures*, **70**: 278–293.

71 Parisi MA and Piazza M (2007). "Restoration and Strengthening of Timber Structures: Principles, Criteria, and Examples". *Practice Periodical on Structural Design and Construction*, **12**(4): 177–185.

72 Riggio M, Tomasi R and Piazza M (2014). "Refurbishment of a traditional timber floor with a reversible technique: Importance of the investigation campaign for design and control of the intervention". *International Journal of Architectural Heritage*, **8**(1): 74–93.

73 Valluzzi MR, Garbin E and Modena C (2007). "Flexural strengthening of timber beams by traditional and innovative techniques". *Journal of Building Appraisal*, **3**(2): 125–143.

74 Valluzzi MR, Garbin E, dalla Benetta M and Modena C (2013). "Experimental Characterization of Timber Floors Strengthened by in-Plane Improvement Techniques". *Advanced Materials Research*, **778**: 682–689.

75 Borri A and De Maria A (2004). "Alcune considerazioni in materia di analisi e di interventi sugli edifici in muratura in zona sismica". *XI Convegno Nazionale "L'Ingegneria Sismica in Italia" - ANIDIS*, 25-29 January, Genoa, Italy [in Italian].

76 Valluzzi MR (2006). "On the vulnerability of historical masonry structures: analysis and mitigation". *Materials and Structures*, **40**(7): 723–743.

77 DM 14/01/2008 (2008). "*Norme Tecniche per le Costruzioni - NTC 2008*". Ministero delle Infrastrutture, Italy [in Italian].

78 Circ. 617/2009 (2009). "*Istruzioni per l'applicazione delle "Norme Tecniche per le Costruzioni" di cui al D.M. 14 Gennaio 2008*". Consiglio Superiore dei Lavori Pubblici, Italy [in Italian].

79 Valluzzi MR, Cardani G, Binda L and Modena C (2004). "Seismic vulnerability methods for masonry buildings in historical centres: validation and application for prediction analyses and intervention proposals". *13th World Conference on Earthquake Engineering*, 1-6 August, Vancouver, Canada.

ANNEX 1. ON-SITE AND LABORATORY INVESTIGATIONS OF STONE MASONRY STRUCTURES

This annex provides a brief summary of the suggested investigations for stone masonry buildings versus the required information and parameters. Tests are clustered in NDT, MDT and DT.

	NON-DESTRUCTIVE (NDT)					MINOR-DESTRUCTIVE (MDT)							DESTRUCTIVE (DT)		
INFORMATION REQUIRED \ INVESTIGATIONS	SONIC PULSE VELOCITY	RADAR TEST	TOMOGRAPHY	THERMOGRAPHIC ANALYSIS	PENDULUM HAMMER	DIRECT INSPECTION	CORE DRILLING	BOROSCOPY INSPECTION	DRILL-ENERGY TEST	PULL-OUT TEST	SINGLE FLAT-JACK	DOUBLE FLAT-JACK	COMPRESSION	DIAGONAL COMPRESSION	SHEAR COMPRESSION
Quality of the masonry	X					X	X								
Presence of voids or chimney flues	X	X		X			X								
Detection of water and heating systems		X		X											
Presence of structural irregularities/different materials	X	X	X	X		X		X							
Presence and thickness of multiple leaves	X	X				X	X	X							
Presence and level of moisture and salt		X		X											
Qualify the state of conservation or damage		X		X		X									
Quality and resistance of mortar					X				X	X					
Compressive strength and elastic modulus											X	X	X		
Deformability and stress/strain behaviour												X	X		
Stiffness of the inner core in multi-leaf walls															
Poisson’s ratio													X		
Shear strength and shear modulus														X	X
Tensile strength														X	X
Ductility															X
Energy absorption capacity															X
Stiffness degradation and deterioration												X	X	X	X
Effectiveness of the grout injection	X	X	X									X	X	X	X

ANNEX 2. STONE MASONRY MATERIAL PROPERTIES

This annex provides default probable material properties for stone masonry. These values can be used for assessment of stone URM buildings in the absence of a comprehensive testing programme. However, to arrive at any reliable judgement, some on-site testing as suggested in Annex 1 is required.

Table 1 shows the reference values for the masonry properties (compressive strength, shear stress, elastic and shear moduli) in the case of low masonry quality (poor mortar strength, thick mortar joints, absence of connections) which is the most commonly encountered condition in vintage stone URM buildings, and the specific masonry weight depending on the stone masonry typology. The method for recognizing the stone masonry typology (surface texture and cross-section) is described in [15] and [1]. If in doubt, it is recommended that the lowest listed value be used. In the case of masonry with better characteristics, such as good quality of the mortar (mechanical properties, conservation state, etc), adequate mortar joint thickness, presence of some courses of bricks to regulate the stone masonry and of transverse connections, or consolidated walls with grout injection or reinforced plaster, the design material parameters shall be corrected by the multipliers (Correction Factors) listed in Table 2. A correction factor is also provided for stone masonry with a wide and/or poor internal core.

The material data shown below are taken from the Italian Code, [77] and [78], assumed that no site testing has been undertaken, and hence it is strongly recommended that an appropriate testing campaign be performed on site in order to confirm the data presented below.

Table 1: Reference values for different material parameters (min and max) and specific weight depending on the stone masonry typology. Based on Table C8A.2.1 in [78].

	Stone masonry typology (see [15]**)**	f_m **[kPa]** min max	τ_0 **[kPa]** min max	E_m **[MPa]** min max	G_m **[MPa]** min max	W_m **[kN/m^3]**
A	Rubble stone masonry (pebbles, erratic and irregular stone) and field-stone walls	1000 1800	20 32	690 1050	230 350	19
B	Coursed rubble stone masonry with facing walls of limited thickness and infill core	2000 3000	35 51	1020 1440	340 480	20
C	Broken ashlar stone masonry with good bonding	2600 3800	56 74	1500 1980	500 660	21
D	Soft stone masonry (tuff, Limestone, etc.)	1400 2400	28 42	900 1260	300 420	16
E	Coursed or random-cursed or blocked cursed ashlar (squared stone blocks)	6000 8000	90 120	2400 3200	780 940	22

Where: f_m = Masonry compressive strength; τ_0 = Shear stress; E_m = Masonry modulus of elasticity; G_m = Masonry shear modulus; W_m = Masonry weight.

Table 2: Correction factors for material parameters in case of specific masonry characteristics. Based on Table C8A.2.2 in [78].

Stone masonry typology (see Table 1)	**Good mortar quality** *	**Joint thickness <10 mm** **	**Regular courses of bricks** ***	**Transverse connection** ***	**Wide and/or poor internal core** *	**Grout injection** *	**Reinforced plaster** *
A	1.5	-	1.3	1.5	0.9	2.0	2.5
B	1.4	1.2	1.2	1.5	0.8	1.7	2.0
C	1.3	-	1.1	1.3	0.8	1.5	1.5
D	1.5	1.5	-	1.5	0.9	1.7	2.0
E	1.2	1.2	-	1.2	0.7	1.2	1.2

* For both strength parameters (f_m, τ_0) and elastic moduli (E_m, G_m)

** For compressive strength (f_m) and elastic moduli (E_m, G_m); only for shear stress (τ_0) use 50% of the factors.

*** only for strength parameters (f_m, τ_0)

ANNEX 3. SHEAR MECHANISMS (IN-PLANE)

Brief descriptions of shear mechanisms are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Façade (C3); Side walls (C6); Transept (C11); Presbytery and Apse (C17); Chapels (C23); Piers (P5); Spandrels (P6); Internal walls (P7) – Referring to the Italian damage catalogue [39]

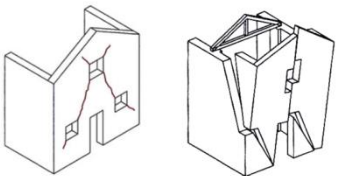
Description: Shear diagonal cracking (single or crossed), vertical or arched cracking due to the rotation of the involved elements. If only the spandrels are involved then the static behaviour of the building is not compromised. In-plane mechanisms typically do not lead to collapse.

Causes: (i) Presence of large openings; (ii) Local discontinuities (previously plugged gaps, etc); (iii) High slenderness of the wall (height-thickness ratio); (iv) Poor quality of the masonry; (v) Presence of heavy roof or rigid concrete stringcourse.

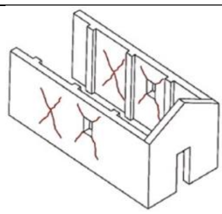
Warnings: (i) Diagonal cracks; (ii) Vertical or ached cracks; (iii) Failure in correspondence to the local discontinuities or existing damage.

Examples:

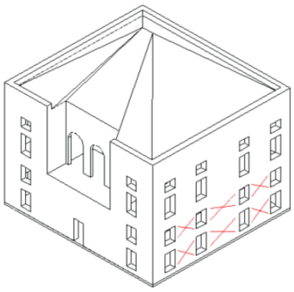
(C3)



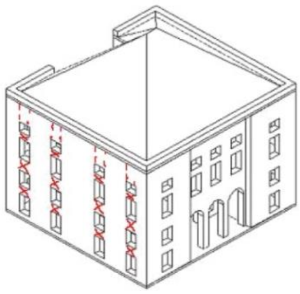
(C6)
(C11)
(C17)
(C23)



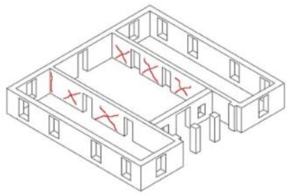
(P5)



(P6)

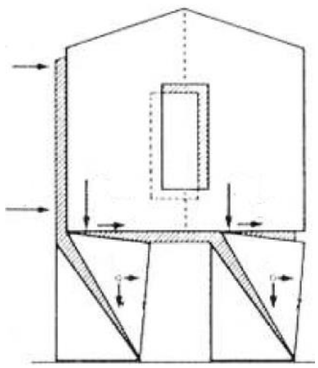


(P7)

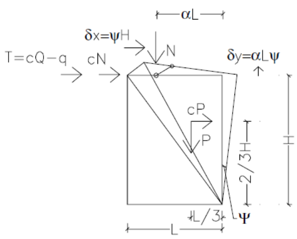


Force diagram:

[Source: www.reluis.it]



Bomporto, Emilia Romagna (Italy) – C3



Equilibrium condition:

$$-N\times\alpha L - P\times\frac{L}{3} - q\times H + c(N\times H + Q\times H + P\times\frac{2}{3}H) = 0$$

Collapse coefficient:

$$c = \frac{N\times\alpha L + P\times\frac{L}{3} + q\times H}{N\times H + Q\times H + P\times\frac{2}{3}H} = \frac{L}{H}\times\frac{N\alpha + \frac{P}{3} + q\frac{H}{L}}{N + 2\frac{P}{3} + Q}$$

[Source: Valluzzi et al [79]]

Sydenham, Christchurch (New Zealand) – C3

San Pietro di Coppito, L’Aquila (Italy) – C6

Christchurch (New Zealand) – P7

ANNEX 4. OVERTURNING MECHANISMS (OUT-OF-PLANE)

Brief descriptions of overturning mechanisms are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Façade (C1-P1); Damage at the top of the building (C2-P17); Transept (C10); Apse (C16); Chapels (C22); Corners (P4) – Referring to the Italian damage catalogue [39]

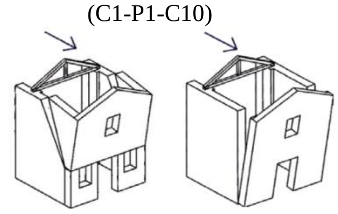
Description: Detachment or failure of the full or partial façade from the orthogonal walls. Delamination of masonry leaves. The mechanism could involve (i) one or more levels; (ii) different parts in relation to the position of the openings and the presence of discontinuity; (iii) the corner and part of the orthogonal wall.

Causes: (i) Poor wall-to-wall or wall-to-diaphragm connections; (ii) Poor quality of masonry; (iii) Insufficient connection between the leaves; (iv) Absence of stringcourse or cross-ties; (v) Presence of external horizontal forces (arches, vaults, heavy roof, annex constructions) without contrast; (vi) Presence of large openings near the corner.

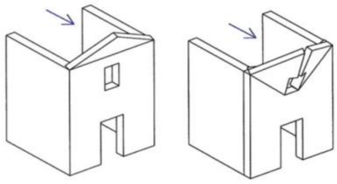
Warnings: (i) Cracks in correspondence to the wall-to-wall or wall-to-diaphragm connections; (ii) Out-of-plane of the wall; (iii) Pull out of the diaphragm or roof beams; (iv) Full or partial collapse of the façade.

Examples:

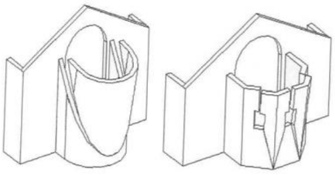
(C1-P1-C10)



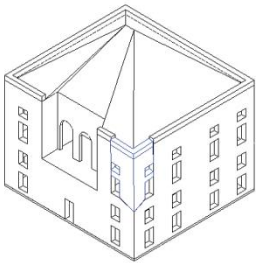
(C2-P17)



(C16-C22)



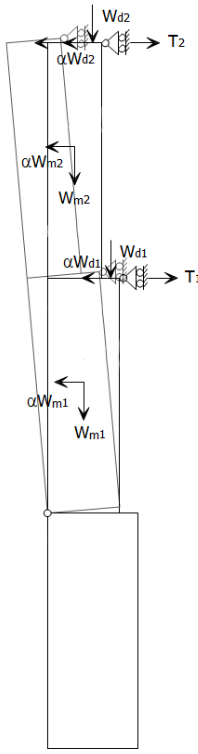
(P4)



L'Aquila (Italy) – C1

Christchurch (New Zealand) – C2

Force diagram:
[Source: www.reluis.it]



Where:
W_m = Masonry self-weight
W_d = Diaphragm self-weight
T = Action of the wall-to-diaphragm connection

Christchurch (New Zealand) – C1

Christchurch (New Zealand) – C2

Kathmandu (Nepal) – C1
Example of delamination of leaves

ANNEX 5. BENDING MECHANISMS (OUT-OF-PLANE)

Brief descriptions of bending mechanisms are presented below, including causes, warnings, schematic and photographic examples.

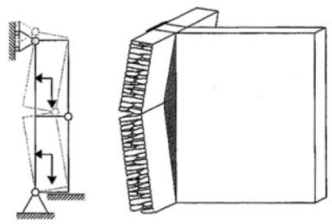
Mechanisms: Horizontal (P2) or Vertical (P3) – Referring to the Italian damage catalogue [39]

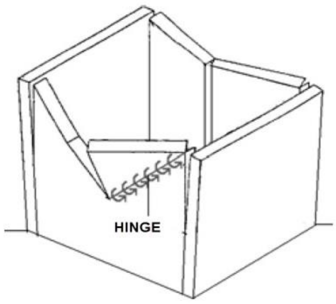
Description: The mechanism is activated by the formation of a hinge that subdivides the panel in two different blocks that mutually rotate leading to collapse. Horizontal bending causes a vertical instability of the walls, while vertical bending causes a flexural behaviour. The mechanism could involve: (i) the entire cross-section or only one leaf; (ii) different parts of the wall in relation to the position of the openings and the presence of discontinuity or concentrated loads; (iii) one or more levels.

Causes: (i) Poor wall-to-wall or wall-to-diaphragm connections; (ii) Poor quality of masonry; (iii) Insufficient connection between the leaves; (iv) Presence of external horizontal forces (arches, vaults, heavy roof or diaphragms) without contrast; (v) High slenderness (height/thickness ratio) of the wall; (vi) Absence of stringcourse or cross-ties.

Warnings: (i) Out-of-plane movement of the wall; (ii) Horizontal and vertical cracks; (iii) Pull out of the diaphragm's beams; (iv) Collapse of the wall or part of it.

Examples: (P2) (P3)





[Source: ReLUIS]

L'Aquila (Italy) – P2

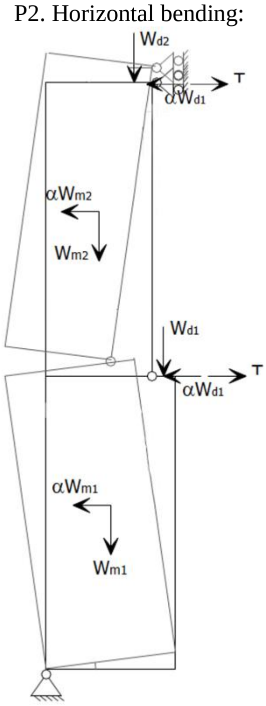
Emilia Romagna (Italy) – P3

L'Aquila (Italy) – P2

Force diagram:

[Source: www.reluis.it]

P2. Horizontal bending:



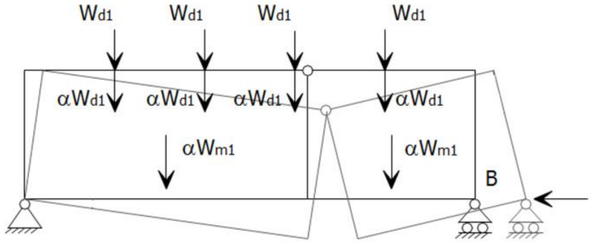
Where:

W_m = Masonry self-weight

W_d = Diaphragm self-weight

T = Action of the wall-to-diaphragm connection

P3. Vertical bending (plan view):



ANNEX 6. ROCKING MECHANISMS

Brief descriptions of rocking mechanisms are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Transverse response of the nave (C5); Longitudinal response of the colonnade (C7) – Referring to the Italian damage catalogue [39]

Description: Rocking behaviour of the lateral walls (transverse response of the nave, C5) or of the columns (longitudinal response of the colonnade, C7) due to the high slenderness (height/thickness ratio) of the walls and the absence of horizontal connections between the walls (diaphragms, cross-ties,...). These mechanisms are very common in the churches. They could involve (i) roof, diaphragm, vaults or ceilings; (ii) lateral walls or columns.

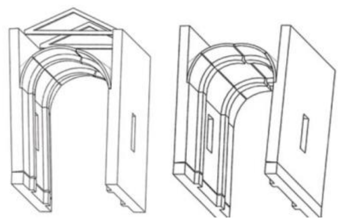
Causes: (i) High slenderness (height/thickness ratio) of the wall; (ii) Poor wall-to-wall or wall-to-diaphragm connections; (iii) Absence of stringcourse, cross-ties or buttresses; (iv) Presence of external horizontal forces (arches, vaults, heavy roof or diaphragms, annex constructions).

Warnings: Transverse response: (i) Cracks in the orthogonal arches involving vaults or ceilings nearby; (ii) Cracks or collapse of the vaults and arches in the side aisles; (iii) Horizontal cracks at the base of the walls; (iv) partial or total collapse of the roof, the vaults or the ceilings in the nave.

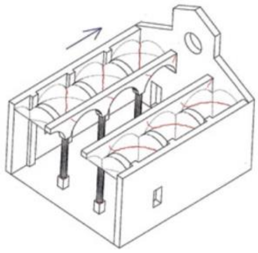
Longitudinal response: (i) Cracks in the longitudinal arches or beams; (ii) Compression cracks at the base of the columns; (iii) Shear cracks in the vaults of the side aisles.

Examples:

(C5)



(C7)



Emilia Romagna (Italy) – C5

Emilia Romagna (Italy) – C7

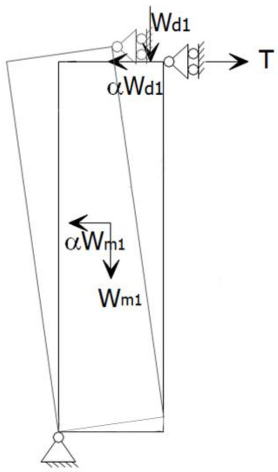
Emilia Romagna (Italy) – C5

Emilia Romagna (Italy) – C7

Force diagram:

[Source: www.reluis.it]

C5. Out-of-plane rocking:



Where:
W_m = Masonry self-weight
W_d = Diaphragm self-weight
T = Action of the wall-to-diaphragm connection

ANNEX 7. MECHANISMS IN ROOF OR DIAPHRAGM ELEMENTS

Brief descriptions of the possible mechanisms in roof or diaphragm elements are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Side walls of nave and aisle (C19); Transept (C20); Presbytery and apse (C21); Shear sliding at floor levels (P8); Hammering or roofing elements (P10); Local collapses of slab or vault (P11); Roof structural elements (P15); Roof covering elements (P16) – Referring to the Italian damage catalogue [39]

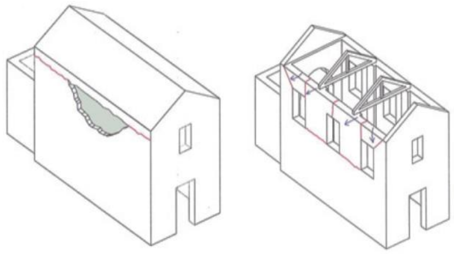
Description: Local or global failure of the top of the wall due to hammering or sliding behaviour of the roof or diaphragm elements. Damage of the roof or diaphragms beams. The mechanism could lead to collapse of both the wall and the roof.

Causes: (i) Presence of heavy roof or diaphragms or stringcourses; (ii) Absence of cross-ties or anchors; (iii) Poor connection beams and walls.

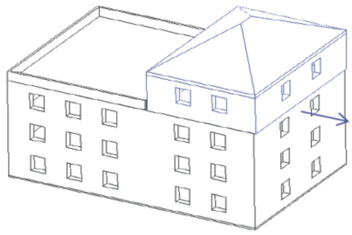
Warnings: (i) Cracks in correspondence of the head of the beams; (ii) Movements, sliding or hammering of beams; (iii) Lack of bond between stringcourse and masonry wall; (iv) Local collapse of slab, vaults or ceiling.

Examples:

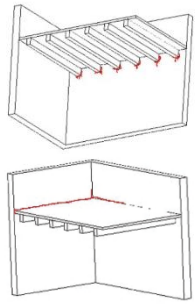
(C19-C20-C21)



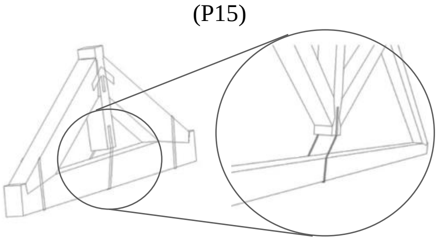
(P8)



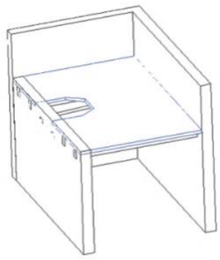
(P10)



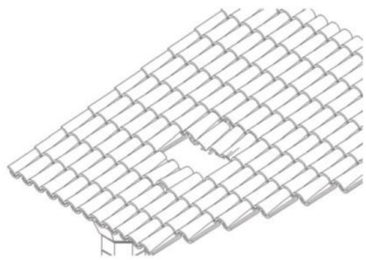
(P15)



(P11)



(P16)



L'Aquila (Italy) – C19

Emilia Romagna (Italy) – P10

Emilia Romagna (Italy) – C19

Kathmandu (Nepal) – P10

L'Aquila (Italy) – P16

ANNEX 8. MECHANISMS IN VAULTS AND ARCHES

Brief descriptions of the possible mechanisms in vaults and arches is presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Central nave (C8); Aisle (C9); Transept (C12); Triumphal arches (C13); Presbytery and Apse (C18); Chapels (C24); Arches and arcades (P9); Vaults (P12, P13) – Referring to the Italian damage catalogue [39]

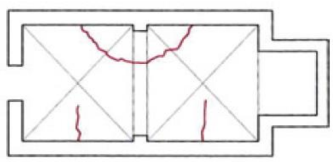
Description: Damage of vaults and arches due to rotation of their supports, floor deformations or concentrated loads. Failure mechanisms of the triumphal arch are caused by rocking, the transverse response of the nave or concentrated loads. Note: the triumphal arch is the main arch located between hall and transept or hall and presbytery and supporting dome and drum, it is relevant for the structural stability of the church.

Causes: (i) Presence of concentrated load; (ii) Thickness of the vaults; (iii) Large spans; (iv) Absence of stringcourse, cross-ties or buttresses.

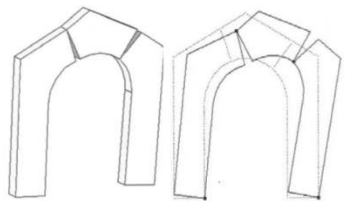
Warnings: (i) Partial or total collapse of slab, vaults or ceiling; (ii) Cracks in the vaults or arches; (iii) Lack of connection between arches and walls; (iv) Sliding of ashlars; (v) Compression at the base of the columns/piers of the triumphal arch.

Examples:

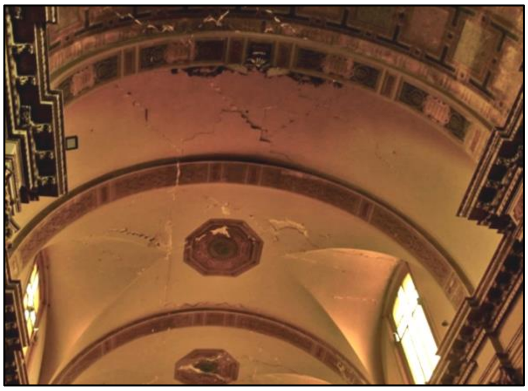
(C8-C9-C12-C18-C24-P12-P13)



(C13-P9)



Emilia Romagna (Italy) – C8



Emilia Romagna (Italy) – C8

Christchurch (New Zealand) – C13

Emilia Romagna (Italy) – C8

Emilia Romagna (Italy) – C13

ANNEX 9. MECHANISMS DUE TO IRREGULARITIES

Brief descriptions of the possible mechanisms due to irregularities are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms: Interaction between structures with different shape and height (C25, P20); Irregularities of construction or materials (P19); Annexes (P21) – Referring to the Italian damage catalogue [39]

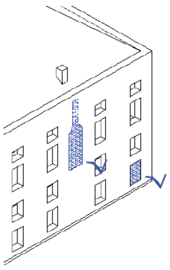
Description: Local collapses and cracks at the interaction point between structures with different dimensions, shape and materials due to the different seismic behaviour and hammering. The mechanisms include also failures caused by the lack of connection and anchoring between original structure and added elements, such as non-structural elements, in-fill openings or previous interventions.

Causes: (i) Significant difference of mass and rigidity between adjacent structures; (ii) Insufficient or absent connection and anchoring between original structure and added elements; (iii) Significant difference in stiffness between original and added materials; (iv) Poor execution of previous interventions; (v) Poor quality of the masonry; (vi) Presence of concentrated loads.

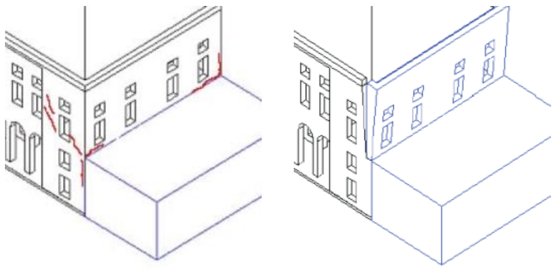
Warnings: (i) Cracks or hammering failure at interaction points (corners or edges); (ii) Rotation of the highest block; (iii) Cracks or local collapse due to the changing of material.

Examples:

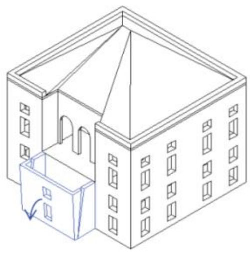
(P19)



(P20-C25)

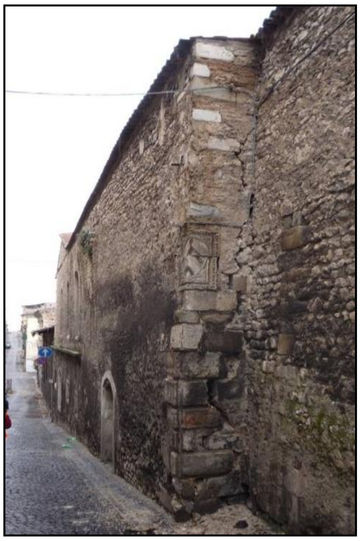


(P21)

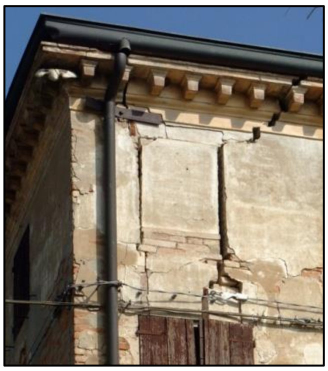


Emilia Romagna (Italy) – P20

Emilia Romagna (Italy) – P20



L’Aquila (Italy) – P20



Emilia Romagna (Italy) – P19

Emilia Romagna (Italy) – P20

Christchurch (New Zealand) – P20

ANNEX 10. MECHANISMS INVOLVING PROJECTIONS OR INDEPENDENT BLOCKS

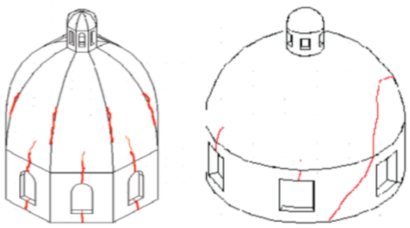
Brief descriptions of the possible mechanisms involving projections or independent blocks are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms:	Narthex (C4); Main dome and drum (C14); Lantern (C15); Domed, vaults, pinnacles, statues, parapets, chimneys (C26, P18), Bell or clock tower (C27); Belfry (C28) – Referring to the Italian damage catalogue [39]
Description:	Failures due to different seismic behaviour between the main body and each independent element such as parapet, chimney, statues, pinnacles, narthex, dome, drum and lantern. The bell or clock tower and belfry are also included. The failures observed are related to causing hammering, rocking, sliding and overturning.
Causes:	(i) Absence of cross-ties, confining rings, tie-beams, buttresses or anchors; (ii) Presence of heavy roof or ring beams and of concentrated loads; (iii) Presence of large openings and small piers; (iv) High slenderness (height/thickness ratio) of the element. Bell or clock tower: (i) Absence of diaphragms or distributed cross-ties; (ii) Poor quality of the masonry; (iii) Presence of a large number of openings; (iv) Presence of columns; (v) Presence of heavy roof.
Warnings:	(i) Failure, cracking or permanent rotation, sliding or rocking of the projected elements; (ii) Local collapses. Bell or clock tower and belfry: (i) Cracks or hammering failure at the interaction points (corners or edges); (ii) Diagonal and horizontal cracks due to shear, sliding or rocking behaviour; (iii) Cracks in the arches and columns of the belfry.

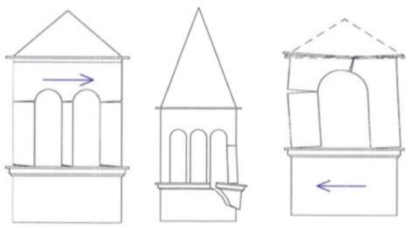
Examples:

(C15)

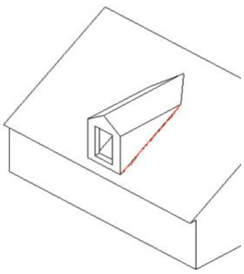
(C14)



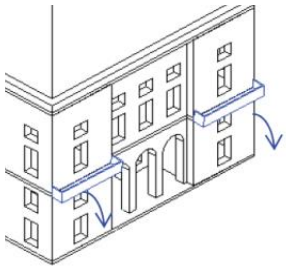
(C28)



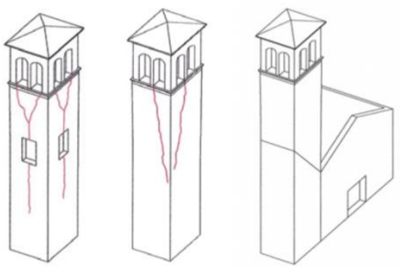
(P18)



(P18)



(C27)



L'Aquila (Italy) – P18

Christchurch (New Zealand) – C27

Christchurch (New Zealand) – C27

L'Aquila (Italy) – C28

L'Aquila (Italy) – C28

L'Aquila (Italy) – C27

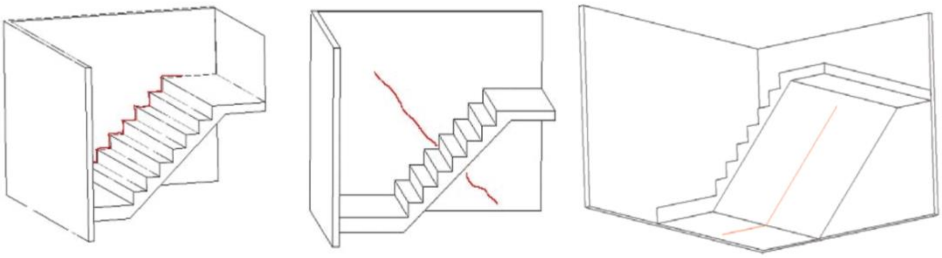
ANNEX 11. DAMAGE DUE TO STAIRS

Brief descriptions of the possible mechanisms involving stairs are presented below, including causes, warnings, schematic and photographic examples.

Mechanisms:	Damage of stairs (P14) – Referring to the Italian damage catalogue [39]
Description:	Failures of the stairs involving the lateral walls, the stair-to-wall or stair-to-diaphragm connections and other structural elements.
Causes:	(i) Insufficient connections with the lateral walls or the diaphragms; (ii) Poor connection between beams and walls; (iii) Poor quality of the masonry; (iv) Insufficient connection between the leaves
Warnings:	(i) Diagonal cracks in the lateral walls; (ii) Cracks at the head of the beams; (iii) Movements, sliding or hammering of beams; (iv) Local collapses.

Examples:

(P14)



Emilia Romagna (Italy) – P14

Emilia Romagna (Italy) – P14

Kathmandu (Nepal) – P14

Emilia Romagna (Italy) – P14

ANNEX 12. DAMAGE DUE TO FOUNDATIONS

Brief descriptions of possible mechanisms and damages due to the foundations are presented below, including causes, warnings, schematic and photographic examples.

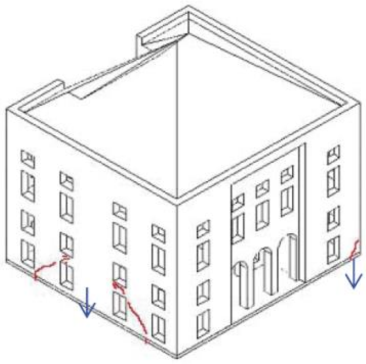
Mechanisms: Damage due to foundation settlement (P22) – Referring to the Italian damage catalogue [39]

Description: Vertical cracks or local shear failure due to ground subsidence or liquefaction. The building could lean to one side.

Causes: (i) Absence, insufficient or poor design of the foundation; (ii) Poor ground subsidence

Warnings: (i) Vertical cracks; (ii) Local shear failures with changing in level between two parts of the building; (iii) Local or global leaning or collapse; (iv) cracks in the surrounding floor.

Examples: (P22)

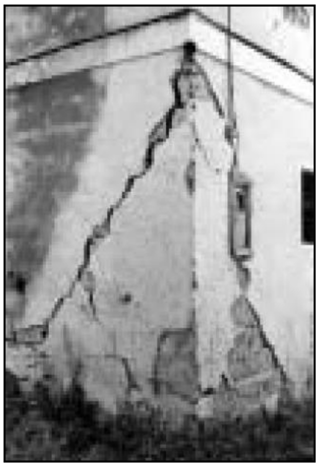


Christchurch (New Zealand) – P22

Christchurch (New Zealand) – P22

Christchurch (New Zealand) – P22

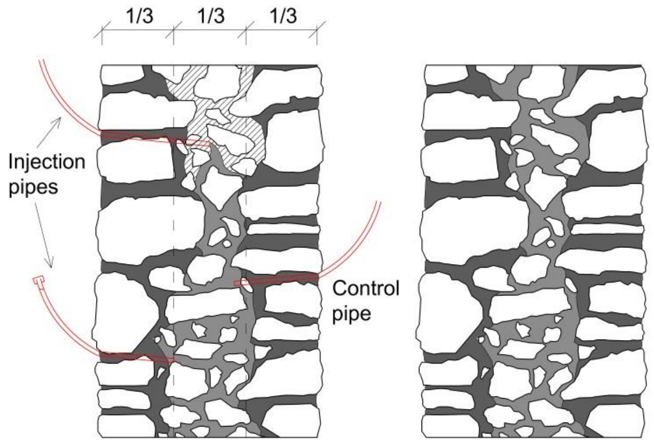
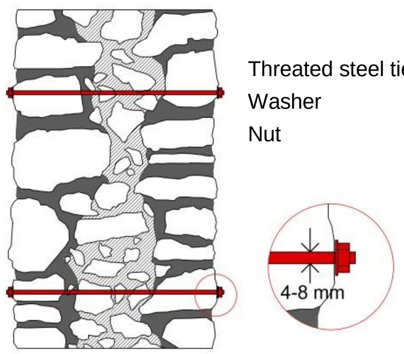
Christchurch (New Zealand) – P22

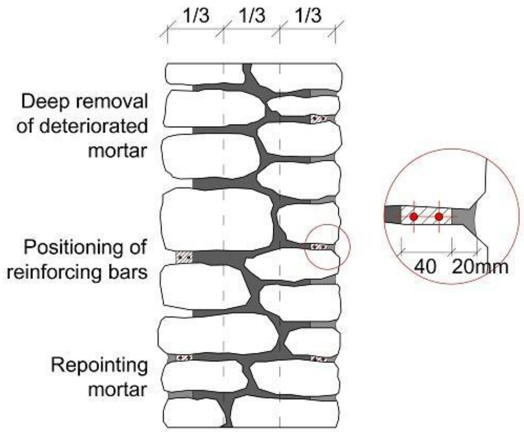
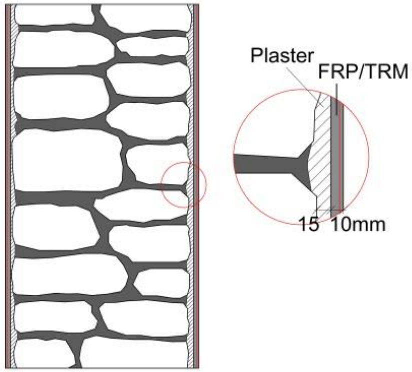


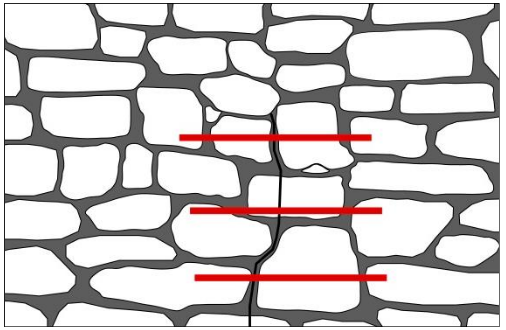
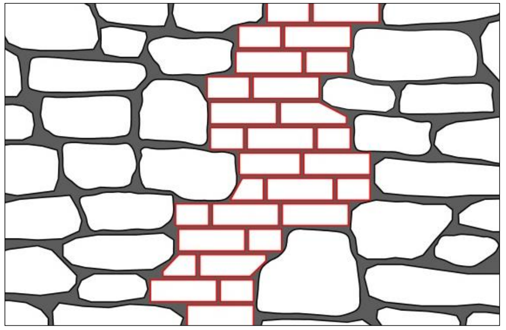
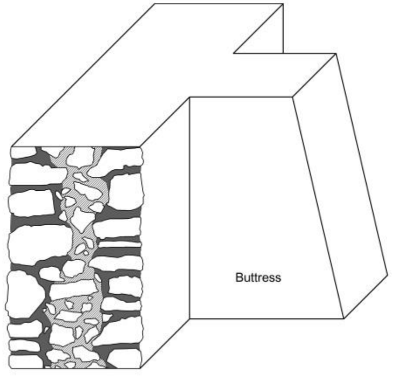
L’Aquila (Italy) – P22

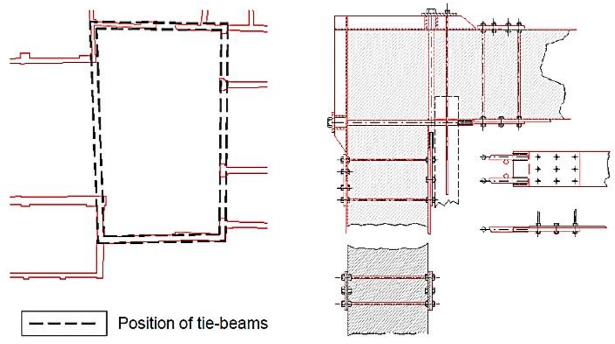
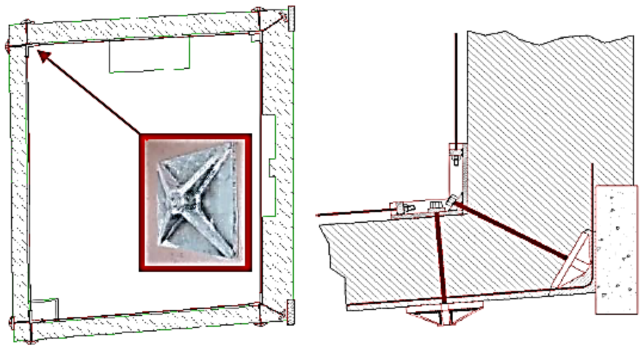
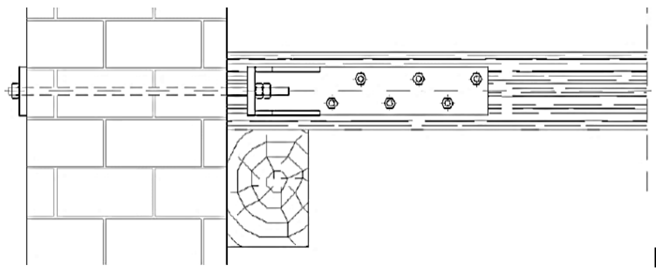
ANNEX 13. SEISMIC IMPROVEMENT INTERVENTIONS

This annex is a brief summary of possible interventions for stone masonry buildings. For each intervention references, considerations, advantages, concerns and schematic details are presented.

	Considerations	**Advantages**	**Concerns**	**Schematic**
Grout injection of the inner core **Grout injection of cracks** References: Grout [31][46][47][48][49][45][54] Test standards [45][50][51][52] Recommendations [53]	Suitability of the wall cross-section (percentage of internal voids higher than 10% of the entire wall cross-section) Selection of the grout considering chemical, physical and mechanical compatibility with original mortar and unit, durability, injectability, fluidity and stability, low hydration heat, hygroscopic properties, adhesion, salt resistance Preparation of the wall (choice of the injection point, hole drilling, repointing of joints and cracks, cleaning the core with water or air) considering 2 to 3 injection points per m^2 The injection tubes (Ø10-15 mm) must penetrate to ≥1/3 of the cross-section, and some of them at 1/2 Use of low injection pressure (0.7 to 1.5 atm)	Restore the homogeneity of the cross-section filling voids and cracks Improve the mechanical properties of the masonry Improve the bond between cross-section leaves Low-tech equipment and workmanship (although experienced) Possible use of premixed grouts	Not-reversible intervention Not suitable for walls with percentage of voids less than 4% of the entire wall cross-section Avoid the use of grout that causes segregation or shrinkage Sensitivity to sulphates and other salts (gypsum mortars, impurities in bricks and stones, cement binder)	
Insertion of tie-rods References: [32][54]	Use of steel bars fixed by bending or injection If threaded bars with a bolted head are used, place the bar in tension by warm and tighten the nut Compatibility between the type and material of the ties and the masonry May necessitate anchorage	Limit out-of-plane deformation of the wall Ensure stability, integrity of the wall Simple, fast and low cost execution Removable intervention with minor alteration of the wall	May lead to corrosion problems if ties are not suitable May be obtrusive	

	Considerations	Advantages	Concerns	Schematic
Bed joint structural repointing References: [56][57][58][59]	Selection of the mortar considering chemical, physical and mechanical compatibility with the original materials Deep removal of deteriorated mortar (50-70 mm and max 1/3 the entire cross-section) avoiding strong vibration and percussion of the masonry and washing with low pressure Structural repointing is suitable for regular texture surface (ashlar) Select the reinforcing bars considering size (Ø4-6 mm) and corrosion resistance	Increase in strength of masonry component Effective in prevention of water penetration through joints Confining effect in the masonry component Can be reversible depending on the mortar used	Not suitable if the original plaster has to be conserved May lead to corrosion if insufficient cover is provided Structural repointing may not be effective in irregular masonry (rubble)	
On-surface mounted reinforcement using: **Fibre-Reinforced Polymers (FRP)** **Textile-Reinforced Mortar (TRM)** References: [60][61][62][63] Recommendations [64][65][66]	Determination of reinforcing material Ensure bonding between reinforcing matrix and masonry Transverse connections are often recommended Selection of chemical, physical and mechanical compatible materials Can be used to stitch a crack	Increase the strength and ductility of structure High stiffness-to-weight ratio Fatigue and corrosion resistance Improve the global behaviour of the structure Flexibility and easy application allow a wide range of intervention scenarios	Requires smooth surfaces and the removal of the original plaster May be irreversible, obtrusive and invasive Heat and radiation sensitivity of FRPs May lose bond due to trapped moisture	

	Considerations	Advantages	Concerns	Schematic
Crack stitching and anchoring References: [68]	Selection of chemical, physical and mechanical compatible materials Location and type of anchoring	Improve the connections of adjacent walls or portion of wall Can increase strength and stability of structure or element Add extra strength and ductility to the wall Can control the crack propagation	Might be irreversible by harming the original fabric Can be obtrusive and invasive depending on the type and shape of material used May need intense intervention for the placement of bars	
Local dismantling and rebuilding (unstitch-stitch) References: [54]	Selection of mortar and units considering chemical, physical and mechanical compatibility with the original materials Scraping the joints and removing the units without causing percussion or vibration Ensure penetration and connection between the original and new masonry	Can be used to repair a single crack or a more extensive area Reconstitute the structural continuity of the masonry	Ineffective when there is insufficient or absent connection between existing and new masonry	
Addition of buttress	Location of application Adequate connection or interlocking must be provided between existing and new leaves Must resist lateral forces and deformations using the buttress self-weight Typically thickness of the buttress changes with wall height	Increase the stiffness and integrity of the structure Improve the global behaviour Horizontal confinement of the structure Relieve stresses in certain places	Alteration in appearance Might not be reversible depending on size and form of the buttress Lack of connection can lead to separation and collapse	

	Considerations	Advantages	Concerns	Schematic
Building confinement using tie-beam or steel cables References: [60]	Selection of chemical, physical and mechanical compatible confining materials (metal or FRP strips) Determination of locations for confining rings/bars/tis Emergency confinement could be done using polyester bands On large structures the confinement actions could be obtained using bed joint structural repointing	Improve the stability, strength and stiffness of whole element or structure Prevent separation of external leaves or other parts Can be reversible, depending on the material used and on the tension load	Can be obtrusive depending on the material used May lead to deterioration problems if not protected against environmental effects or if unsuitable material is selected Requires special care for stress distribution when adopted for square or irregular columns	 [60]
Addition of cross-ties References: [54][60]	Positioned at the roof or diaphragm level Dimension of the anchor is related to the tie tension and masonry quality Corners require special care due to the stress distribution	Improve integrity of the structure Low cost, easy installation, easy maintenance	Excessively high tension in the ties could damage the masonry	 [60]
Metal or wooden anchors **Timber tie-beam** References: [67]	Wooden beams are slightly tensioned	Prevent the beam slipping and hammering Distribute shear action Restrain the walls preventing possible overturning Reversible and not invasive	May lead to deterioration problems if not protected against environmental effects or if unsuitable materials are selected May be obtrusive	 [60]