Chapter 15
Conservation Principles and Performance Based Strengthening of Heritage Buildings in Post-event Reconstruction

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Abstract Recommendations for repairing and strengthening historic buildings after an earthquake and before the next in modern times go back to the contribution to the ICOMOS General Assembly of 1987 by Sir Bernard Fielden "Between two Earthquakes" (Fielden 1987). In that circumstance two important points were made: the first is that failure and damage should be used to understand performance and behaviour, so as to avoid measures that do not work. The second is that the engineer work should be integrated into the architecture historical methodology. Almost 30 years later this contribution investigate to which extent these two recommendations have been fulfilled, whether there is a common understanding between the conservation and the seismic engineering community and whether lessons from past failures are informing new strengthening strategies.

15.1 Introduction

The global seismic response of historic masonry buildings is highly influenced by the integrity of the connections among vertical and horizontal structural elements, to ensure the so-called box behaviour. Such behaviour, providing the transfer of inertial and dynamic actions from elements working in flexure out-of-plane to elements working in in-plane shear, leads to a global response best suited to the strength capacity of the constitutive materials, and hence enhanced performance and lower damage levels. While, many properly designed buildings of the past demonstrated such behavior when exposed to seismic action and successfully survived ground shaking (D'Ayala 2011; Tavares et al. 2014), too often, due to

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inherent defects, alterations or decay, such resilient features are not present or are not effective and lack of connections among orthogonal walls and walls and floors structures are clearly apparent. In churches with a Latin cross plan shape, delivering the box action, might result particularly difficult, due to the change in stiffness between the nave and the central crossing area and often the presence of trusting arches and domes over the central crossing pillars. The engineering community has historically remedied to such problems by developing strengthening devices, to be applied either as repair to damaged buildings or, often enough, as a retrofit and upgrading programme to improve the seismic performance of the existing building stock before the next damaging event. Such attitude towards strengthening and retrofitting is not confined to modern earthquake engineering, as retrofit programmes were promulgated around the turn of the twentieth century for instance in Turkey and Italy after major earthquakes in Istanbul (D'Ayala and Yeomans 2004) and Messina (Barucci 1990). However from recurring observation of damage in earthquakes worldwide in the past three decades, and more recently from the Pisco, Peru' 2007, L'Aquila, Italy, 2009, Maule, Chile 2010, Christchurch, New Zealand 2011, and even from the very recent 2013 Philippines event, the lack of a systematic critical approach to strengthening of historic buildings to prevent damage and casualties while preserving architectural value, clearly stands out. In general the use of materials and elements with strength and stiffness greater than the original materials is still prevalent and recommended in several guidelines. Design provisions for strengthening usually rely on capacity design approach, assuming that the retrofitted building should withstand an action proportional or equal the one decreed for new buildings of the same structural typology.

Alternatives to increase in strength and stiffness are the concepts of base isolation and introduction of damping devices aimed at modifying the response of the structure, aiming at shifting its fundamental frequencies from the frequency content of the ground shaking and increasing its damping capacity. Examples of these solutions exist in history. In modern times they have been unfrequently used from the 1980s onward, in very high profile cases, but guidelines and recommendations for application to more ordinary cases do not currently exists.

After introducing the context of structural conservation and its principles, the paper will review typical damage observed in the events listed above outlining the shortcoming of conventional strengthening approaches, strengthening interventions currently advocated by guidelines and implemented in post-earthquake retrofit programmes and proposals for alternative strategies.

15.2 Structural Conservation Principles

Seismic retrofitting intervention in heritage structures, while satisfying seismic code performance requirements, should also comply with recognized conservation principles, enshrined in international documents such as the Venice Charter of 1964 (Venice Charter 1964) and, more specifically, in the ICOMOS/ISCARSAH

Recommendations for the Analysis and Restoration of Structures of Architectural Heritage, (ICOMOS/ISCARSAH 2003), and the Annex on Heritage Structures of ISO/FDIS 13822, (ISO/TC96/SC2 2010). These criteria however do not have the same legal enforcement framework of a seismic standard and hence should be seen as guidelines useful to strike a balance between the improvement of the seismic behavior and the retention of the existing fabric and architectural and cultural value. The ISCARSAH Principles besides reconfirming the more generic conservation principles of conserve as found, minimal intervention, compatibility, and reversibility of repair, introduce concepts specific to the structural and seismic performance of buildings and have direct consequences on seismic strengthening. These are the concepts of:

- *Structural authenticity*, which should be preserved as much as the architectural authenticity, ensuring that the original mechanical and resisting principles governing the structural response are not altered and original structural elements are not made redundant.
- *Structural reliability*, relates to the necessity of striking the correct balance between the public safety requirements and the preservation requirements. Conventionally it is accepted that buildings of high cultural significance may be intervened upon so as to ensure damage limitation as a performance target, in events where for ordinary buildings, life-safety is the performance requirement. However in many occasions the attainment of such target may cause a significant loss of artistic or cultural value, maybe greater than the ones bestowed by the earthquake damage, in probabilistic terms. Hence the extent of seismic upgrading should be verified by a cost-benefit analysis including the intangible value losses. According to ISO/FDIS 13822, the solution finally adopted should consist of "an intervention that balances the safety requirements with the protection of character-defining elements, ensuring the least harm to heritage values". This is also defined as "optimal or minimal intervention".
- *Strengthening compatibility, durability, reversibility, monitorability*. These criteria influence more directly the technical choices and details of the interventions and impact upon: the suitability of "new" materials and structural elements in terms of their physical and mechanical performance when compared with original materials and structural elements; their performance in time; the possibility of removing partially or totally the intervention if monitoring proves that it is not suitable. Compatibility should be such that the new materials and elements not only do no harm to the original ones, but also they act as sacrificial elements in presence of external actions, i.e. they should act as fuses of the structural system. At the same time the new elements should be durable as to extend the expected life of the original structures as intended, but should also be non-intrusive, non-obtrusive and reversible. The concept of reversibility, or more realistically removability, is a very interesting one, as it acknowledges limitation in current practice and the possibility of finding better solutions in future. Removability is strictly correlated with the idea of monitorability, i.e. the possibility of observing and recording the performance of both the original

structure and the intervention, to ascertain its effectiveness or alert of any possible undesirable side-effect.

These criteria, although having being actively debated and applied in the international structural conservation community for at least the last 25 years, to my knowledge, they were eventually given recognized status, in 2003 with the approval of the ISCARSAH principle by the ICOMOS general assembly in Zimbabwe. It is hence worthy, a decade later, to verify on one hand how they have been incorporated into national and European seismic codes and on the other whether they had any impact on current seismic strengthening practice. A useful point to start this investigation is to review the performance in recent past earthquake of buildings strengthened with conventional force enhancing systems.

15.3 Damage of Heritage Buildings Strengthened with Conventional Capacity Enhancing Systems

In the last two decades increasing attention has been paid worldwide to the performance of historic and heritage buildings during major seismic events and specific surveys included in reconnaissance missions and reports. It is recognized that such buildings represent on one hand valuable cultural and economic assets to their country and to humanity at large, on the other they are in some cases responsible for non-negligible death tolls and casualties, hence appropriate mitigation measures need to be considered (see Blue Shield statements, after natural disaster, such as http://www.usicomos.org/international-icomos-news/blue-shield-statement-haiti-earthquake).

Well known examples of the lethality of heritage buildings are the collapse of the vaults of San Francis of Assisi basilica in the 1997 Umbria Marche earthquake (Spence and D'Ayala 1999), the collapse of several timber and mud vaulted roofs caused by the 2007 Pisco earthquake in Peru'(Cancino 2010), collapses of several adobe churches in the Colchagua Valley during the 2010, Maule Chile event in (D'Ayala and Benzoni 2012), partial collapses of several churches in the 2009 L'Aquila, Italy and the dramatic collapse of the Bell tower and spire of Christchurch Cathedral, New Zealand, in 2011. (Dizhur et al. 2011.) Following a two year long legal battle, what remains of the cathedral is now listed for demolition. A similar approach to damaged heritage was witnessed in Peru' following the 2007, Pisco earthquake and in Chile following the 2010, Maule earthquake. Indeed in Chile a generalized call for demolition of the architectural heritage damaged in the earthquake seemed to be the immediate reaction common to the people living in the small traditional communities as much as to the Governmental Authority of the Santiago Metropolitan Area. This approach is in contrast with the ICOMOS charters (Venice 1964; Cracow 2000) and with the attitude exhibited, for instance, by the communities of Bam (Fallahi 2008; Ghafory-Ashtiany and Hosseini 2008) or L'Aquila (D'Ayala and Paganoni 2011; Rossetto et al. 2014), which have seen their

Fig. 15.1 (**a**) Church in Lalol, Colchagua, Chile. Collapse of the lateral adobe wall, strengthened by shotcrete. (**b**) Church in Curepto, Maule, Chile. Collapse of the lateral adobe wall, strengthened by shotcrete

historic centers evacuated while waiting for funds and strategies to repair and rebuild. Montez and Giesen (2010), observe that the lack of provisions in Chile for the retrofit of historic buildings creates two options: to leave the building untouched or to adapt the structure to the present code, introducing reinforced concrete or steel elements. In the visited sites in the Valle de Colchagua, where historic structures experienced damages during previous earthquakes, recurring typologies of repairs and strengthening were observed. These consisted in prevalence of shotcreting of longitudinal adobe walls, although this was not always implemented in conjunction with wire mesh and through thickness ties. The shotcreting often accelerated deterioration of the original adobe wall. In general shotcreting has not been sufficient to prevent cracking and partial or total collapse of the adobe walls as evidenced by the collapses in the church in Lalol and in the church in Curepto (Fig. 15.1a, b). Current research on geo-synthetic mesh is aimed at providing a more effective alternative than wire-mesh for confinement of adobe walls (Torrealva et al. 2008), however interventions using geo-synthetic mesh on heritage buildings have yet to be reported in literature.

The general lateral stability of churches is a main issue, due to substantial difference in lateral flexibility of internal timber colonnades and external longitudinal adobe walls. This behavior is also common to churches of similar typology in Perù that were affected by the 2007, Pisco earthquake. Blondet et al. (2008) summarized the following recurring damage observed in single naves churches:

- Horizontal cracks on the lateral walls at about 1/3 of their total height. These cracks can even break through the earthen pilasters, causing the walls to collapse.
- Diagonal cracks on some of the lateral walls.
- Detachment of the choir and the altar's wall (parallel to the façade) from the church's lateral walls and cylindrical vault ceiling.
- Appearance of vertical cracks and fissures on the church towers and detachment of the towers from the rest of the church.
- Humidity related damage.

Fig. 15.2 (**a**) Ica cathedral, (Peru') collapse of the timber barrel vaults. (**b**) Collapse of the brickwork façade of the Church San Francisco of Curico', revealing the timber structure supporting the roof. Maule, Chile

The first two points highlight the out-of-plane rocking and in plane shear, respectively, of the lateral walls. All other observations describe failures of connections among macro-elements and resulting partial or total collapse. In churches with lateral aisles created by pillar-and-arch timber frames, the author observed failures due to excessive displacement of the internal pillars and collapse of the supported vaulted roof (Fig. 15.2a).

The lateral stability could be enhanced by bracing roof structures and by providing better transverse connections between the columns and walls. On the visited sites it was noted that many of the columns did not have foundations or plinths, but were simply sitting on the ground. Possible improvements in behavior could be achieved by the addition of a foundation system and the connection of the longitudinal and transverse roof structure to both the columns and the adobe walls. Use of timber wall-plates anchored to the walls by means of timber pegs should help redistribute the load of the roof structures, avoiding concentration of stresses and hence unfavorable localization of vertical cracks. Loss of the façade by overturning was not usually an issue, neither in Peru' or Chile, except for one surveyed case in Curico' (Fig. 15.2b). This show of resilience can be attributed to the relatively low horizontal and vertical slenderness ratio of the main facade the presence of two flanking bell towers, and in general the absence of very steep gables.

An extensive review of damages to churches following the earthquake in L'Aquila was conducted by Lagomarsino (2012) with the aim of correlating some constructive and strengthening features with corresponding collapse mechanisms.

Fig. 15.3 (**a**) Cathedral of L'Aquila, Italy; (**b**) The basilica of Collemaggio in L'Aquila, Italy

This study highlights the generally positive performance of wooden ties and ring beams found in heritage buildings which survived or where repaired in the aftermath of the 1709 devastating earthquake, and identifies the façade overturning and the gable overturning among the most common observed mechanisms, triggered by a general lack of connections of these macroelements to the longitudinal walls and the roof structure, but rarely resulting per-se in collapse. Indeed many detached facades were visible in the earthquake aftermath. The most recurring observation made in L'Aquila by several researchers (D'Ayala and Paganoni 2014; Augenti and Parisi 2010), refers to the pervasive substitution of historic timber roof trusses with twentieth century concrete trusses and slabs. Many of the observed collapses are directly connected with this change in stiffness and mass of the roof and are usually affecting the area of the transept and main crossing of the church. The most notorious examples are the Collemaggio basilica and the Cathedral of St. Massimo and Giorgio (Figure 15.3a and 15.3b). In both cases ring beams had been added at the top of the walls and the arches over the central crossing. However several other churches in L'Aquila had similar interventions, such as the church of St. Marco or the church of Santa Maria Paganica (Fig. 15.4), and although the roof was made with slightly less heavier solutions, the outcome was still the loss of the cover of the central crossing and of the nave. The church of Santa Giusta (Fig. 15.5), where the ring beam had been made by reinforced masonry rather than concrete performed marginally better with localized damaged but without major collapse.

An extensive survey of damaged churches was also conducted in the aftermath of Christchurch earthquake swarm of 2010–2011, by the Masonry Recovery Project (Dizhur et al. 2011). While the majority of the churches surveyed in L'Aquila were first built in the mediaeval period with poorly cut masonry stones and relatively poor lime mortar, then altered in the eighteenth century with baroque additions, the religious heritage in Christchurch mostly dates from the nineteenth Century and

Fig. 15.4 Collapse of the roof and vaults of the church of Santa Maria Paganica, L'Aquila

Fig. 15.5 Partial collapse and evidence of a reinforced masonry ring beam in the Church of Santa Giusta in L'Aquila

beginning of the twentieth. However, as just less than 50 % of the churches surveyed were built in either stone or clay brick masonry a comparison between the observed damage and any strengthening that was implemented at the time of the earthquake for the two sites might be of some interest. Statistics of damage reveal that for both typologies, brick and stone masonry, approximately 80 % of the buildings surveyed were either structurally damaged or presented partial collapses. This corresponds to either yellow or red tagging and according to New Zealand rule, implies demolition, if the structure is deemed unsafe. The two recurring mechanisms observed were partial overturning of the main façade and in-plane failure of the longitudinal walls. Although various strengthening techniques are mentioned by Dizhur et al. (2011) including shotcreting, steel strong-backs and steel moment frames, besides the use of adhesive anchors, it is not stated whether

and to which extent any of these systems were used in heritage buildings. Following the New Zealand 1991 Building Act (New Zealand Parliament 1991), all unreinforced masonry buildings deemed earthquake prone (EPB) with an ultimate lateral capacity less that 1/3 of new built demand at the same site, should have been retrofitted to raise their ultimate capacity to 50 % of new built demand. NZSEE advocated this to be raised to 67 % of the ultimate demand for new design and this change was included in the 2004 Building Act (New Zealand Parliament 2004). Heritage buildings listed as EPB should be either strengthened or demolished, within a timeframe varying from 5 to 25 years, depending on enforcement provisions of the single territorial authority in relation to the perceived risk (McClean 2009).

According to Turner et al. (2012), a large proportion of retrofitted masonry buildings surveyed in the Commercial Business District of Christchurch, post February 2011 event, only had restrained gables and wall anchorage to floors and roofs, with a few cases of roof diaphragm improvements, while a minority also had installed additional vertical elements to the original lateral force resisting system. These would include concrete and steel moment frames, reinforced concrete and masonry walls, steel diagonal braces, and strongbacks. Horizontal retrofit elements included addition of plywood sheeting to roofs and floors as well as horizontal steel trusses to improve diaphragm action. In many cases was noted that irregularly spaced, insufficiently sized and too far apart anchorage proved ineffective in avoiding the separation of walls from floor structures or external wythes from internal ones, whilst regular layouts prevented out-of-plane failures. Weak mortar was also a cause of premature bond failure in the mortar joints, preventing stress transfer from the anchor to the masonry fabric (Wilkinson et al. 2013). In several cases buildings retrofitted with additional steel or concrete frame did not performed well with partial or total collapse of the masonry walls (Wilkinson et al. 2013; Turner et al. 2012). From a conservation point of view, this type of intervention is considered totally against the principle of authenticity and reliability stated in Sect. 15.2, but also against several of the strengthening criteria. In the aftermath of the Christchurch earthquake, the issue of how heritage buildings should be dealt with was brought to front by a Governmental public consultation exercise closed in March 2013, the Building Seismic Performance Consultation document, Proposals to improve the New Zealand earthquake-prone building system (Ministry of Business, Innovation and Employment (MBIE) 2012). Five questions were specifically aimed at heritage buildings, including, what factors should be considered when balancing heritage values with safety concerns, what are the deterrents for heritage building owners to proceed to strengthen their buildings, what are the cost and benefits of setting a consistent set of rules across the country for heritage building strengthening, what guidance will be needed by owners and communities to strengthen heritage buildings. SESOC (NZ Structural Engineering Society 2013) provided a very comprehensive answer to these questions in terms of expected performance target, specifically noting that "Heritage buildings in private ownership are potentially under threat due to the high cost of compliance." (SESOC 2013) On one hand if the standard is only concerned with life safety compliance "may

result in buildings that are unlikely to be practically repairable after the event." (SESOC 2013) On the other hand, a higher level of protection can only come at extra cost to the owner, as the current system does not make provision for public subsidies. "SESOC supports the Historic Places Trust recommendation for the development of a National Risk Map for New Zealand's heritage. This may form the basis of a prioritization of heritage buildings requiring additional protection; and could also inform an approach to public funding (or partfunding)" (SESOC 2013). Answers to the first of the five questions are particularly relevant to this paper. SESOC viewpoint is that buildings should not be assessed in terms of percentage of capacity of new build demand, but instead specific vulnerabilities should be identified and amended. The major drawback of the current assessment approach is seen as the lack of an assessment of actual ductility reserves. A major issue felt is whether there is consistency on the application of ICOMOS principles, for instance in relation of clearly visible, external to the original fabric retrofit elements, which are less costly to implement and more likely to be effective. Finally it is not clear whether the driver for decision making should be the public safety concern or the preservation of the heritage value.

15.4 Strengthening Strategies Included in Standards and Guidelines

It was seen in the previous section that the re-instatement of continuity of load paths and the delivery of a robust global behavior are paramount for the seismic upgrade of historic buildings. A wide range of techniques and products are described in the scientific literature and used in current practice to ensure the enhancement of damaged or underperforming connections. However, as observed in the introductory section, in respect to engineered structures, heritage buildings require far more attention, especially when dealing with issues such as the compatibility between the chemical and mechanical properties of the strengthening system and the parent material. Many strengthening techniques, after an initial success and a strong commercial promotion, have proved to be unable to perform at the required level and showed unexpected drawbacks when undergoing dynamic loading outside the controlled conditions of the laboratory environment (see for instance the extensive programme of onsite testing of adhesive anchors connections conducted within a joint project of University of Auckland and University of Minnesota, Dizhur et al. 2011). On the other hand, strengthening systems can provide highly flexible applications and meet the expected requirements in terms of performance; indeed, some of these systems draw on traditional reinforcement techniques, with the addition of innovative materials and a deeper insight in the laws governing the dynamics of structures. In the following we briefly review the provisions included in the standards and Codes of practice of the countries considered, before looking at some implementation on heritage buildings observed in L'Aquila.

15.4.1 Peruvian Code

"Strengthening of structures" is ruled in the National Building Code, E.030, Section 8, in its 2014 version proposed for public approval (Comité Técnico Permanente Norma E.030 Diseño Sismorresistente NTE E.030 2014).[1] The provision are easily summarised: structures damaged by earthquakes should be evaluated and repaired so that the possible structural defects that cause the failure can be amended and they can recover their resisting capacity toward a new seismic event, according to the Earthquake –Resistant Design Philosophy of the Code. Structures affected by an event, should be evaluated by a civil engineer, to determine whether reinforcement, repair or demolition is required. This study must consider the geotechnical characteristics of the site. The repairing process should be able to give the structure an adequate combination of stiffness, resistance and ductility and should guarantee its good behaviour for future events.

The repairing or reinforcement project will include the details, procedures and constructive systems to be followed. No further details are provided in this version of the code and the document itself does not include unreinforced masonry or adobe structures. Current work undertaken by the author's research group in collaboration with Getty and PUCP aims at providing guidelines for assessment and strengthening of four common types of Peruvian heritage buildings (Ferreira et al 2014).

15.4.2 European and Italian Codes

Eurocode 8, Section 6.1 Retrofit Design Procedure for existing building (EN 1998–3:2005), states that the design process of strengthening elements should cover:

1. Selection of techniques and/or materials, as well as of type and layout of intervention;
2. Preliminary sizing of additional structural parts;
3. Preliminary calculation of stiffness of strengthened elements;
4. Analysis of strengthened structure by linear or non-linear analysis. The typology of analysis is chosen depending on the level of knowledge regarding the geometry detailing and materials of the structure;
5. Safety verifications for existing, modified and new structural elements carried out by checking that the demand at three different limit states – Damage Limitation, Significant Damage and Near Collapse – is lower than the structural capacity.

The safety checks should be carried out using mean values of mechanical properties of existing materials obtained from either in-situ tests or other available documentation, taking into account the confidence factors (CFs) specified in

[1] Consulted in Spanish version.

Eurocode 8 Section 3.5 (EN 1998–3:2005). Conversely, for new materials, nominal properties shall be used without modification by confidence factor. The code also states that in case the structural system, comprising both existing and new structural elements, can be made to fulfill the requirements of EN1998-1:2004, the checks may be carried out in accordance with the provisions therein (EN 1998–1, 2004). This last sentence indicates that for systems such as reinforced concrete ring beams or corner confinement, reference can be made to the specifications for RC members in the relevant sections of EC8 and other Eurocodes for new design. However, this leaves open the problem of quantifying the interaction between original and additional structural elements and the assessment of the global seismic performance of the strengthened structure will still be affected by a large number of uncertainties.

Other strengthening systems hardly feature in codes. This could be due to the fact that the sizing of the element itself, for instance a steel cross-tie with end plate, is fairly straightforward and established in the current technical know-how; furthermore, formulas can be drawn from those of other structural members, e.g. axial capacity of steel element. Still, designers are left to their own devices when assessing the interaction between old and new, the hierarchy of failure mechanism that the connection should comply with, the value of bond or slip that should correspond to a specific performance target.

In other cases the lack of standardization is caused by the recent development of techniques as well as the high level of expertise and financial resources required for their implementation. Innovative technologies haven't been extensively applied and validated in real-life situations yet and the retrofit of a complex, precious building by means of unconventional systems is a difficult task that goes beyond the standard conservation practice. In fact, looking at the current scientific literature, it is clear that many projects of restoration and upgrade of monumental buildings are carried out by organizations within the framework of specific research projects, or by large enterprises that specialize in the production and design of advanced strengthening devices. On the other hand, it could be argued that it is this lack of appropriate standards and procedures which leads to incorrect application of novel strengthening systems and lack of awareness of innovative more suitable techniques.

In some occasions, following major destructive events, ready to the market technology finds a sudden growth in popularity and implementation which pre-date the standardisation phase.

It is worth noting however, that some systems, in spite of their relatively recent development, have already been included in specific technical guidance documents, as in the case of Fibre Reinforced Polymers, whose use in retrofit of substandard structures is addressed in the CNR-DT 200 R1/13, Italian National Research Council (CNR), (CNR-DT 200 R1/13, 2013).[2] This recently re-issued Italian

[2] This version of the Guidance document is in Italian. A previous version CNR-DT 200 /2004 is translated in English.

guidelines (Italian National Research Council [CNR]) for use of FRP for the "Design, installation and control of strengthening intervention with Fibre Reinforced composites", provides advice for use of such techniques to either strengthen or reconstruct some elements, or to connect the various structural elements to improve the behaviour of the whole structure. The document covers all structural materials, including masonry. The objectives that any strengthening intervention on a masonry structure should have are listed as follows:

- The masonry structural substratum should be adequately consolidated to withstand the design actions or replaced
- Orthogonal walls should be appropriately connected
- Inadequate connections between the walls and the horizontal and roof structures should be improved
- Thrust from roofs, arches and vaults should be adequately contained
- Floors should be sufficiently stiff in their plane to redistribute the horizontal action while at the same time act as constraint for out-of-plane motion of walls.

It is not openly stated whether strengthening with FRP is suitable to meet these performance criteria or whether these are prerequisites to the use of FRP in masonry structures, however some disclaimers are included:

- Interventions with FRP cannot as a rule improve or amend situations characterised by strong irregularities in terms of strength and stiffness, even though, if applied to a reduced number of elements, they can provide a more even distribution of strength
- Interventions with FRP aimed at improving local ductility such as columns or pillars confinements are always appropriate, although
- Local intervention with FRP should not reduce the overall ductility of the structure.

Besides this very specific document, the most updated relevant legislation for interventions on heritage buildings is represented by the guidance document "Linee Guida per la valutazione e riduzione del rischio sismico del patrimonio culturale – allineamento alle nuove Norme tecniche per le costruzioni", become ministerial decree as Circolare 26/2010 (Circolare 26/2010) (see also NTC, 2008). This document incorporates all aspects of the ISCARSAH guidelines mentioned in Sect. 15.2, while at the same time conforming to the performance based approach of the latest version of the technical standards for implementation of the Eurocode at national level. The specific recommendations of the Linee Guida are further described in the next section.

15.4.3 New Zealand Provisions

The New Zealand provisions for strengthening and retrofit are summarised in the NZSEE document "Assessment and Improvement of the Structural Performance of

Buildings in Earthquakes" (NZSEE 2006, revised version 2012). The document focuses on the assessment of all type of structures including unreinforced masonry buildings, but it does not distinguish for buildings of historical or cultural value. Moreover Derakhshan et al. (2009) have proven that some of the criteria used in NZSEE 2006 are over-conservative when considering the out-of-plane response of masonry walls and have proposed an alternative displacement based procedure. The strengthening strategies are confined to Sect. 13.6 and subdivided by the strengthening effect in in-plane strengthening, face load strengthening, combined face load/in-plane strengthening, diaphragm strengthening and chimneys towers and appendages.

Shotcrete is recommended for in-plane as well as out-of plane performance enhancement, as well as FRP wrapping. To prevent out-of plane failure anchoring to floors and walls is recommended, as well as buttressing and addition of columns, while the in–plane performance can be enhanced by introduction of concrete frames and v-braced frame. There is no value judgement or guidance for which intervention is most suitable to specific conditions or to which extent any of the suggested interventions contributes typically to the lateral capacity demands enhancement. Moreover no advice is given of how to choose among different strengthening options from each set that together would deliver the best integrated and overall performance. A commentary provides for each technique further details that should ensure good quality implementation and effectiveness.

15.5 Evidence from the Field: Strengthening in L'Aquila

In conjunction with a return mission to L'Aquila organized by EEFIT in November 2012, (Rossetto et al. 2014), the author had the opportunity to inspect a small number of building sites where conservation and repair projects were underway. These visits provide some insight on how retrofitting strengthening projects are implemented. The masonry fabric typologies most frequently observed in the district of L'Aquila for heritage buildings are rubble stone, roughly squared stone blocks mixed with bricks, sometimes in regular courses, brick masonry, and dressed stone blocks. Walls in a few cases appear to be massive, but most commonly are formed by the so called "muratura a sacco", namely two wythes of dressed stones poorly connected, sometimes with a rubble infill. Mortar is mainly lime mortar. Large squared stone blocks are used for quoins. A typical intervention that was observed to be extensively used at the few sites which were undergoing restoration at the time of the EEFIT mission and that could be visited is fluid mortar injection grouting of all bearing walls (Fig. 15.6). The aim of such an intervention is to improve the coherence and cohesion of existing walls by injecting them with fluid grout through a series of drilled holes regularly spaced on a 500 mm grid and proceeding from the bottom to the top, after having sealed and repointed the mortar joints. Although for material compatibility only lime-based grouts should be used, often epoxy additives or cement are included in the mix for faster setting. While

Fig. 15.6 Wall prepared for grout injection

such additives might improve the short term strength and cohesion of the masonry, they can create serious long term problems in terms of decay of the original materials due to different hygro-thermal behaviour and salt content release. One of the major issue is that such interventions are not directly monitorable. One way of verifying their effectiveness is to conduct flat jack tests of the masonry wall, before and after strengthening, although this is partially destructive.

Strengthening of floor to improve diaphragm action is recommended by the Linee Guida (Circolare n. 26/2010). This can be achieved by either nailing superimposed sets of floorboards at right angles or by adding a lightweight reinforced lime-based concrete screed above the existing set of floorboards. The reinforcement should be anchored in the perimeter masonry walls. Extensive tests campaign have been carried out at several institution in Italy in past years to devise the best technical details and performance improvement that can be obtained with such interventions (Riggio et al. 2012). The joists and beams forming the floor structures should also be anchored to the walls by means of ties. A similar approach should be followed also for roof structures (Giuriani and Marini 2008). This type of intervention was traditionally extensively applied in the past and it can be observed that in cases where the ties have been well maintained and are regularly distributed on the wall, the damage is usually no greater than airline cracks.

A common structural element of many buildings in L'Aquila is the brick vault. Brick vaults are present in lower floors of residential buildings as a load bearing structure with a typically shallow cross-shaped arch profile, as a non-loadbearing false ceiling in upper floors (built in folio) and in most religious buildings as support to the roof structure. Post-earthquake surveys have revealed partial collapse and extensive damage of these structures. The Linee Guida (Circolare n. 26/2010) recommend either the use of traditional steel ties or specifically built spandrels at the extrados (Ferrario et al. 2009) while strengthening intervention with extradossal reinforcement made of FRP strips (see Fig. 15.7) are tolerated with numerous provisos. While a body of research exists on the strength gain benefit of such

Fig. 15.7 Reinforcement of a cross vault with strips of FRP laid at the extrados

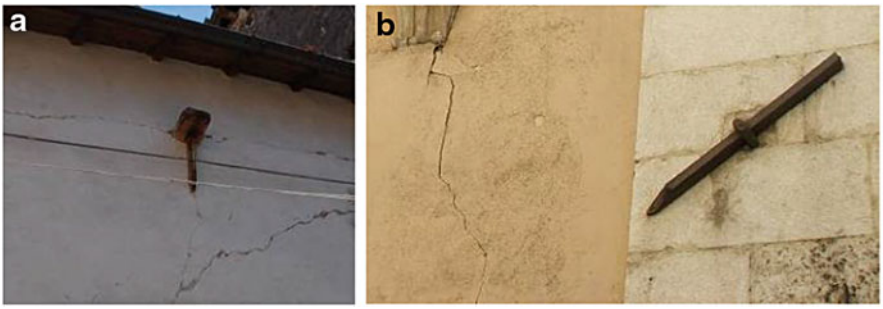

Fig. 15.8 Two examples of traditional reinforcement: (**a**) timber tie, (**b**) wrought iron cross tie inserted in a quoin

interventions, most of the experimental research conducted to date focus on static concentrated loading conditions, or support movement, rather than dynamic performance (Modena et al. 2009). Durability and breathability are the major concerns.

The Linee Guida (Circolare n. 26/2010) recommends the use of ties and anchors to connect vaults and timber floors to walls, and walls to walls. A thorough review of traditional and modern solutions, their effectiveness, shortcomings and possible improvement by use of dissipative devices is included in D'Ayala and Paganoni (2014) and some surveyed examples are illustrated in Fig. 15.8. In the few sites undergoing repair or strengthening at the time of the return mission, there was no evidence of such strengthening devices being implemented.

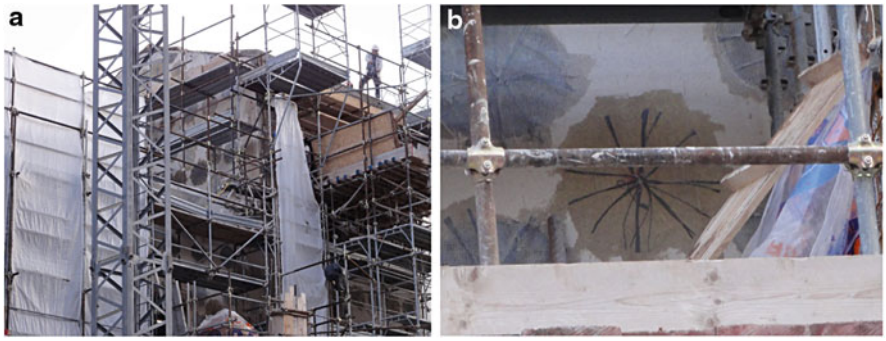

Fig. 15.9 Extensive use of reinforced coring with grouted injection with epoxy resins on the end wall of a 5 storey residential palace in the historic centre of L'Aquila

In one of the few on-going projects seen during the return mission, it was noticed that transversal reinforcement was applied to masonry walls by use of FRP bars, drilled through the thickness and then anchored by opening the threads as a star (Fig. 15.9). In the Guidelines issued in 2010 (Circolare n. 26/2010) it is stated that "the use of reinforced cores should be limited to cases where there is no other alternative due to the extreme alterations and disturbance produced vis a vis [its] doubtful effectiveness, especially in the presence of walls with several wythes not well connected. In any case the durability of the strengthening element, whether of stainless steel, composite plastic materials or other material, should be ensured and the grouts used should be compatible with the original materials". Moreover it is advised that this type of intervention only has at best a local effect (Circolare n. 26/2010).

15.6 Dissipating Energy as an Alternative to Strengthening

The drawbacks of strength-based systems were clearly brought to the fore by the seismic events reviewed in Sect. 15.3. Low compatibility in terms of mass and stiffness of concrete ring beams, often inadequately connected to the existing masonry, concurred to cause tragic collapses, as in the case of the Collemaggio basilica in L'Aquila (Gattulli et al. 2013). Numerous are the failures observed when traditional timber roof and floor structures are substituted with concrete ring beams and slabs in an attempt to deliver diaphragm action. The sudden change in stiffness and the difference in shear capacity of the two systems is simply too substantial to be accommodated by the interface. Shotcreting has also proven inadequate when coupled to both adobe and stone masonry due to poor bond to the parent material that can be achieved and maintained as the masonry decays for lack of proper aeration. The New Zealand approach of inserting new lateral resisting system, such

as steel or concrete frame, while not always effective, is certainly, if not extremely sensitively designed, in breach of most of the ICOMOS/ISCARSAH acceptance criteria.

On the other hand, cross-ties, which have been and still are commonly applied in rehabilitation practice not just in Europe (Tomaževič 1999), but also in Latin America (D'Ayala and Benzoni 2012) and new Zealand (University of Auckland 2011), are able to restore the box-like behaviour, without a substantial increase of mass, if they are regularly distributed and properly sized. Indeed, traditional cross ties can provide connection at the joints of perpendicular sets of walls, where poor quality, previous damage, or general wear and tear facilitate crack onset and otherwise out-of-plane failure. Nonetheless, localised damage at the head of the anchorage similar to punching shear is a possible drawback, which might become a major problem when damage limitation and protection of valuable finishes should be pursued or might eventually lead to the wall overturning failure (Wilkinson et al. 2013).

The concept of reducing demand by dissipating energy in a controlled way is not novel, nor recent. With specific references to applications of the concept to masonry structures and heritage buildings in particular, Benedetti (2004, 2007) developed a series of energy absorbing devices drawing on the observation that the more energy is absorbed through damage by non-critical elements of the structure the less likely is that global failure occurs. Key feature of the devices were activation for small relative displacement (1 mm) and long displacement range (up to 10 mm), i.e. low level of damage, ability to accommodate both in plane and out of plane movements, low magnitude of forces at the interface with the parent material (0.3–0.5 KN). The devices were set in series with traditional steel ties connecting parallel walls. Martelli (Martelli 2008) also highlights a relatively conspicuous number of high-profile cultural buildings in Italy that have been strengthened, either post or prior a damaging event, using one or more energy based devices such as shock transmitter units (STUs) and shape memory alloy devices (SMADs) in the period 1997–2008 by using technologies developed within European Frameworks Programmes. It is stated that STUs were inserted as a dynamic constraint between a new stiffening truss and the original walls at a height of 8 m along the longitudinal walls of San Francis Upper Basilica in Assisi. The displacement range in the STUs is ± 20 mm with maximum forces of 220–300 KN. Among these early interventions listed by Martelli (2008) stands out the Santa Maria di Collemaggio Cathedral at L'Aquila, which was retrofitted by installing Elasto-Plastic Dampers (EPDs), within a system of diagonal cable braces in the bottom plane of the roof trusses. The aim of the intervention was to limit transmission of large forces from the nave walls to the façade and the transept due to the truss structure inserted at the roof level to ensure coupling in the vibration of the longitudinal walls. The appropriateness of this intervention, among the few being tested by a real event, was reassessed after the collapse of the central crossing (Gattulli et al. 2013). A rocking-damper system, called DIS-CAM (DISsipative Active Confinement of Masonry) was developed and installed within the framework of the project of restoration of the drum of the dome of S. Nicolò church in Catania, although the collapse in this case was due to long term decay (Di Croce et al. 2010).

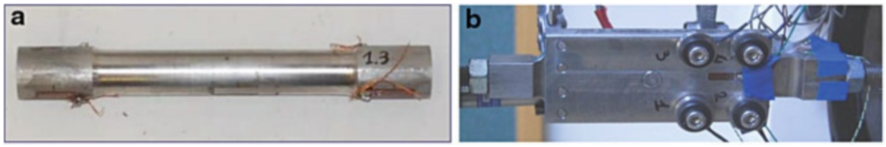

Fig. 15.10 Dissipative devices prototypes: (**a**) hysteresis based; (**b**) friction based

Drawing on the principles of performance based design, allowing and controlling modest drift and limiting damage by providing sacrificial elements able to dissipate energy, Paganoni and D'Ayala (2010) in collaboration with Cintec International developed two prototypes of dissipative anchor devices to address the problem of out-of-plane mechanisms of facades and lateral walls (Fig. 15.10).

The devices are conceived to be inserted at the connection between perpendicular walls, as part of longitudinal steel anchors grouted within the thickness of the walls. This type of installation ensures a low impact on the aesthetic of the building as it doesn't affect the finishing. The anchors can also be installed between floors elements and walls.

While the anchors improve the box-like behaviour of the building, contributing to an increase of stiffness that improves the structural response to small excitation, the devices allow small relative displacements between orthogonal sets of walls; for higher horizontal loads, they dissipate part of the energy input into the structure so that problems of localised damage can be avoided. Therefore, the design focuses on the achievement of control of displacements and reduction of accelerations and stress concentration.

Of the two developed prototypes, one is based on yielding, the other on friction. The former relies on a stainless steel element with a lower capacity in respect to the anchor, this lower capacity depending on a reduction of cross sectional area and the use of a different steel strength class. The friction prototype consists instead of a set of metallic plates able to slide past each other once a pre-set threshold of force is overcome, this been governed by controlled pressure.

The two dissipative devices, covered by patents, have been extensively validated by cyclic pseudo-static and dynamic tests on the isolated devices (Paganoni and D'Ayala 2010), and by cyclic pull-out tests on specimens modelling the T joint between two perpendicular walls connected by a passing anchor (D'Ayala and Paganoni 2014). The devices' performance has then been calibrated by using real time history obtained by obtaining from a finite element nonlinear analysis the relative motion at the crack of two orthogonal walls of a two storey house subjected to a real accelerogram from the L'Aquila earthquake. The response of the two devices is shown in Fig. 15.11.

What is relevant to the above discussion is the possibility to determine a rigorous design and dimensioning procedure, based on experimental results and on the principle of performance based seismic response. The strengthening apparatus can be seen as a relatively simple system made of a number of components in series. The objective is to determine the performance criteria of the dissipative

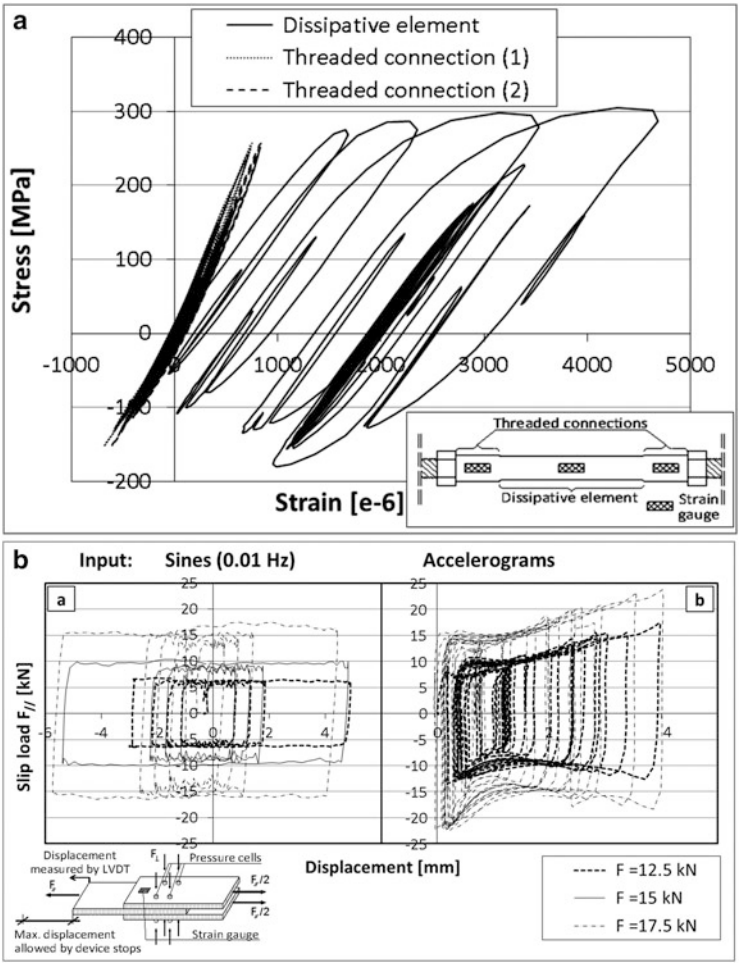

Fig. 15.11 Devices' response to accelerogramme excitation (**a**) hysteretic device and (**b**) friction device

device so that damage to the parent material can be controlled. The procedure is briefly summarised herein.

For the demand to the strengthening system, depending on the importance of the heritage building and its protection status a choice can be made to use a suite of non-linear time-history analyses of the building to determine envelop of displacement demand requirements, or to use reference drift limits from seismic code. Then use output of above analysis or modal analysis with spectrum superposition, or other simplified procedure as advised by seismic code, to determine acceleration amplification at selected heights of structure to determine the axial force on each of a set of anchors so as to determine the number of anchors required at any given

storey of the structure, by using the initial assumption that failure of bond between anchor grout and parent material is prevented:

$$F_{bond,b/p} = f_{b,b/p} \cdot \pi \cdot D \cdot L \geq \gamma_D M_i \cdot a_j = \gamma_D \rho_m l_i h_i t_i a_j \tag{15.1}$$

Where $f_{b,b/p}$ is the bond strength of grout to parent material including safety coefficient; D and L are the diameter and length of grouted anchorage; γ_D, design safety coefficient; M_i: mass of portion of structure that bears on the ith anchor; ρ_m, l_i, h_i, t_i, density and dimensions of the portion of structure restrained by the ith anchor; and a_j is the horizontal acceleration at storey j of the structure, calculated on the basis of the performance target defined in BS EN 1998–3:2004 depending on the performance criteria and hazard return period defined for the structure with:

- F_{DNC}: near collapse (2 % exceedance in 50 years);
- F_{DSD}: significant damage (10 % exceedance in 50 years);
- F_{DDL}: damage limitation (20 % exceedance in 50 years).

Once the anchor is preliminary sized, the capacity of the dissipative devices can be determined by using two different approaches depending on the device. In case of grouted metallic ties with hysteretic device:

- Step 1: Determine the minimum among:
  - Yielding strength of tie,
  - Adhesion strength tie/grout
  - Adhesion strength grout/masonry
  - Punching through strength of surrounding masonry

  Hence, yielding point of hysteretic device < Minimum

  If punching through of surrounding masonry is critical, it will be necessary to improve the masonry locally with grouting, for instance.
- Step 2: Determine the ductility requirements which will lead to maximum elongation of the device, while preventing buckling.

  These two conditions will determine the yielding point of the device as well as its geometric dimension and cross section shape.
- Step 3: Verify that performance is not compromised by instability of cycles and hardening limits

In the case of grouted metallic ties with friction device

- Step 1: Determine the minimum among
  - Yielding strength of tie,
  - Adhesion strength tie/grout
  - Adhesion strength grout/masonry
  - Punching through strength of surrounding masonry

  Hence, tightening of device < Minimum

- Step 2: Determine maximum sliding requirements and energy dissipation which will determine the size of the plate and the value of friction.
- Step 3: Control stick–slip, stability of cycles, apparent hardening

The above approach requires a series of laboratory tests to determine all material characteristics and certify performance requirements of the devices before installation, and a series of onsite tests to determine quality and characteristics of the parent material and quality and strength of the bond, which can be ascertained by, for instance, static pull-out-test, aimed at ensuring also the quality of the installation.

The dissipative devices are designed to be activated at the threshold of damage limitation of the structural response, while all other components are designed to withstand the forces associated with near collapse. If the damage limitation threshold is not a requirement for the building, then the devices can be designed to greater strength capacity. In the case of the friction device it will just be a matter of determining the different activation level of the slider for different performance requirements.

But the dimensioning of the devices should not be based on the force but on the amount of energy to be dissipated and hence on the associated deformation/sliding past the force threshold. While the two values of triggering force and demand displacement are independent for the friction device this is not the case for the hysteretic device, which needs also to be dimensioned to control buckling. Hence the design will need to undergo a series of iterations to optimise the elongation of the device and its axial buckling limit. As seen in previous applications typical relative displacement is of the order of 10–20 mm leading to interstorey drifts of the order of 0.3 %, corresponding to the damage limitation threshold for historic building according to the Circolare n. 26/2010. Finally, devices need to be designed so that they can offer additional capacity at NC limit state. In particular, referring to the experimental results reported by D'Ayala and Paganoni (2014), it is important that:

- Yielding devices reach the threshold of the 5 % elongation, so that they can offer extra capacity both in terms of displacement and load capacity;
- Frictional devices reach the end of their run. This ensures that the device will offer additional load capacity, this being quantified by a safety factory of 10 (D'Ayala and Paganoni 2014).

15.7 Conclusions

A considerable amount of research has and is being conducted to improve the way in which the issue of strengthening historic buildings is approached by the engineering community. This research has led to novel assessment procedures which were not covered in details here, novel strengthening techniques which best meet the requirements of the conservation principles and attempt at maintaining both the

original structure and the historic fabric, without substantial disruption. Indeed the tragic events of the last 4 years have triggered generally very good and responsible response on the part of the engineering community, clearly more sensitive to the cultural heritage agenda than not in the past.

Public cultural differences exist and cannot be ignored when devising policies. In some countries demolition is still considered in many respects a more viable option than repair and retrofit. However recent initiatives such as the ICOMOS New Zealand Charter 2010 (ICOMOS 2010) or the new regulations for earthen buildings of historic significance, which the Ministerio de Vivienda y Urbanismo of Chile is drafting in the document NTM002 (Ministerio de Vivienda y Urbanismo, 2010), currently in the pre-standard stage, show a change in perspective of the public as well as the engineering community towards historic buildings and perhaps a different acceptance of risk.

From a technical point of view however, much training and education of professional engineers is needed to ensure that the shift in design emphasise from force requirements to displacement and energy requirements is fully understood. As seen from evidence in the field far too often strengthening of historic buildings is still pursued in terms of increasing strength and stiffness, while some assessment criteria are far too conservative. A similar training is also needed among contractors.

Hurdles of other nature, related to the economics of developing and installing dissipative devices, can be overcome, as shown by the prototype devices described in the previous section which can be manufactured in small sizes and at costs which is affordable in the retrofit of residential historic buildings, as well as more prestigious landmark. However robust testing and design protocols need to be develop to gain confidence among practitioners.

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