PERFORMANCE OF UNREINFORCED STONE MASONRY BUILDINGS DURING THE 2010/2011 CANTERBURY EARTHQUAKE SWARM AND RETROFIT TECHNIQUES FOR THEIR SEISMIC IMPROVEMENT

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Summary

The sequence of earthquakes that has greatly affected Christchurch and Canterbury since September 2010 has again demonstrated the need for seismic retrofit of heritage unreinforced masonry buildings. Commencing in April 2011, the damage to unreinforced stone masonry buildings in Christchurch was assessed and recorded with the primary objective being to document the seismic performance of these structures, recognising that they constitute an important component of New Zealand's heritage architecture.

A damage statistics database was compiled by combining the results of safety evaluation placarding and post-earthquake inspections, and it was determined that the damage observed was consistent with observations previously made on the seismic performance of stone masonry structures in large earthquakes. Details are also given on typical building characteristics and on failure modes observed.

Suggestions on appropriate seismic retrofit and remediation techniques are presented, in relation also to strengthening interventions that are typical for similar unreinforced stone masonry structures in Europe.

1 INTRODUCTION

The damage assessment inspections that were undertaken in September 2010 and again in April and May 2011 (see Moon et al., 2011 and http://www.geonet.org.nz/earthquake/historic-earthquakes/ for further details of the 4 September 2010 and 22 February 2011 earthquakes) identified 90 unreinforced stone masonry buildings in Christchurch, many of which are included on the Historic Places Trust register of heritage buildings. Most of these stone masonry buildings were constructed between 1850 and 1930 and are masterpieces by important architects of the period, such as Benjamin Mountfort, Cecil Woods and John Goddard Collins, and are excellent examples of the Gothic Revival style. Significant examples include the Canterbury Provincial Council Buildings and the former Canterbury University College, which is now referred to as the Christchurch Arts Centre. Besides their architectural value, these buildings represent the history of a relatively young country and for this reason resources should be directed towards their preservation and seismic improvement.

Most of the buildings considered in the study are now used for a variety of public functions, ranging from churches to public offices, schools and colleges, and incorporating both commercial and cultural activities.

The stone masonry buildings in Christchurch have similar characteristics both in terms of architectural features and in the details of their construction. This observation derives primarily from the fact that most of these structures were built over a comparatively short time period and were designed by the same architects

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or architectural firms. The vast majority of structures, and in particular those constructed in the Gothic Revival style, are characterized by structural peripheral masonry walls that may be connected, depending on the size of the building, to an internal frame structure constituted of cast iron or steel columns and timber beams or to internal masonry walls that support flexible timber floor diaphragms and timber roof trusses. However, there are a few commercial buildings in the Christchurch Central Business District (CBD) that are characterized by slender stone masonry piers in the front façade with the other perimeter walls constructed of multiple leaves of clay brick. These buildings are typically two or three stories in height, with two storey buildings being most common, and may be either stand-alone or row type buildings (see Russell & Ingham (2010) for further details of URM building typology). The wall sections can be of different types:

- Three leaf masonry walls, with dressed or undressed basalt or lava flow stone units on the outer leaves (wythes) while the internal core consists of rubble masonry fill (see Figure 1(a));
- Three leaf masonry walls, with the outer layers in Oamaru sandstone and with a poured concrete core, such as for the Catholic Cathedral of the Blessed Sacrament (see Figure 1(b));
- Two leaf walls, with the front façade layer being of dressed stone, either dressed basalt or bluestone blocks, or undressed lava flow units, and the back leaf constituted by one or two layers of clay bricks, usually with a common bond pattern, with the possible presence of a cavity or of poured concrete between the inner and outer leaves (see Figure 1(c)).

(a) Cramner Court – 3 leaves with rubble fill

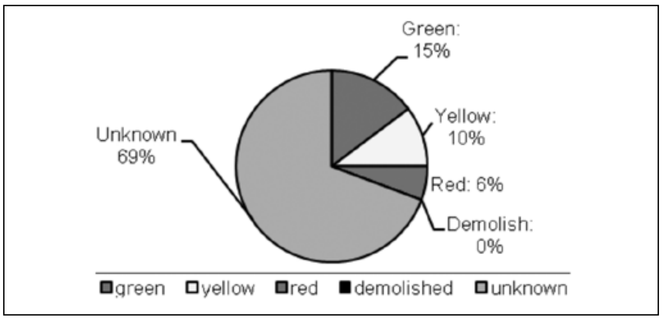
(b) Cathedral of the Blessed Sacrament – Oamaru stone with poured concrete

(c) St. Luke's Anglican Church – stone front façade with clay bricks back layers

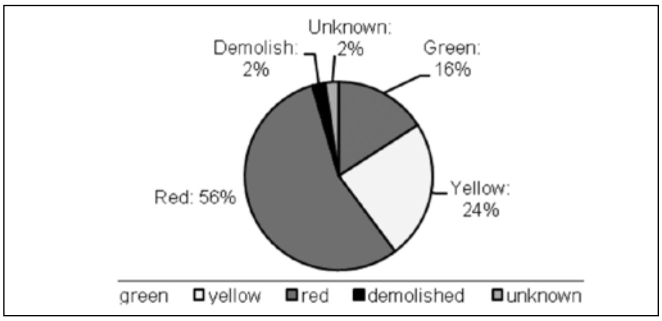
Figure 1: Representative examples of wall cross-sections for Christchurch stone masonry buildings

2 POST-EARTHQUAKE ASSESSMENT AND BUILDING DAMAGE STATISTICS

The seismic performance of stone masonry buildings was partially identified by considering the safety assessment data that was collected following the earthquakes that occurred in September 2010 and February 2011. Figure 2(a) and Figure 2(b) show the different percentages of building safety assessments after the 4 September 2010 and 22 February 2011 earthquakes, respectively. From these figures it can be seen that there was a significant escalation of damage due to the continuing earthquake activity in the Christchurch region. Figure 3 gives a further breakdown of this data for the two major earthquakes on the basis of building usage. Green placards were assigned to structures that were deemed to be safe to re-enter and required no further intervention; yellow placards were applied to buildings whose accessibility was restricted due to minor damage; and red placards were applied to buildings that were considered unsafe and likely to have a moderate to severe level of damage. At the time of the study reported here, several buildings had been demolished already because of the hazard associated with their damage state.

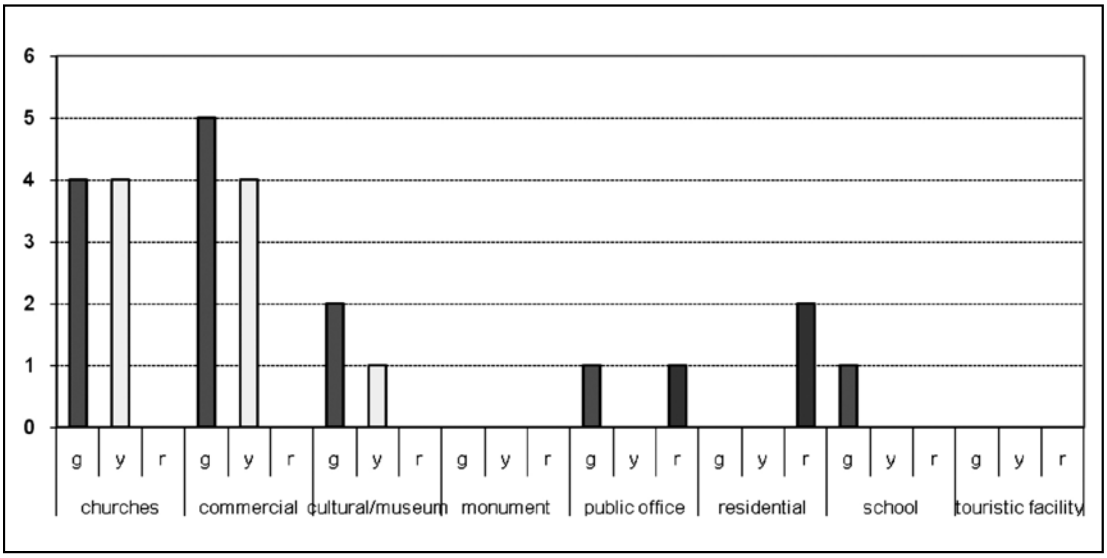


(a) *After September 2010*

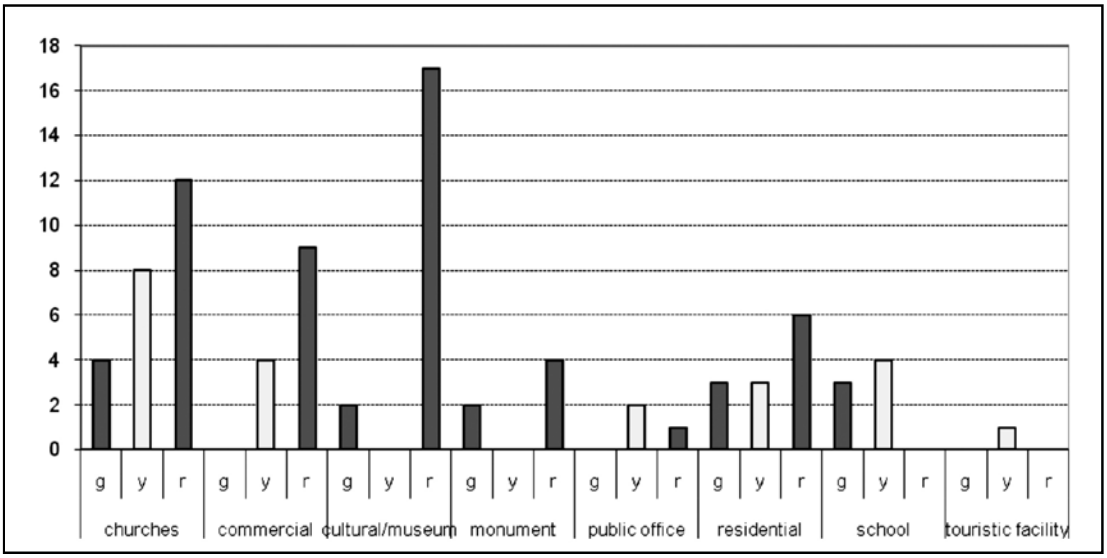


(b) *Data updated 07 June 2011*

Figure 2: Distribution of safety evaluation placarding applied to stone masonry buildings



(a) After September 2010



(a) After February 2011

Figure 3: Distribution of safety evaluation placarding applied to stone masonry buildings differentiated by building usage

(a) Unstable front wall (prior to June 13)

(b) Return wall separation

Figure 4: Christchurch Anglican Cathedral – front façade damage

3 DAMAGE MECHANISMS IN STONE MASONRY BUILDINGS AND CHURCHES

Many examples of earthquake induced damage mechanisms to stone masonry buildings were observed, with a detailed description of the most recurrent mechanisms presented below.

3.1 OUT-OF-PLANE FAILURE MECHANISMS

As expected for buildings having architectural features typical of the Gothic Revival style (long span façades, flexible floor diaphragms and weak connections between walls), partial or global overturning or instability of the façades was reported for most of the structures inspected, with damage ranging from moderate to severe and in some cases reaching collapse. Examples are shown in Figure 4 to Figure 6 relative to the main façade of the Anglican Cathedral (now partially collapsed after the 13 June 2011 earthquake and aftershocks), the Rockvilla dwelling that experienced complete collapse of the north and east façades, and the former Old Boy's High building in which the north façade was propped to avoid collapse due to out-of-plane failure. All of these buildings appeared to have poor connections between the walls at their corners, leading to return wall separation and subsequent out-of-plane failure of entire walls, as in the case of the Rockvilla house (see Figure 5).

Figure 5: Rockvilla dwelling with complete collapse of the north and east façades

Figure 6: Christchurch Arts Centre (former Old Boy's High building), with severe damage due to instability of the façade at the second storey

Many of the stone masonry buildings that were constructed in the Gothic Revival style sustained partial damage to their gable ends, with many cases of complete collapse of the gable. The absence of significant gravity loads and inadequate connection between the gable and roof trusses are primary contributing factors to this failure mode, along with increased accelerations experienced at the top levels of the structure (see Figure 7).

Figure 7: Cramner Court, showing complete collapse of a gable

3.2 IN-PLANE RESPONSE OF WALLS

Because the 22 February 2011 earthquake was predominate in the east-west direction, and because many of the buildings in the CBD are primarily oriented in the same direction, evidence of in-plane wall damage in the east-west running walls (see Figure 8 and Figure 9) was reported in conjunction with overturning of façades oriented in the orthogonal direction (see Figure 4).

Figure 8: Christchurch Anglican Cathedral – diagonal cracks in the south façade piers

Figure 9: Canterbury Provincial Chambers – diagonal crack through entire south façade of the east annex

3.3 DAMAGE DUE TO GEOMETRIC IRREGULARITIES

Damage that was attributable to plan irregularity was frequently observed, particularly for stone churches, due to interaction between adjacent structural elements at the intersections between walls. In most churches where the bell tower or low annexes are connected to the nave, damage developed at the intersection of the different structures (see Figure 10 and Figure 11).

(a) Interior view *(b) Exterior view*

Figure 10: St. Barnabas' Church, showing interaction between the nave and the bell tower

Figure 11: St. Mary's Anglican Church – detachment of the bell tower from the nave

Another distinct example of damage due to plan irregularity in association with differential foundation settlement was observed at the former Old Boy's High building. Figure 12 shows the vertical crack that formed at the intersection between two buildings constructed in successive phases, attributable to the lack of connectivity between the structural walls and to their separate foundations.

(a) Distant view *(b) Close up view*

Figure 12: Interior views of Old Boy's High, showing interaction between adjacent buildings

3.4 DIAPHRAGM AND ROOF SEISMIC RESPONSE

The influence of both inadequate and adequate securing of walls and diaphragms using wall-diaphragm anchors was observed. In some cases anchors were either absent or were spaced too far apart to prevent bed joint shear failure of the masonry at the location of the anchorage. In those cases where anchoring had been seismically designed, or sufficiently closely spaced to resist lateral loads, the overturning of gables and other portions of walls was prevented.

(a) Overturning of the front façade gable

Figure 13: Former Trinity Church, showing details of gable ended out-of-plane wall failure (continued)

(b) Detail of failed wall-to-roof anchorage

Figure 13: Former Trinity Church, showing details of gable ended out-of-plane wall failure (concluded)

Two cases are presented to show the different behaviour induced by the presence and effectiveness of anchoring. Figure 13(a) shows the damage resulting from overturning of the gable of the main façade of the former Trinity Church in the Christchurch CBD while the detail in Figure 13(b) illustrates how the anchoring was insufficient in size and spacing to secure the wall in place. Figure 14 shows some examples of successful wall-to-roof anchoring in the Arts Centre building.

(a) Former Old Girl's High

(b) Former Canterbury Engineering Department
Figure 14: The Christchurch Arts Centre, showing successful use of wall-diaphragm anchorages

In the case of churches, pounding of roof trusses was reported as for the case of St. James' Church shown in Figure 15.

(a) Damage due to pounding of roof truss

(b) Horizontal crack above arched window due to pounding of roof trusses

Figure 15: St James' Church, showing pounding of roofing elements on the walls of the nave

3.5 DAMAGE INDUCED BY POOR QUALITY OF CONSTRUCTION MATERIALS

The quality of construction materials played a key role in the response of stone URM buildings. As previously described, one of the typical features of stone URM buildings in Christchurch is the different types of stone and mortar quality present in structures built with three-leaf walls. The use of soft limestone, such as Oamaru stone or the red tuff extracted in the Banks Peninsula, in conjunction with the use of low strength lime mortar, often lead to poor earthquake response. Examples of such behaviour include the Holy Trinity Church in Lyttelton, which is one of the oldest buildings in Canterbury, and St. John's the Baptist church and the Timeball Station, as represented in Figure 16 to Figure 18.

Figure 16: Lyttelton Holy Trinity Church showing damage induced by hammering of the roof

Figure 17: St. John's Baptist Church showing local collapse of the stone masonry walls

It was reported that after the 13th June 2011 earthquakes, the remaining parts of these two buildings, and several others in Lyttelton that were in a similar state of damage, completely collapsed.

(a) Out-of-plane wall failure

(b) Damage to stone tower

Figure 18: Timeball Station. Damage in the Timeball tower

4 RETROFIT INTERVENTIONS

4.1 GENERAL PRINCIPLES AND SUGGESTED PROCEDURES

The observed poor seismic performance of unreinforced stone masonry buildings in Christchurch is a reminder of the necessity of retrofitting heritage buildings in an earthquake prone country such as New Zealand. Suggestions for appropriate strengthening principles and techniques should be gathered from the experiences accumulated by researchers and practitioners in other seismic areas of the world having stone masonry buildings with similar characteristics, such as European countries.

Retrofit interventions should be aimed at improving the performance of the structure as a complete entity, by elimination or significantly reducing structural deficiencies associated with design and execution errors, and deterioration and damage. Issues relating to both the vulnerability and the suitability of retrofit interventions should be accounted for, with particular attention given to the effects of variations in stiffness between elements and the stiffness changes associated with various retrofit techniques. Strengthening interventions should enhance the global behaviour of the structure and also the performance of isolated structural elements, and should seek to keep loads well distributed so that elevated stress levels are avoided. Where necessary, interventions should address the possibility of rocking and over-turning instability, and should support a clearly defined load path through in-plane shear walls. Furthermore, repair and retrofitting techniques should respect the original structure in order to avoid incompatibility with the original structure and materials.

Interventions should be regular and uniformly distributed on the structure. The execution of strengthening interventions on isolated parts of the building must be accurately evaluated (with the aim of reducing or eliminating vulnerable elements and structural irregularity) and justified by calculating the effect in terms of the modified stiffness distribution.

Particular attention should be given to correct implementation of the intervention strategy, as poor execution can cause deterioration of masonry characteristics or worsening of the global behaviour of the building, reducing the global ductility capacity. Some examples of the performance of retrofit interventions are described in Binda et al. (1994), Vintzileou et al. (1995), Valluzzi et al. (2004), Valluzzi (2007) and Augenti & Parisi (2010).

Different types of interventions are suggested in well known Building Codes and Guidelines (such as EC 8 (2005), NTC (2008), ASCE (2006) or FEMA 547 (2006). These intervention types can be distinguished as follows:

- Improvement of connections (walls and floors) by introducing anchoring ties, reinforcing ring beams and floor-to-wall connections (FEMA 547, 2006);

- Improvement of the behaviour of arches and vaults, installing ties and extrados metallic elements, or applying composite materials;
- Reduction of excessive floor deformability (in-plane and flexural stiffening with dry techniques, extrados intervention with boarding, steel or Fibre Reinforced Polymer (FRP) straps; bracing or other interventions at the intrados);
- Improvement of the roof or floor structures and the load transfer fixings into the supporting walls;
- Strengthening masonry walls, by local rebuilding of the walls, by grout injections, application of anti-expulsive tie-rods (such as helical wall ties and anchoring systems), repointing of the mortar joints (reinforced repointing (Borri et al., 2008), jacketing, insertion of artificial through-stones, application of transverse tying (Dolce et al., 2001);
- Improvement of pillars and columns, through measures such as circumferential hoops and reinforced injections;
- Improvement of the connection of non-structural elements.

4.2 MATERIAL STABILISATION

Good quality masonry is essential for adequate seismic response of a masonry structure. Deterioration because of aging and environmental agents results in local failures, which affects the overall effectiveness of the building. Local rebuilding of portions of masonry, repointing of mortar bed joints, or grout injections are the most commonly used techniques for material stabilisation.

4.2.1 GROUT INJECTION

Grout injection is a suitable technique for the case of three–leaf stone walls having distributed cracks, a high void ratio and sufficient porosity. Mortars with suitable fluidity are required so that all voids are properly filled. This technique should be used to:

- Fill large and small voids and cracks, thereby increasing the continuity of the masonry and hence its strength;
- Fill the gaps between two or more leaves of a wall, when they are badly connected.

In order to achieve the above aims it is necessary to accurately know details of the materials constituting the wall, and their composition (in order to avoid chemical and physical incompatibility with the grout), the crack distribution and connectivity, and the size, percentage and distribution of voids. Multiple leaf walls may be constructed with poor mortars and stones but may have a low percentage of voids (less than 4% of voids is not injectable) and an internal filling with loose material which is not injectable.

Prior to injection, loose materials such as unbonded masonry mortar should be removed and replaced, and the surface projection of internal cracks should be temporarily sealed to prevent grout from leaking. All crack and void cavities should be thoroughly flushed with clean water to remove as much dirt, debris and contaminants as possible and to pre-saturate the areas that are to be grouted. The diameter and spacing of injection holes shall be determined during the initial site investigation. Grout injection will proceed within specific repair areas, commencing from the base of the repair area to the top, moving first across the wall horizontally and then upward.

4.3 ENHANCING THE PERFORMANCE OF EXISTING WALLS

Most URM walls are required to transfer some degree of shear loading along their length. If a building has insufficient shear capacity in a particular direction, the capacity of existing walls may be increased instead of inserting additional structure (Goodwin et al., 2010). Techniques to enhance the capacity of existing stone masonry walls are described below.

4.3.1 JACKETING

Jacketing with fibre reinforced fabrics or plates is normally used in the case of regularly coursed brick or block masonry, rather than with undressed stone and rubble masonry. This intervention technique is most effective when executed on both sides of the wall but by itself does not guarantee transverse connection through the wall cross-section and therefore its effectiveness has to be carefully evaluated. For stone masonry walls jacketing should be considered for cases where stone walls lack significant artistic value but have badly deteriorated with widespread damage.

A technique that has been widely applied in Italy to irregular multiple leaf stone masonry walls and is recommended by the Italian Code (NTC, 2008) consists of positioning a reinforcing net (with bar diameters ranging from 6 mm to 8 mm) on both faces of a wall, connected by frequent transversal steel ties, and then applying a cement mortar based rendering (see Figure 19). The same technique can be undertaken to connect load-bearing and shear walls and to close large cracks. However, due to the non-homogeneity of irregular stone masonry walls, and the cost and difficulty of connecting the two outer faces, execution of this technique on site is not easy and the retrofit may be inefficient (Valluzzi et al, 2002).

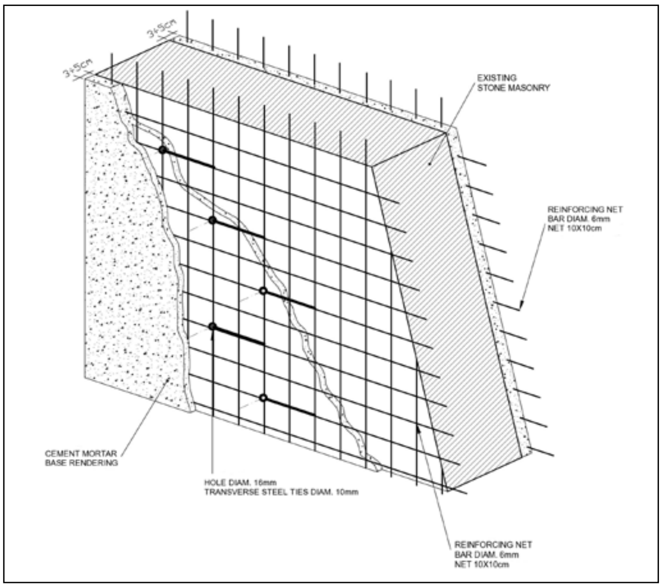
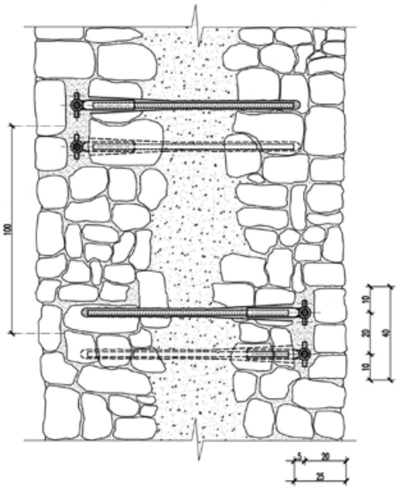


Figure 19: Schematic diagram of the jacketing technique (Mariani, 2006)

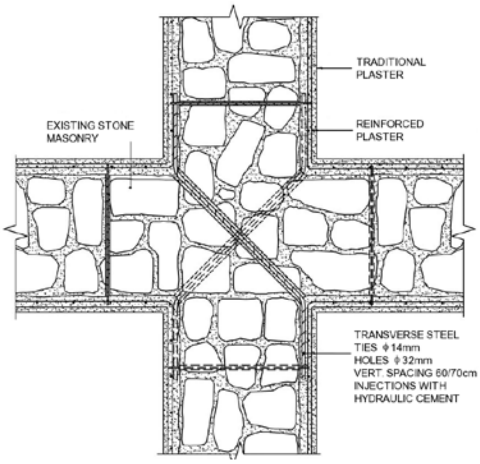
Because this technique substantially increases wall stiffness, partial interventions could be detrimental to overall structural response and so variations in stiffness should be accounted for when evaluating the response of the retrofitted structure. Furthermore, for effective performance using this retrofit technique it is necessary to ensure that there is adequate transverse connection across the wall cross-section, by providing suitable overlapping of the reinforcement net at the corners. Also, it is important to ensure that oxidation of the steel nets is avoided by providing a sufficient cover to reinforcement and to control water ingress.

4.3.2 REINFORCED INJECTIONS AND TRANSVERSE TYING

Reinforced injections are suggested for buildings having façades with a particular artistic and historical value, as the technique is less invasive than jacketing. Reinforced injections are designed to both strengthen structural elements (by acting as header units connecting the different stone and rubble leaves) or providing proper interlocking at corners or between orthogonal walls and connecting damaged parts of the wall that are separated by cracks (see Figure 20). Compressive and shear strength of structural elements are increased, as well as the 'box-type' structural behaviour of the masonry building.



(a) Reinforced injections to connect outer layers of a three–leaf stone masonry wall



(b) Transverse tying at the intersection of two perpendicular walls

Figure 20. Reinforced injection and transverse tying of stone masonry walls (Mariani, 2006)

Patented anchoring systems are available that involve pre-drilling an oversized hole in the stone masonry structure and inserting an anchoring element surrounded by a fabric sock. A cementitious grout is injected through the middle of the anchor under low pressure. This grout passes through a series of grout flood holes into the fabric sock, insuring that the anchor conforms to all voids in the hole, hence creating a mechanical bond that locks the anchor in place.

A further improvement of the technique has been obtained with the introduction of stainless-steel helical ties, used to anchor building façades to structural members or to stabilize multiple-leaf block walls. The helical design allows the tie to be screw-driven into a pre-drilled pilot hole to provide a mechanical connection between a masonry façade and its backup material, or between multiple leaves. As it is driven, the fins of the tie undercut the masonry to provide an expansion-free anchorage that will withstand tension and compression loads.

Dolce et al. (2001) have reported on a patented three-dimensional tying system made of pre-tensioned stainless steel ribbons, so that a beneficial light pre-compression state is applied to the masonry. Using special connection elements, a continuous horizontal and vertical tie system is realised, that improves the shear and bending in-plane and out-of-plane strengths of single panels and entire walls.

4.4 IMPROVING THE GLOBAL RESPONSE OF STONE MASONRY STRUCTURES

The global response of a stone masonry structure can be improved by ensuring proper connection between the walls and sufficient redistribution or forces from diaphragms. This objective may be achieved through the insertion of steel tie rods or ring beams at floor or roof level, that will contribute to the development of 'box type' response of the masonry building.

4.4.1 STEEL TIE RODS

Steel tie rods are the most common and most ancient retrofit solution in Italy, and in general in the south of Europe. If properly designed in terms of pre- and post-tension force, tie rods allow a better connection between orthogonal walls, ensuring a global type of structural response. Tie rods are also installed to improve the connection between floor diaphragms and walls, and also to reduce the thrust action from vaults and roof trusses (Binda et al., 1997).

As recommended by the Italian Code (NTC, 2008), steel tie rods should be anchored to the wall through steel plates, with appropriate dimensions to allow the correct distribution of stresses to the wall. To ensure adequate anchoring, any loose plaster at the location of drill holes should be removed and where required, the quality of the masonry should be enhanced through adequate strengthening techniques, such as reinforced injections.

4.4.2 RING BEAMS

Ring beams (or tie-beams) at the summit of masonry walls (see Figure 21) are an effective solution to connect the walls in a region where the masonry is less cohesive due to the limited amount of compression from gravity load, and to improve the interaction of the wall and roof. In contrast, ring beams at intermediate levels within the thickness of the wall are to be avoided (especially if the wall is constructed of random rubble masonry), given the negative effects that openings in rubble can produce on the stress distribution of the wall leaves.

The ring beam or tie beam type of intervention has been commonly applied in Italy in past decades. However, the Umbria-Marche earthquake in 1997 (Corradi et al., 2002) demonstrated that excessive burdening and stiffening can cause concentration of tensile stresses that are detrimental to structural performance, particularly when using reinforced concrete ring beams (Mariani, 2006). Hence, the design of ring beams should account for both connectivity between walls and the seismic loads on horizontal and vertical structural elements to ensure adequate global structural seismic response.

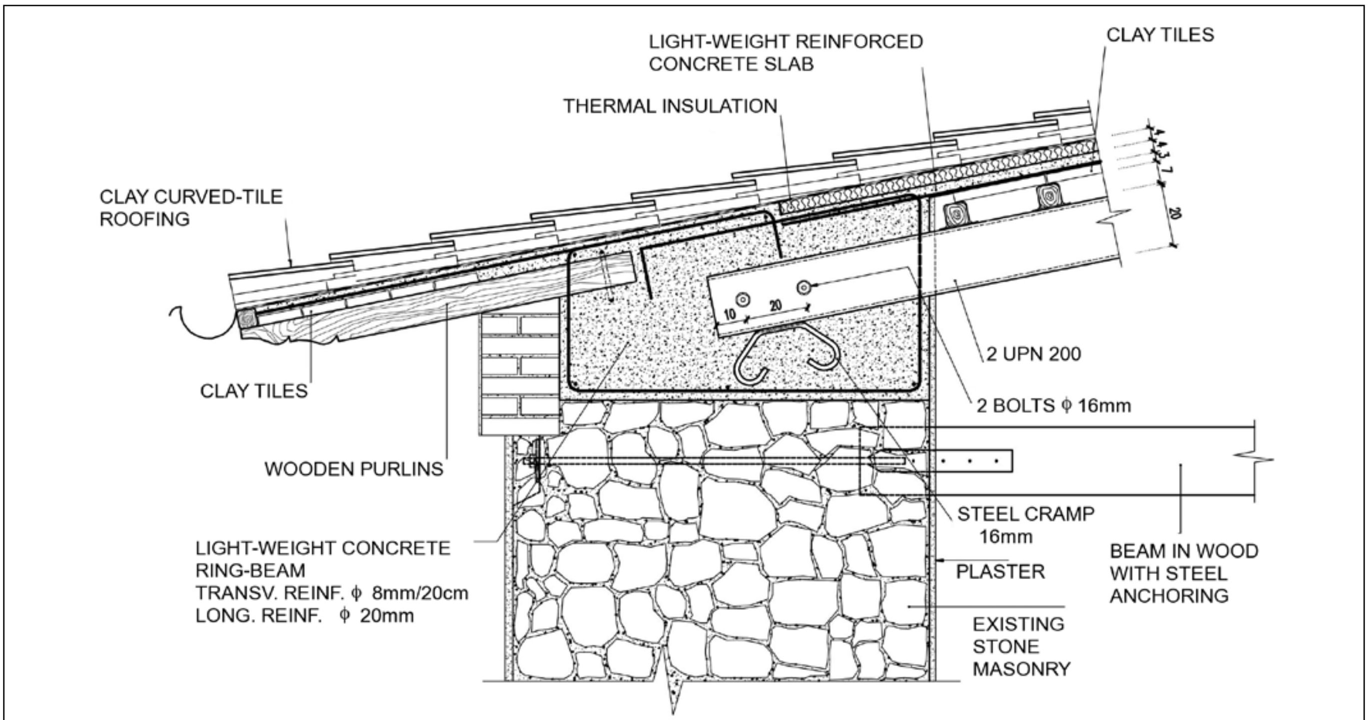


Figure 21: Details of a reinforced light-weight concrete ring beam at roof level and wall-to-floor anchoring (Mariani, 2006)

4.4.3 POST-TENSIONING

Post-tensioning is not a common retrofit technique in Europe, although in New Zealand there were several examples of good seismic performance of unbounded post-tensioned stone masonry construction during the 2010/2011 Canterbury earthquake swarm, as in the case of the Great Hall and the Chemistry Department buildings of the former Canterbury University in Christchurch (see Figure 22).

(a) View of horizontal and vertical unbonded post-tensioning

(b) Detail of tendon anchorage

Figure 22: External unbounded post-tensioning of stone masonry building

Insertion of post-tensioned vertical tie-rods is an intervention that is applicable only in particular cases and only when the masonry is capable of supporting the increase in vertical stresses, both locally and globally, at the anchorage points. The initial loss of tension due to the deferred deformation in masonry has to be taken into consideration and time dependent losses due to masonry creep and tendon relaxation effects should also be accounted for.

5 CONCLUDING REMARKS

Damage assessment of unreinforced stone masonry buildings in Christchurch was performed between April and May 2011 and consequently the presented description of their seismic response is relative to that period. Following the 13th June 2011 earthquake and successive aftershocks, the conditions of damaged heritage stone masonry buildings continued to deteriorate, with more cases of partial or complete collapse, as for the Time Ball station in Lyttelton and the Rose Window of the Anglican Cathedral. Hence, the importance of earthquake strengthening New Zealand's heritage masonry architecture to preserve a key element of the nation's history continues to be emphasised.

A number of strengthening techniques have been presented, and when implemented in conjunction with a correct reading of the architectural character and structural behaviour of a masonry building, these techniques should lead to an appropriate retrofit strategy.

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