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Senaldi, I., Magenes, G., & Ingham, J. M. (2015). Damage Assessment of Unreinforced Stone Masonry Buildings After the 2010-2011 Canterbury Earthquakes. *International Journal of Architectural Heritage*, 9(5), 605-627. doi: [10.1080/15583058.2013.840688](https://doi.org/10.1080/15583058.2013.840688)

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<http://www.tandfonline.com/doi/full/10.1080/15583058.2013.840688>

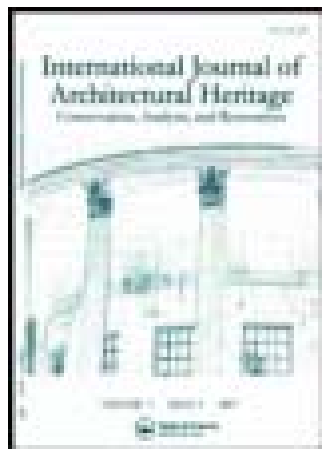
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International Journal of Architectural Heritage: Conservation, Analysis, and Restoration

Publication details, including instructions for authors and subscription information:

<http://www.tandfonline.com/loi/uarc20>

Damage Assessment of Unreinforced Stone Masonry Buildings After the 2010-2011 Canterbury Earthquakes

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Accepted author version posted online: 10 Nov 2014.



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To cite this article: Ilaria Senaldi, Guido Magenes & Jason M. Ingham (2014): Damage Assessment of Unreinforced Stone Masonry Buildings After the 2010-2011 Canterbury Earthquakes, *International Journal of Architectural Heritage: Conservation, Analysis, and Restoration*, DOI: [10.1080/15583058.2013.840688](https://doi.org/10.1080/15583058.2013.840688)

To link to this article: <http://dx.doi.org/10.1080/15583058.2013.840688>

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DAMAGE ASSESSMENT OF UNREINFORCED STONE MASONRY BUILDINGS AFTER THE 2010- 2011 CANTERBURY EARTHQUAKES

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ABSTRACT

The sequence of earthquakes that has affected Christchurch and Canterbury since September 2010 has caused damage to a great number of buildings of all construction types. Following post-event damage surveys performed between April 2011 and June 2011, an inventory of the stone masonry buildings in Christchurch and surrounding areas was carried out in order to

assemble a database containing the characteristic features of the building stock, as a basis for studying the vulnerability factors that might have influenced the seismic performance of the stone masonry building stock during the Canterbury earthquake sequence. The damage suffered by unreinforced stone masonry buildings is reported and different types of observed failures are described using a specific survey procedure currently in use in Italy. The observed performance of seismic retrofit interventions applied to stone masonry buildings is also described, as an understanding of the seismic response of these interventions is of fundamental importance for assessing the utility of such strengthening techniques when applied to unreinforced stone masonry structures.

Keywords: *Unreinforced stone masonry, seismic response, Canterbury earthquake, damage mechanisms*

1. INTRODUCTION

Commencing on 4 September 2010, the Canterbury region and the city of Christchurch were stricken by an intense and damaging seismic sequence that caused structural damage to a significant number of engineered buildings of all construction types. Damage was particularly extensive in unreinforced masonry (URM) buildings that had typically been designed with little or no consideration given to earthquake effects.

An international team of researchers was assembled during March 2011, to document and interpret the observed earthquake damage to masonry buildings by investigating and cataloguing the failure patterns and collapse mechanisms commonly encountered in the masonry building

stock of Christchurch. The initiative was undertaken as part of the “Project Masonry” Recovery Project (Dizhur et al., 2011; Moon et al., 2013) funded by the New Zealand Natural Hazards Research Platform, with particular emphasis given to unreinforced clay brick and stone masonry buildings because it was recognized that a large proportion of New Zealand’s older building inventory nationwide is of URM construction, built prior to the introduction of mandatory earthquake design requirements. Consequently, observations on the performance of this building class in the Canterbury earthquakes were of direct relevance when seeking to understand the earthquake vulnerability of similar buildings located both elsewhere in New Zealand, and in other countries such as Australia and West Coast North America that have a comparable heritage building stock.

Part of Project Masonry was specifically devoted to studying the seismic performance of unreinforced and earthquake retrofitted stone masonry buildings in Christchurch. Between April 2011 and June 2011 a database of all the stone masonry buildings in Christchurch and on the Banks Peninsula was assembled. Because of their particular importance as part of New Zealand’s architectural and historical heritage, most of these stone buildings are listed in the Register of the New Zealand Historic Places Trust (New Zealand Historic Places Trust, 2012a) and in the Heritage Items List of Christchurch City Council (Christchurch City Council, 2012). However, neither list includes all the stone masonry buildings in Christchurch and surrounding districts, nor provides sufficient details on the structural characteristics of the buildings. A first attempt to catalogue the stone masonry buildings in several cities in New Zealand was made by Hayward (1987), who compiled a list of 31 of buildings in the Christchurch Central Business District (CBD), in which stone was used either for construction or as a decoration material. The database

of stone masonry was compiled by combining Hayward's list of buildings with data available from the NZHPT Register and the Heritage Items List of Christchurch City Council, while the work by Hamilton and Hamilton (2008) allowed the identification of stone masonry churches in Christchurch and vicinity.

In addition to identifying Christchurch stone masonry buildings and describing their typical features, the main purpose for assembling the detailed database was to quantify the impact of the Canterbury earthquakes on the heritage stone masonry building stock and to characterise the failure modes and damage mechanisms observed after the 22 February 2011 Christchurch earthquake. This damage assessment was based on the analysis of data collected both by New Zealand authorities undertaking building safety evaluations and by developing an inventory of damage mechanisms that occurred through on-site damage inspections of churches and buildings using the procedure standardized by the Italian Civil Protection Department. The most recurrent damage mechanisms experienced by Christchurch buildings in the September 2010 and February 2011 earthquakes are described below.

2. THE 2010-2011 CANTERBURY EARTHQUAKE SEQUENCE

The Canterbury Region and Christchurch city in particular are considered to be a moderate seismic hazard area, as specified in the NZ Loadings Standard (SNZ, 2004). Since the time of European settlement in Canterbury, seismic activity in the region was first recorded in 1853 with approximately ten events having since occurred and with three earthquakes of high intensity (the

MM7 June 1869 Christchurch earthquake, the MM6 August 1870 event at Lake Ellesmere and the MM9 September 1888 Amuri District earthquake (Geonet, 2012; Christchurch City Libraries, 2006)). However, despite the seismic activity in the South Island during the 19th century and the occurrence of the 1931 Hawke's Bay earthquake in the North Island of New Zealand (Dowrick, 1998), in the Canterbury Plains there was no evidence of fault activity, with any existing faults hidden by the river gravel layers of the Plains.

The Canterbury earthquake sequence started on 4 September 2010 (M_w 7.1) and was followed by a significant sequence of aftershocks with magnitude M_w 3 or greater. Besides the 4 September 2010 Darfield earthquake, four other severe events with a magnitude of approximately M_w 6 occurred on 22 February 2011 (M_w 6.3), 13 June 2011 (M_w 6.2) and 23 December 2011 (M_w 6.0). The four earthquakes having a magnitude greater than M_w 6 are represented in Figure 1. The proximity of the rupture fault to central Christchurch was one of the main factors contributing to the extent of damage and fatalities that occurred, particularly on 22 February 2011.

The magnitude M_w 7.1 earthquake occurred at 4:35am on 4 September 2010, revealing the existence of the hidden trace of the Greendale fault (the epicentre was at 37 km west of Christchurch, close to Darfield) (Allen et al., 2010; Gledhill et al. 2010; Geonet, 2010). Extensive damage was caused to lifelines, bridge abutments and residential houses due to the widespread liquefaction and lateral spreading in areas close to major streams, rivers and wetlands through the region, in particular in the eastern suburbs of Christchurch along the Avon River (Allen et al., 2010).

The magnitude M_w 6.3 event of 22 February 2011 occurred at 12:51pm, and was the most damaging aftershock among the almost 1500 events having a magnitude greater than M_w 3 that followed the September 2010 earthquake. The earthquake-induced ground shaking caused widespread and severe liquefaction in native soils throughout Christchurch, and numerous rock falls and slope failures in the Port Hills, all of which resulted in substantial damage to the city's infrastructure because of the short epicentral distance of the event (only 10 km south-east of Christchurch) and its shallow depth (5 km) (Bradley and Cubrinovski, 2011; Geonet, 2011). The earthquake caused 185 fatalities and a similar number of people were severely injured (Cubrinovski et al., 2011; NZ Police, 2012).

3. CHARACTERISTICS OF THE UNREINFORCED STONE MASONRY BUILDINGS OF CHRISTCHURCH

3.1. Location, function and history of stone masonry buildings in Christchurch

The assembled unreinforced stone masonry buildings database consists of 96 buildings located mainly in Christchurch city and the surrounding suburbs. As shown in Figure 2, the inspected buildings are mainly located in central Christchurch, with stone masonry churches also present on the Banks Peninsula and in the nearby towns of Ashburton, Hororata and Leeston.

Immediately prior to September 2010, and in many cases since their construction, the use of stone buildings in Christchurch is mainly devoted to public functions rather than the housing of private businesses. As shown in Figure 3 (left), only 13% of Christchurch stone masonry buildings are used as residential dwellings, while more than 70% are either schools and cultural institution or public offices, mostly located in the CBD, or are churches.

In the early period of New Zealand colonisation most buildings used for commercial and residential purposes were constructed of timber. However, following a number of severe fires and substantial resulting damage to timber structures, brick and stone masonry became the most commonly used construction techniques for churches, public buildings, universities and schools throughout New Zealand from the second half of the 19th century.

The stone masonry buildings in Christchurch were constructed in a relatively short period of time, between 1850 and 1940 (see Figure 3, right), as identified using data extracted from the New Zealand Historic Places Trust database (New Zealand Historic Places Trust, 2012a). The most iconic Christchurch stone masonry buildings were constructed between 1858 and 1870, including the Canterbury Provincial Council Buildings whose construction commenced in 1858 and was completed in 1864, the former Canterbury University College (now referred to as the Christchurch Arts Centre) that was constructed in 1862, and the Christchurch Anglican Cathedral whose cornerstone was laid in 1864 (New Zealand Historic Places Trust, 2012a; 2012b).

3.2. Architectural features

Most stone masonry buildings in Christchurch have similar characteristics, both from an architectural and from a structural perspective. Many notable buildings, such as the Anglican Cathedral and the Christchurch Arts Centre, followed the architectural principles of the Gothic Revival style, which was in vogue in Christchurch until the 1880s and distinguished the city from elsewhere in New Zealand where buildings were more commonly being constructed in the Classical and Renaissance style. Similarities among the stone masonry buildings are also due to the fact that many were designed by the same architects or architectural firms, such as Cecil Woods, John Goddard Collins and Benjamin Mountfort (Lochhead, 2010).

The vast majority of the stone masonry buildings, and in particular those buildings constructed in the Gothic Revival style, are characterized by structural peripheral unreinforced masonry walls that may be connected, depending on the size of the building, to internal masonry walls that support flexible timber floor diaphragms and timber roof trusses or are connected to an internal frame structure constituted of cast iron or steel columns and timber beams. However, there are a few commercial buildings in the Christchurch 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. The Christchurch stone masonry building stock is constituted of relatively low-rise structures, with not more than three storeys.

3.3. Characteristics of the masonry stonework

The proximity of Christchurch to the Banks Peninsula, and the scarcity of nearby woods, resulted in a considerable variety of volcanic rocks being employed as construction material for Christchurch stone masonry buildings (Hayward, 1987), and in particular tuff and basalt came from the Lyttelton Volcano that is part of the Port Hills. Tuff was employed for example in the Time Ball Station (that was subsequently covered with stucco) in Lyttelton and in the towers of the Provincial Chambers in Christchurch, while Port Hills basalt was used in the construction of numerous Christchurch buildings in the 1880s, such as the Canterbury Museum and some of the buildings of the Christchurch Arts Centre.

Two other types of basalt rock were extracted from Port Hills quarries, being Halswell Basalt and Hoon Hay Basalt, with the former being the dominant stone for paving and buildings constructed since the mid-1860s (Hayward, 1987), like the Provincial Council Chamber.

Building stones of sedimentary origins are widely distributed throughout New Zealand, but the majority are not suitable for construction because of their rapid decay when exposed to weather. Oamaru Limestone, nonetheless, was not only mostly used as decorative quoins and facing stones around windows and doors in conjunction with grey volcanic basalt, as in the case of the Christchurch Anglican Cathedral and the Arts Centre buildings, but also in multi-leaf structural walls, on one or either side of a wall, as in the Catholic Cathedral of the Blessed Sacrament in Christchurch.

Several types of masonry stonework were identified, as well as the different construction materials used, from observation of wall cross-sections exposed because of damage experienced during the Canterbury earthquake sequence. The characteristics of the masonry are described in Table 1. For 25% of the buildings only the type of facing stone is indicated, as most of the damage assessment surveys took place during the state of emergency and hence were undertaken from outside of the buildings only, due to safety issues.

The most representative stone masonry wall types are typically multiple-leaf walls with rubble or ashlar stonework on the façade, either randomly or regularly coursed, with the internal core consisting of stone rubble fill, in particular for the early buildings in Christchurch. Because of the cost of construction, multiple-leaf ashlar masonry was often substituted by a stone ashlar veneer used as facing, being backed with either brickwork or rubble masonry. In some cases between the stone veneering and the stone or brickwork backing, an internal poured concrete core or a cavity between the brick layers was present. In Figure 4 some of the most representative examples of wall cross-sections are presented.

4. BUILDING SAFETY EVALUATIONS AFTER THE 22 FEBRUARY 2011 CHRISTCHURCH EARTHQUAKE

4.1. Post-event building safety evaluations procedures

After the earthquakes that occurred in September 2010 and February 2011 the building safety evaluation process was activated as prescribed by New Zealand's Civil Defence Management Act (2002) during a state of emergency, following the "Building Safety Evaluation during a State of Emergency" procedure developed during the 2009 NZSEE Learning from Earthquakes mission to Padang, Indonesia (Brunsdon et al., 2010).

As a result of the building safety evaluations, placards were assigned to each building with colours used to identify the building's usability. 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 attributed to buildings that were considered unsafe and likely to have a moderate to severe level of damage.

4.2. Analysis of the building safety evaluation results

The seismic performance of stone masonry buildings was initially identified by considering the safety assessment data collected during the building safety evaluation process. Figure 5a and Figure 5b show the different percentages of building safety assessments after the 4 September 2010 and 22 February 2011 earthquakes, respectively. Although only 51% of the stone masonry buildings were inspected after the September 2010 earthquake, 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, evident by the increase in proportion of red tagged buildings in comparison to that of the green or yellow tagged ones. At the time of the study reported here, several buildings had already been demolished because of the hazard associated with their damage state after the February 2011 event.

It is possible to estimate the evolution of damage based on variation of the safety evaluation placards for those buildings that were inspected both after the September 2010 and after the February 2011 earthquakes. As shown in Figure 6, the percentage of buildings considered unsafe for immediate occupancy greatly increased from 14% to 72%, including in the latter those buildings that were demolished for public safety reasons. The increment of buildings with a moderate or severe level of damage after the 22 February 2011 earthquake is, as expected, the result of the accumulation of damage because of continuing seismic activity in the Christchurch region after the September 2010 earthquake.

Since most of the stone masonry buildings in Christchurch belong to the cultural heritage of the region and are used for a variety of public functions, the analysis of the distribution of the safety

evaluation placards based on building usage helped to illustrate the great impact of the Canterbury earthquake sequence on the population of Christchurch. As pictured in Figure 7, almost all of the stone masonry buildings hosting cultural institutions were considered unsafe, as well as churches, of which only 15% were considered suitable for immediate occupancy after the 22 February event. Green placards were attributed to only 25% of residential dwellings.

5. DAMAGE ASSESSMENT OF STONE MASONRY BUILDINGS BASED ON ITALIAN PROCEDURES

5.1. Damage inspections of stone masonry buildings in Christchurch

Recognising that safety evaluation placards provide information on the usability of buildings only, and because the intention of the authors was to investigate the seismic performance of the stone masonry buildings in Christchurch and surrounding districts after the Canterbury earthquake sequence, an assessment of the damage mechanisms activated in stone masonry buildings was necessary. Damage surveys were consequently performed between the beginning of April 2011 and the first week of June 2011. The database of stone masonry buildings was compiled during these investigations, which included results of the safety evaluation process and visual inspections of the damage (inspections were undertaken both from the outside of the buildings and, when permitted, from the inside), in addition to recording the building architectural and structural features mentioned previously.

The approach followed during the damage surveys of stone masonry buildings in Christchurch was derived from the damage assessment procedures developed in Italy based on knowledge acquired during the Umbria and Marche region earthquakes in 1997 (Lagomarsino and Podestà, 2004a) and further improved after the earthquakes that occurred in Molise (2002) (Lagomarsino and Podestà, 2004b) and Garda Lake (2003), which demonstrated the recurrence of damage mechanisms in existing masonry buildings. The definition of the damage mechanisms was based on identification of the macro-elements, being architectural portions that form the masonry building (Giuffrè, 1991) and that are characterized by independent structural behaviour if compared to the global response of the building (DPCM 09/02/2011, 2011). Research on monuments, specifically churches, undertaken after the 1976 Friuli (Italy) earthquake contributed to the first catalogue of the level of damage associated with each kinematic mechanism (Doglioni et al., 1994).

The Italian Civil Protection Department, in cooperation with the Ministry of Cultural Heritage and based on the results of past research studies, defined templates for effective site collection of data from visual inspections, with the main purpose of the exercise being to not only decide whether a building is suitable for immediate occupancy and to give advice regarding the need for provisional interventions to prevent future damage due to aftershocks, but also to catalogue the most recurrent damage mechanisms observed and to study the vulnerability factors that influence the performance of buildings. The damage survey forms for cultural heritage, prepared for churches (A-DC model) and palaces (B-DP model) (Form A-DC, 2006; Form B-DP, 2006), were approved by the Italian Government with decree in 2006 (DPCM 23/02/2006, 2006) and were fully employed by authorities after the L'Aquila earthquake in 2009 (Binda et al., 2011). The

forms are organized in different sections meant to provide not only general notions about the building, such as location, ownership and function or its architectural configuration, but also allow the evaluation of both the structural and the artistic heritage damage. The use of pre-defined forms to collect information regarding the characteristics and damage state of buildings and churches has the aim of not only deciding on the suitability of their immediate occupancy but also of providing information on the need for provisional interventions to prevent further damage due to aftershocks and to estimate the cost of required interventions (Lagomarsino, 2012).

In detail, the form for churches defines 28 possible mechanisms, whereas 22 mechanisms are identified for palaces, as listed in Table 2 and Table 3. In addition, the survey forms depict both out-of-plane and in-plane damage mechanisms for each element of the analysed typology (Borri et al., 2002) (Figure 8 and Figure 9). As a matter of completeness, another mechanism has been added in Table 3 for palaces, with spandrel damage subdivided into shear damage (mechanism n.6) or flexural damage (mechanism n. 23).

For both churches and palaces the damage assessment consisted of assigning to each mechanism identified in the structure a damage grade d_k that ranges between 0 (no damage) to 5 (local or complete collapse), with values of 1 for minor damage, 2 for moderate damage, 3 for medium damage and 4 for severe damage, following the definitions in the EMS-98 proposal (Grunthal et al., 1998). The damage index I_d , which measures the average damage of a building, is evaluated according to the n possible mechanisms that may be activated and on their total level of damage

grade (sum of the damage of each of the N mechanisms considered for each building typology), as shown in Eq. 1:

$$I_d = \frac{1}{5n} \sum_{k=1}^N d_k \quad (1).$$

The value of damage index I_d ranges between 0 (undamaged condition) and 1 (total collapse of the building). Figure 10 shows that 63.2% of the stone masonry building stock, in most part palaces, had a low damage index (below 0.2) while 26.3 % had moderate damage (falling in the interval 0.2-0.4). Only six structures had index values between 0.4 and 0.6, indicating medium to high damage, whereas the remaining 4.2% were severely damaged up to complete collapse. However, as already underlined by Da Porto et al. (2012) for the case of churches assessed after the 2009 L'Aquila earthquake, because of the same weight assigned to each possible mechanism in the current definition of damage index (Eq. 1) a low I_d may represent either slight or moderate damage that affected a significant part of the building or a severe damage, up to collapse, for a limited number of mechanisms.

The average damage level $d_{av,k}$ of each mechanism was also calculated as in Eq. 2, as the sum of the total damage level scores of the k -th mechanism d_k in all the j structures (either churches or palaces):

$$d_{av,k} = \frac{\sum_{i=1}^j d_{k,i}}{j} \quad (2).$$

5.2. Activated damage mechanisms in churches and palaces

Because each stone masonry building in Christchurch was assessed following the same procedure, it is possible to evaluate the rate of occurrence of a certain activated mechanism and correlate it to the average damage level experienced and hence to determine the vulnerability of the macroelement affected by the mechanism analysed. It has been demonstrated by Leite et al. (2012) that there is a good correspondence in the case of churches between the results of the safety evaluation process and the relative attribution of safety placards, with the fitness for use (FFU) index and the damage index I_d derived from the damage assessment forms used by the Italian Civil Protection.

Damage mechanisms in churches

The most recurrent damage mechanisms in churches affected those macroelements that constitute the fundamental components of the architecture of a church. As seen in past earthquakes, for example in Italy (Lagomarsino and Podestà, 2004a; Lagomarsino and Podestà, 2004b; Lagomarsino, 2012, Da Porto et al., 2012) and other seismic prone countries like Chile (Decanini et al, 2012), façade walls are amongst the most vulnerable elements of churches and are often subjected to out-of plane overturning mechanisms either partially or completely involving the façade or top gable. Examining the case of stone masonry churches in Christchurch, the possibility of activation of a certain mechanism, depending on the structural configuration of each church, and its actual occurrence during the seismic event are depicted in Figure 11a. Some of the mechanisms, such as those related to the response of chapels

(mechanisms 22 and 23) or of vaults and domes (mechanisms 8, 9, 14, 15, 18 and 24), which are the most vulnerable macro elements in this particular building typology, have a low frequency of activation, because most of the churches in Christchurch and surrounding districts are generally small in size and are characterized by the presence of a simple nave closed by a transept and an apse, with roof pitches in wood. The Catholic Cathedral of the Blessed Sacrament is an exception, being the only stone church in Christchurch built in the Neo-classical style still existing at the time of the assessment.

Mechanisms 1 and 2 that relate to out-of-plane overturning of the façade and mechanisms 10 and 16 that pertain to out-of-plane overturning of the transept and of the apse end walls respectively were present in up to 65% of the stone churches (Figure 11b).

Other frequent mechanisms that were encountered in Christchurch stone masonry churches are associated with in-plane response of the structure, such as mechanisms 3, 6, 11, and 17 that are related to damage due to shear in the façade, in the side walls of the nave, in the transept walls and in the apse respectively. In the case of the latter three mechanisms, 64.7% of churches were affected, with more than 35% experiencing medium to serious levels of damage.

The roof covering and roof structural elements of Christchurch stone masonry churches sustained significant damage. As shown in Figure 11b, mechanisms 19, 20, and 21 which are related to the damage to structural components of the roof and to the walls because of the hammering of roof trusses and roof beams, were activated in more than 40% of churches. Local or global collapse of bell towers (mechanism 27) occurred also, with 9% of churches affected, as in the case of the Christchurch Anglican Cathedral. In conjunction with the damage to bell towers, because they

are usually located adjacent to the nave of the church or to the transept, or at the intersection between nave and transept, damage mechanisms due to the effect of plano-altimetric irregularities of the structure occurred in more than 40% of churches, causing medium to serious damage in most cases.

The average damage value d_{av} obtained for each damage mechanism listed in the A-DC form (DPCM, 2006) was correlated to actual activation of the mechanism, in order to determine the most vulnerable structural elements of churches and to define the level of damage that they experienced. In Figure 12, mechanism 2, that refers to out-of-plane overturning of the walls of the church (the gable walls of the façade in particular) is the most recurrent, together with mechanisms involving the activation of shear mechanisms (Mechanisms 6, 11, 17) with an activation exceeding 60%, and with a level of average damage close to 3.5 and 2.5 respectively, corresponding to an average medium damage. Although mechanisms 1 and 6 associated with overturning of the façade and of the apse respectively affected 30% to 40% of churches, both mechanisms had an average value close to 3.5 that indicates a medium to serious level of damage that reached collapse in some cases.

Damage mechanisms in palaces and buildings

The damage mechanisms exhibited by palaces and buildings are mostly associated with out-of-plane and in-plane response of the structural elements, as shown in Figure 13a where the percentage of actual activation of each mechanism is compared with the number of structures in which that mechanism was possible. As for churches, all the mechanisms associated with vaults and arches are characterized by an almost null percentage of activation, either because typically

the stone masonry buildings constructed in the Gothic Revival style are characterized by timber floor diaphragm with a scarcity of vaults, or because most of the stone masonry buildings were inspected from the outside only and hence, even if present, the performance of masonry vaults could not be assessed.

In Figure 13b, where the frequency of activation and the damage level of each mechanism are presented, overturning of the façades (mechanism 1) is reported for 30% of buildings, with 7% having collapsed. Because many of the buildings in Christchurch were constructed according to Gothic Revival architectural principles, and are hence characterized by the presence of gables not only in the outer transversal walls but also in the front façades as decorative elements in correspondence with openings, more than 40% of buildings experienced damage in the top portion of the façade (mechanism 17) with an occurrence of local or global collapse close to 17%.

In-plane response resulted in 45% of buildings exhibiting pier cracks due to shear (mechanism 5), with less than 10% of buildings having minor to moderate damage, while the remaining 35% of buildings sustained medium to serious damage. The relatively low percentage of buildings with shear damage in internal walls (mechanism 7), in comparison with damage due to mechanism 5, was because most inspections were performed by the authors from the outside of the buildings only, due to limited access because of safety reasons, in particular for those buildings that had been assigned a red placard. Spandrels and lintels were also subjected to damage, with mechanism 6 (damage due to shear) occurring in almost 15% of buildings and with mechanism 23 (damage due to flexure) occurring in approximately 25% of cases.

The out-of-plane overturning of projections, such as chimneys or parapets, was rather frequent, with the activation of mechanism 18 encountered in 35% of structures, sometimes causing damage to the roof covering below (mechanism 16).

The percentage of activation of the damage mechanisms was related to the average damage level quantified, as suggested in the EMS-98 proposal. As show in Figure 14, among the most frequent mechanisms in stone masonry buildings were those regarding out-of-plane response of the façades (mechanisms 1 and 17) which occurred with an average damage value ranging from 3.5 to 4 and a percentage of activation of between 30% and 40%. Mechanism 5 that is related to the in-plane response of piers had the highest occurrence, with more than 45% of buildings reaching on average a damage value of level 3, which indicates a medium level of damage due to shear in this case.

5.3. Damage observations from on-site inspections

Examples of the most significant and frequent damage mechanisms to both buildings and churches due to the 22 February 2011 earthquake are presented below. The most probable causes that facilitated activation of these mechanisms are described, taking into account vulnerability factors associated with local construction techniques and materials.

Out-of-plane mechanisms

As described previously, most of the buildings and churches in Christchurch were designed following the architectural principles of the Gothic Revival style and hence are characterized by long span façades, flexible floor diaphragms and weak connections between walls. As expected

for structures having these characteristics, it is not surprising that one of the most recurrent damage mechanisms reported for most of the structures inspected was the partial and global overturning of façades, with damage levels ranging from moderate to severe and in some cases reaching collapse, in both buildings and churches. The poor connections between the walls at their corners led to return wall separation and subsequent out-of-plane wall failure, either entirely as in the Anglican Cathedral or in the top portion of the façade as in the case of the Time Ball Station. Figure 15 (a, b) shows an example of an out-of-plane failure mechanism activated in the main façade of the Anglican Cathedral (partially collapsed after the 13 June 2011 earthquake and aftershocks).

The low level of vertical compression and inadequate connection between the gable and roof trusses are primary contributing factors to the occurrence of out-of-plane overturning of gable end walls, along with increased accelerations experienced at the top level of the structure. Many of the stone masonry buildings that were constructed in the Gothic Revival style sustained partial damage because of the activation of overturning mechanisms in the gable, with cases of local or complete collapse of the top portion of the façade, as shown in Figure 15 (c, d) in the Arts Centre and in the Rose Historic Chapel.

In cases when the façade span was rather wide and internal bearing walls exerted almost no restraining action, a horizontal arch mechanism occurred. In the example presented in Figure 15 (f), where the north-west façade of Strange's Building in High Street is pictured, the formation of a horizontal cylindrical hinge occurred along the floor level of the top storey and the complete

lack of proper connection between the perimeter wall and the roof structure resulted in a trapezoidal portion of the façade plane overturning about the cylindrical hinge.

In-plane response

Evidence of in-plane wall damage in the east-west running walls of buildings was reported in conjunction with overturning of façades oriented in the orthogonal direction, because of the directivity effect of the February 2011 Christchurch earthquake caused by a fault rupture oriented in the same east-west direction. The damage patterns typical of failure due to shear or to flexure were frequently exhibited by masonry piers and spandrels. Examples of recurrent damage patterns are shown in Figure 16, where masonry piers of the Christchurch Anglican Cathedral exhibited a shear type of response (Figure 16a, b), evident by diagonal cracks that affected the buttress, as well as the piers of Strange's Building.

Light to substantial damage to masonry spandrels was also reported, such as diagonal cracking due to shear or a flexural type of response such as for the spandrels of the south façade of Cramner Court (Figure 16, c).

Damage due to irregularities of construction material

The quality of construction materials played a key role in the response of stone masonry buildings. As mentioned previously, early stone masonry buildings in Christchurch and Lyttelton were constructed with tuff stones because of their ease of working, although they frequently deteriorated due to weathering. The use of Port Hills Tuff or Oamaru limestone, in conjunction with the use of low strength lime mortar, often led to poor earthquake response. One of the

typical features of stone masonry buildings in Christchurch was the different types of stone and mortar quality present in structures built with three-leaf walls. The scarcity of through-stones to connect the outer leaves of walls frequently contributed to the collapse of portions of wall sections. Examples of such behaviour, frequently caused by hammering of the roofing system on the walls, include the Holy Trinity Church in Lyttelton, which is one of the oldest buildings in Canterbury, and St. Cuthbert's Church as represented in Figure 17 (a, b). The Time Ball Station in Lyttelton is another example of poor earthquake performance, due to the weak mechanical properties of the local Port Hill tuff and Oamaru stone used for its construction. The lack of connection across the stone walls is shown in Figure 17 (c).

Damage due to interaction of structural sections

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 as shown in Figure 18 (a, b). Another distinct example of damage due to plan irregularity was observed at the former Old Boy's High building in the Arts Centre complex. Figure 18 (c, d) 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.

5.4. Seismic performance of retrofitted structures

One of the objectives of the damage survey was to investigate the response of stone masonry buildings that had been seismically retrofitted or strengthened prior to the September 2010 earthquake (seismic improvements were identified in a total of 32 buildings, of which only four were churches). As previously illustrated, one of the most common factors that contributes to the vulnerability of unreinforced masonry structure is the lack of connection between walls and diaphragms. The Canterbury earthquakes have confirmed that properly designed securing systems at floor and roof level helped to reduce the likelihood of local failures due to out-of-plane collapse of walls and gables. As shown in Figure 19, buildings with applied seismic improvements have been subjected to a low level of damage, with approximately 23% of the total population having I_d values between 0 and 0.2, although three strengthened churches (i.e. Christchurch Anglican Cathedral, the Catholic Cathedral of the Blessed Sacrament and St. Luke's Church) were subjected to a moderate to high level of damage with corresponding damage indices from 0.44 up to 0.64.

Several types of strengthening interventions were encountered in Christchurch stone masonry buildings, whose application had the objective to prevent or restrain the activation of out-of-plane overturning mechanisms and to enhance the global type of response of structures. Steel exterior plate anchors (in some cases applied at the time of construction) or adhesive anchors were frequently used to provide gable end wall restraint or as wall-to-diaphragm anchorages. Diaphragm improvements consisted of plywood overlays or of collaborating light-weight reinforced concrete slabs. To enhance a global type of response or even supplement the seismic

resistance of the original stone masonry structure, steel or concrete frames, additional shotcrete walls, post-tensioning or Fibre Reinforced Polymers were employed. Figure 20a shows the corresponding distribution of damage index values for each type of strengthening intervention. It is emphasised that in 13 cases only wall-to-diaphragm anchorages (sometimes connected to steel trusses or steel backing frames) were applied, while in the remaining 19 buildings more than one technique was applied to a single building, as is often the case in current practice.

In order to evaluate the actual effectiveness of the applied retrofit interventions, a comparison between the seismic performance of strengthened and unstrengthened buildings, in particular palaces, can be made. Figure 20b and Figure 20c represent the damage level reached in the mechanisms concerning the out-of-plane response of façades or of portions of walls (namely mechanisms from 1 to 4 and 17) and those describing the in-plane response of masonry structural elements (mechanisms from 5 to 8 and 23) for strengthened and unstrengthened palaces.

The higher percentage of strengthened buildings in which out-of-plane overturning mechanisms caused moderate to serious damage (60% of strengthened palaces were affected by the activation of mechanism 17 and more than 30% by mechanism 1), in comparison with the unstrengthened buildings requires clarification. Most of the strengthening interventions, particularly wall-to-diaphragm anchorages and steel bracings or trusses, were applied on buildings constructed following the principles of Neo-Gothic architecture, and hence these buildings were more prone to the activation of out-of-plane overturning mechanisms, rather than in dwelling or on buildings with simpler architecture and smaller dimensions. A close inspection of buildings that suffered out-of-plane wall failures indeed revealed that in many cases some anchors were present in the

walls that failed, or that inadequate securing of walls and diaphragms using wall-diaphragm anchors could not prevent portions of walls from overturning. 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 buildings where anchoring had been seismically designed or where anchors were sufficiently closely spaced to resist lateral loads, the overturning of gables and other portions of walls was prevented. Figure 21a 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 21b illustrates how the anchoring was insufficient in size and spacing to secure the wall in place. Figure 21 also shows some examples of wall-to-roof anchoring respectively in the Christ's College (Figure 21c) and in the former Canterbury University Engineering Department (Figure 21d). As shown in Figure 21c, the use of a backing steel frame that connects both the masonry gable wall and the timber roof structure prevented complete collapse of the gable because of out-of-plane overturning, although collapse of portions of the wall did occur. A similar solution performed successfully in the former Malthouse, now the Canterbury Children's Theatre, where in addition to grout injection of the multi-leaf stone masonry walls, the diaphragms were stiffened by adding new steel trusses adjacent to the original timber ones at roof level and by replacing the plywood of floors as well as improving the wall-to-floor connections with steel anchors.

Only a limited percentage of buildings was affected by damage due to the in-plane response of walls because of the presence of different types of strengthening techniques to enhance the global response of buildings and to restrain the activation of possible local failure mechanisms. Although 32% up to 40% of strengthened palaces were subjected at most to a medium level of

damage respectively because of flexural failure of spandrels or of shear cracks in piers in the perimeter façades (Figure 20b), a greater number of unstrengthened buildings (e.g. 65% more than the strengthened ones for mechanism 5) were damaged because of the activation of the same type of damage mechanism. The use of steel or reinforced concrete moment frames as a retrofit strategy proved to be efficient, as in the case of the former Lawrie and Wilson Auctioneers Building ($I_d=0.02$, Figure 22a) or in the former Malthouse building ($I_d=0.08$, Figure 22b). Considering the former Chemistry Department that is now part of the Christchurch Arts Centre complex, the insertion of vertical post-tensioned tendons in collaboration with buttresses and of horizontal tie rods in collaboration with floors improved the global response of the structure (Figure 22c). Nonetheless, the building was damaged because of the partial collapse of the tower that was not retrofitted.

The addition of a secondary structural system, such as moment resisting frames or reinforced concrete shear walls, was found by Ingham and Griffith (2011) to be a rather common retrofit solution adopted in unreinforced brick masonry buildings, and not only in the stone masonry cases presented previously. The good seismic performance of such interventions depended upon an appropriate design conception and realization as well as on proper connection of the secondary structural system with the original structure, as also reported by Dizhur et al. (2011) for strengthened clay brick masonry buildings. Alternative solutions were adopted recently, applying innovative techniques such as fibre reinforced polymer (FRP) sheets, steel strapping, or stainless steel rods to reinforce masonry walls, in order to avoid the large impact on the original building configuration of frames and shear walls. These types of retrofit solutions seem to have performed successfully, with minimal or no damage during the Canterbury earthquake sequence,

as in the case of the former Girl's High School in the Christchurch Arts Centre, where advanced Carbon FRP sheets were used along with steel bracings installed to connect the roof to the gables and to additional plywood layers to stiffen the floor diaphragms (NZHPT, 2012). It is clear that a final judgment on the effect of each technique, which is often applied in combination with others, can be understood only after a detailed scrutiny of each specific intervention. However the overall conclusion regarding the performance of well-considered retrofit interventions seems to be encouraging and promising.

CONCLUSIONS

The reported assembly of the stone masonry database for buildings located in Christchurch and surrounding districts, in conjunction with the cataloguing of observed damage mechanisms, allowed the seismic performance of these building to be studied. This damage assessment was based on data collected by NZ authorities during the building safety evaluation process, which entailed a safety placard being assigned to each building to define its usability. Assessment of the seismic performance of stone buildings was further completed using the classification of failure mechanisms contained in the damage survey forms adopted by the Italian Civil Protection Department for churches and buildings. By comparing safety placard data, and in particular the variation of assigned placard colour after the September 2010 and February 2011 events, the damage experienced by stone masonry buildings was evaluated and the significant escalation in the number of buildings having a red placard due to progressive accumulation of damage was quantified.

As a result of the damage surveys performed according to the procedures adopted by the Italian Civil Protection and Ministry of Cultural Heritage, the database of stone masonry buildings was completed with the cataloguing of damage mechanisms activated in each building and the relative damage quantification being recorded based on the EMS-98 proposal. Some of the mechanisms listed in the Italian damage survey forms were scarcely encountered because of the simplicity of the architecture of the buildings and churches considered, in which vaults or chapels are generally absent. From the analysis of activated mechanisms and the corresponding average damage value, it appeared that the most frequent and damaging mechanisms were related to the out-of-plane response of the buildings that caused partial or even complete collapse of the façade, as expected for unreinforced masonry structures and as reported in earthquakes that have recently occurred in Europe such as the 2009 L'Aquila earthquake in Italy (Carocci, 2012; D'Ayala and Paganoni, 2011; Lagomarsino, 2012). Also, the vulnerability of structural elements such as piers and spandrels that often experience serious damage due to shear and flexure was again demonstrated.

The analysis of the seismic performance of retrofitted buildings provided evidence on the effectiveness of strengthening interventions, when properly designed to improve the out-of-plane resistance and to enhance the global response of historic stone masonry constructions.

The description of damage sustained by the stone masonry buildings in Christchurch refers to their seismic response after the 22 February 2011 earthquake and following aftershocks that occurred until the end of May 2011. After the 13 June 2011 event and successive earthquakes the conditions of these heritage buildings continued to deteriorate. It was reported that after the 13

June 2011 earthquakes, the remaining parts of the Time Ball Station and of Lyttelton Holy Trinity Church completely collapsed as did several other unreinforced masonry buildings in Lyttelton that were in a similar state of damage (Ingham and Griffith, 2011, Appendix C). The Christchurch Anglican Cathedral suffered further damage after the 13 June 2011 earthquake and the rose window of the west façade collapsed following the 23 December 2011 earthquake.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support for “Project Masonry” from the New Zealand Natural Hazards Research Platform. The expenses for the research conducted by Ilaria Senaldi while in Christchurch were also partially covered through the DPC-Eucentre Executive Project 2009-2012.

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Figure 1. Location of earthquake epicentres having magnitude M_w greater or equal to 6, updated to December 2011.



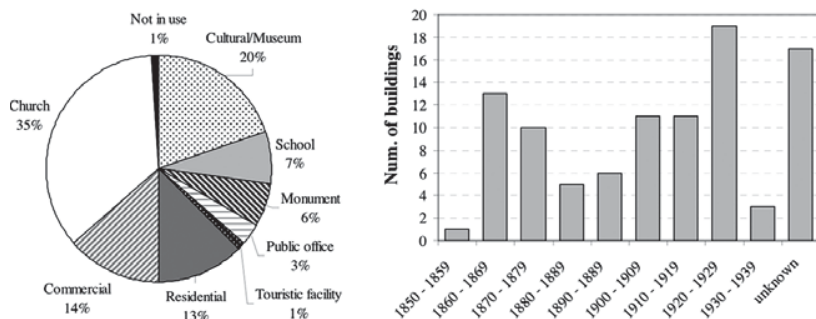
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Figure 2. Location of inspected stone masonry buildings in Christchurch and surrounding vicinity.



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Figure 3. Distribution of functions for the stone masonry building stock (left). Date of construction of buildings, subdivided per decade (right).



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Figure 4. Representative examples of wall cross-sections for Christchurch stone masonry buildings.



(a) Time Ball Station with multiple-leaf stone masonry



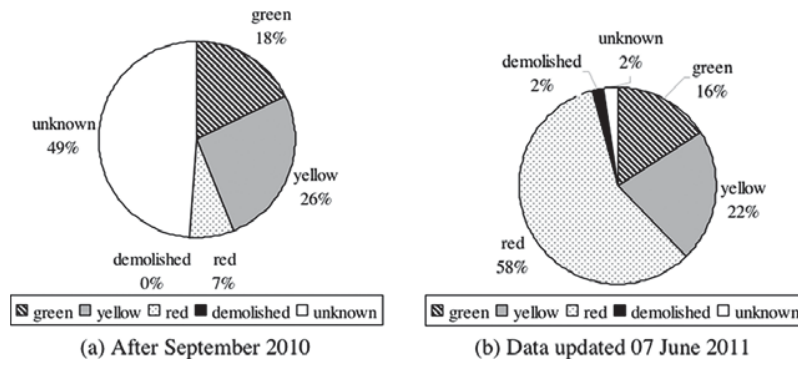
(b) Cathedral of the Blessed Sacrament in Oamaru stone with poured concrete core



(c) St. Luke's Anglican Church characterized by stone facings with clay brick interior leaves

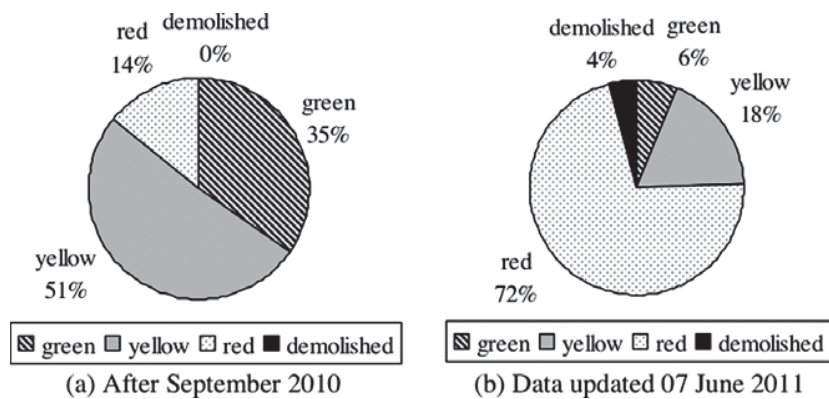
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Figure 5. Distribution of safety evaluation placards applied to stone masonry buildings.



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Figure 6. Safety evaluation placard data for 49 Christchurch stone masonry buildings where data was recorded after both earthquakes



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Figure 7. Distribution of safety evaluation placards applied to stone masonry buildings, differentiated by usage (data updated 07 June 2011).

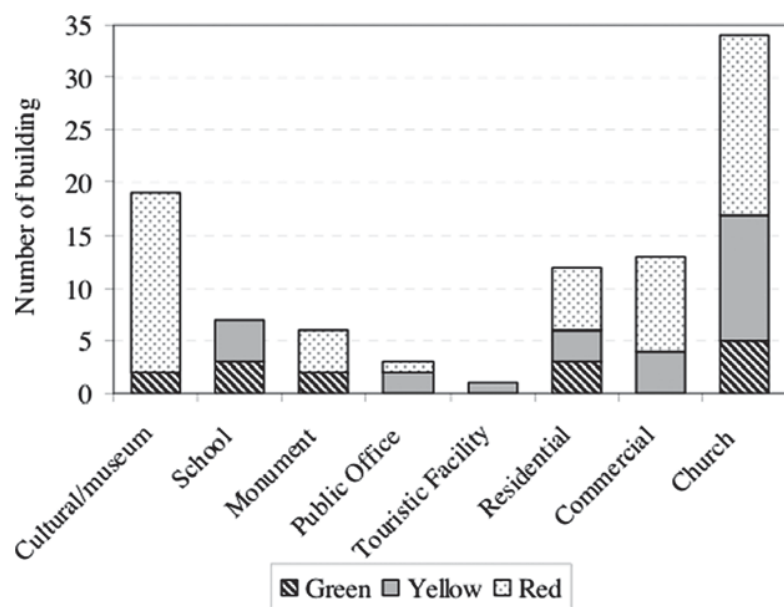


Figure 8. Damage mechanisms in the damage survey Form A-DC for churches (Form A-DC, 2006).

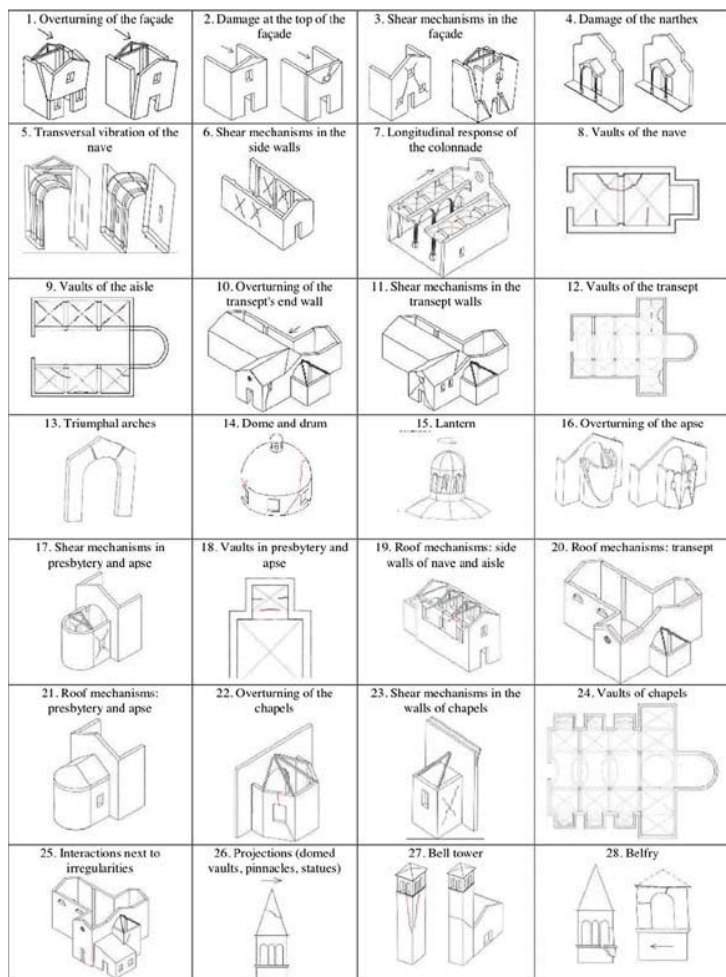


Figure 9. Damage mechanisms in the damage survey Form B-DP for palaces (Form B-DP, 2006).

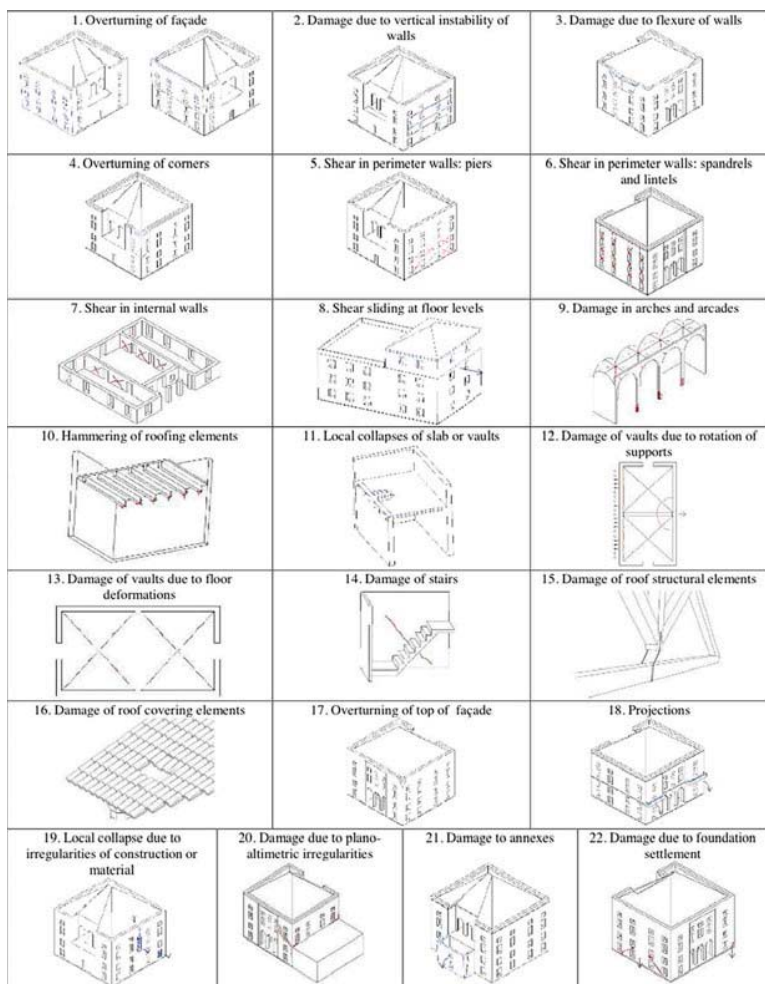
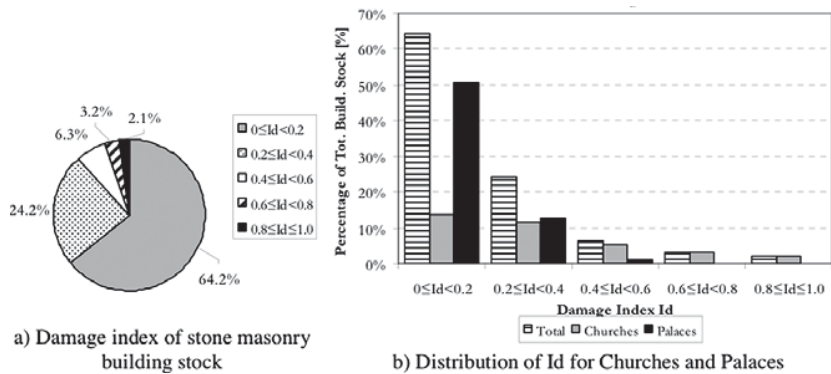
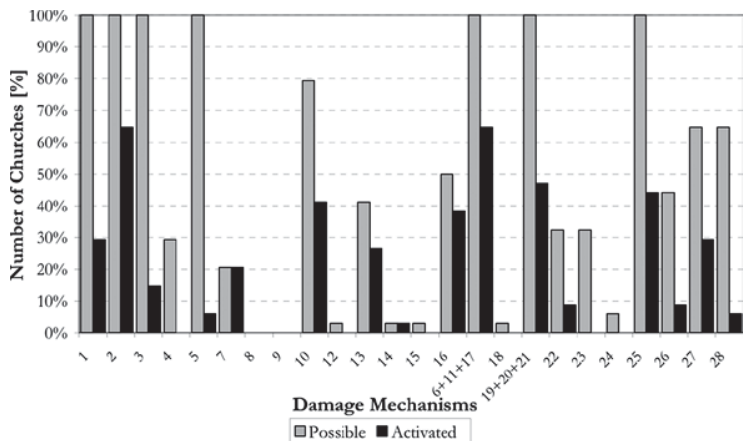


Figure 10. Distribution of damage index (Id)

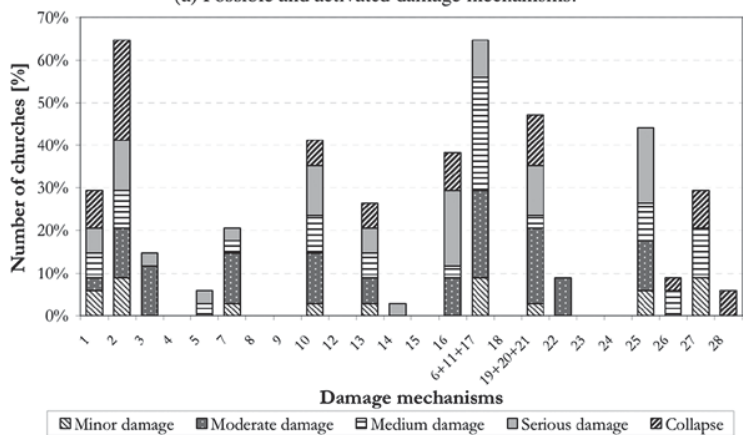


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Figure 11. Damage mechanisms in churches, classified as in the Form A-DC (2006).

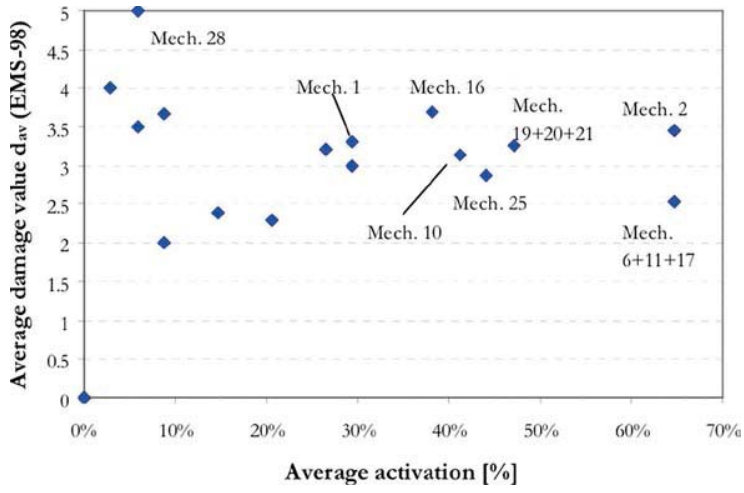


(a) Possible and activated damage mechanisms.



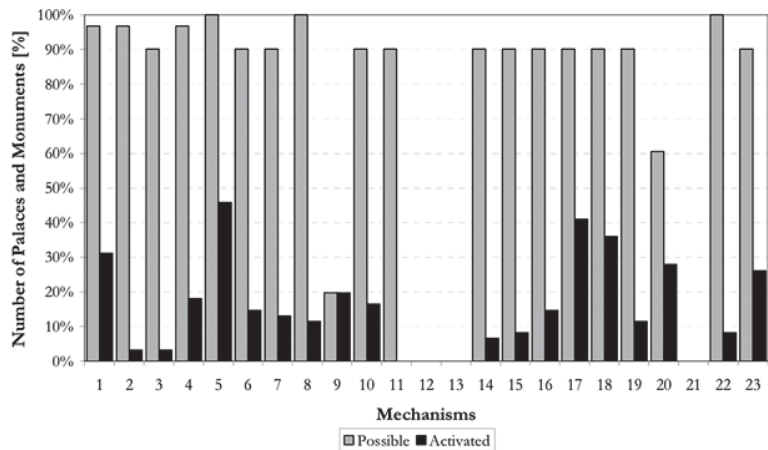
(b) Damage level of activated mechanisms

Figure 12. Average activation of damage mechanisms vs. Average damage value for churches.

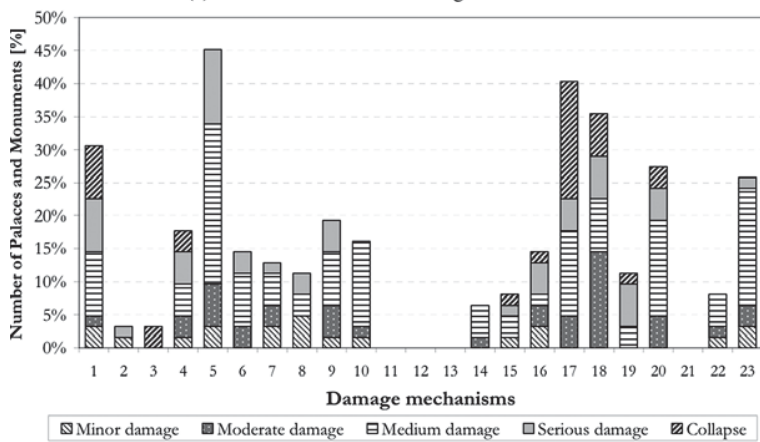


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Figure 13. Damage mechanisms in palaces and monuments, classified as in the Form B-DP (2006).

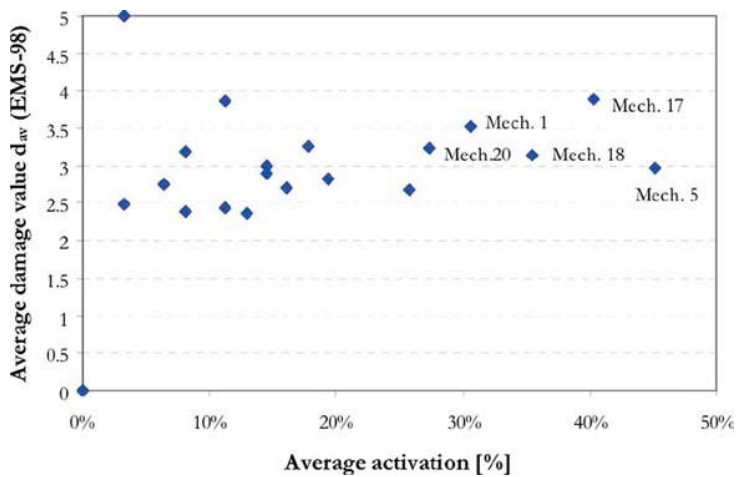


(a) Possible and activated damage mechanisms.



(b) Damage level of activated mechanisms

Figure 14. Average activation of damage mechanisms vs. Average damage value for palaces and buildings.



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Figure 15. Examples of global and local out-of-plane overturning mechanisms of façades



Figure 16. In-plane response of masonry piers and spandrels



(a) Diagonal cracks in the south façade piers of Christchurch Anglican Cathedral



(b) Diagonal pier cracks in the Lichfield Street façade of Strange's Building



(c) Flexural cracks in the spandrels of Cramner Court.

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Figure 17. Damage due to poor quality of materials



(a) Lyttelton Holy Trinity Church



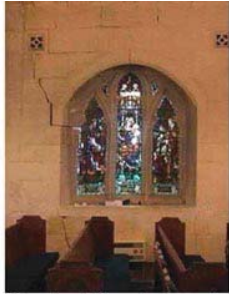
(b) St. Cuthbert's Church in Governor's Bay



(c) Time Ball Station. Damage to the octagonal tower

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Figure 18. Damages due to plano-altimetric irregularities



(a) St. Barnabas' Church, internal view in correspondence with the adjacent bell tower



(b) St. Mary's Anglican Church: detachment of the bell tower from the nave



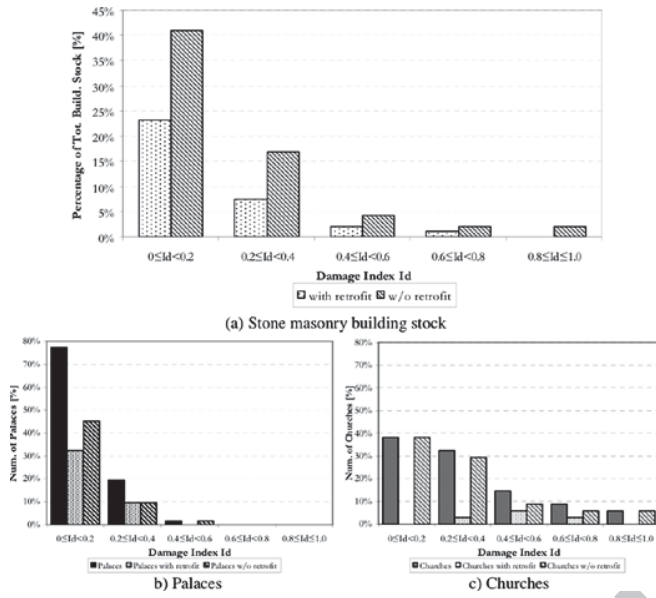
(c) Old Boy's High. Interaction between adjacent buildings (Distant view)



(d) Old Boy's High. Interaction between adjacent buildings (Close up view)

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Figure 19 Distribution of damage index Id, relatively to the effect of presence of strengthening interventions.



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Figure 20 Impact of strengthening interventions on damage index and damage level in selected mechanisms.

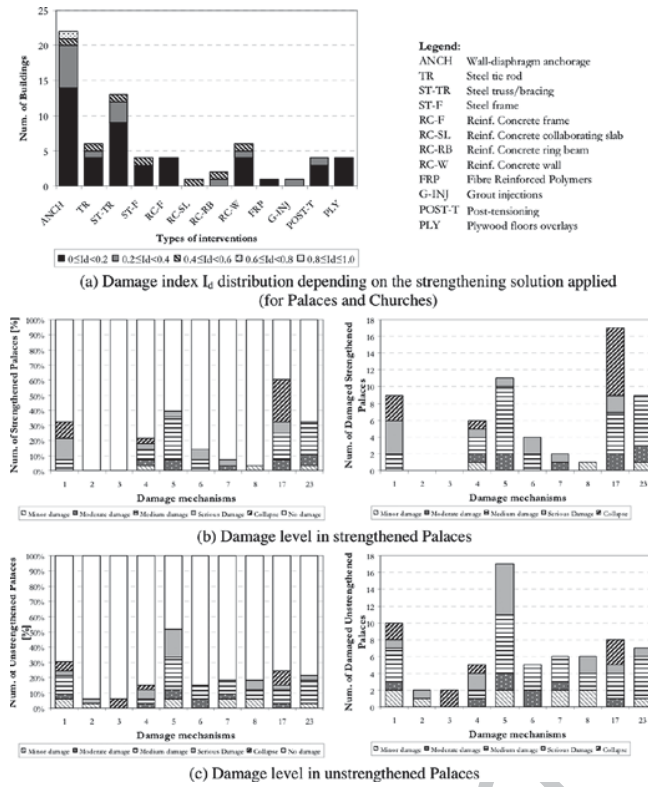


Figure 21. Presence and effects of wall-to-diaphragm anchors in gable end walls.



(a) Cramner Court.
Gable ended out-of-plane wall failure.



(b) Front façade gable of Former Trinity Church.
Presence of original wall-to-diaphragm steel anchors.



(c) Christ's College. Use of wall-diaphragm anchorages and of steel trusses.



(d) Christchurch Arts Centre.
Use of wall-diaphragm anchorages

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Figure 22. Examples of strengthening interventions applied to stone masonry buildings.



(c) Laurie and Wilsons Auctioneers building.
Presence of steel frame



(d) Former Malthouse Building. Presence of reinforced concrete frame



(e) Former Chemistry Department, Christchurch Arts Centre.
Use of post-tensioning.

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Table 1. Stone wall cross-section types and number of buildings for each type

Types of stone walls cross-sections	No. of buildings
Multi-leaf masonry walls (basalt or lava flow)	24
Three-leaf masonry walls with rubble lava flow façade, internal concrete core and Oamaru stone facing	13
Three-leaf masonry walls in Oamaru stone and concrete core	6
Ashlar stone facing (basalt or bluestone)	24
Ashlar stone facing and brickwork backing	17
Ashlar stone facing, concrete core and brickwork backing	3
Others	6
Undetermined*	3
Total	96

* Buildings already demolished at the time of inspections or with façades plastered.

Table 2. List of damage mechanisms in the damage survey form A-DC for churches

Damage mechanisms	Macroelement
1. Overturning of the façade	
2. Damage at the top of the façade	Façade
3. Shear mechanisms in the façade	
4. Damage of the narthex	
5. Transversal vibration of the nave	
6. Shear mechanisms in the side walls	
7. Longitudinal response of the colonnade	Nave
8. Vaults of the nave	
9. Vaults of the aisle	
10. Overturning of the transept's end wall	Transept

11. Shear mechanisms in the transept walls

12. Vaults of the transept

13. Triumphal arches

Triumphal arch

14. Dome and drum

Dome

15. Lantern

16. Overturning of the apse

17. Shear mechanisms in presbytery and
apse

Apse

18. Vaults in presbytery and apse

19. Roof mechanisms: side walls of nave
and aisle

Roof covering

20. Roof mechanisms: transept

21. Roof mechanisms: presbytery and apse

22. Overturning of the chapels

23. Shear mechanisms in the walls of
chapels Chapel

24. Vaults of chapels

25. Interactions next to irregularities

26. Projections (domed vaults, pinnacles,
statues)

27. Bell tower

Bell tower

28. Belfry

Table 3. List of damage mechanisms in the damage survey form B-DP for palaces

Damage mechanisms	Macroelement
1. Overturning of façade	
2. Damage due to vertical instability of walls (horizontal bending)	
3. Damage due to flexure of walls (vertical bending)	Façade
4. Overturning of corners	
5. Shear in perimeter walls: piers	
6. Shear in perimeter walls: spandrels and lintels	
7. Shear in internal walls	Internal walls
8. Shear sliding at floor levels	Global response
9. Damage in arches and arcades	Arcade

10. Hammering of roofing elements
11. Local collapses of slab or vaults
12. Damage of vaults due to rotation of supports
13. Damage of vaults due to floor deformations
14. Damage of stairs
15. Damage of roof structural elements
16. Damage of roof covering elements
17. Overturning of top of façade (gable walls)
18. Projections (parapets, chimneys, pinnacles, statues)
19. Local collapse due to irregularities of construction or material
20. Damage due to plano-altimetric irregularities
- Floors and vaults
- Stairs
- Roof
- Projections
- Local collapses
- Interactions

21. Damage to annexes

22. Damage due to foundation settlement

23. Bending in perimeter walls: spandrels and lintels

Façade

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