Damage Inspection in Churches – The Canterbury Earthquake (New Zealand) Experience

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ABSTRACT: The Canterbury Region of New Zealand experienced an extensive earthquake sequence during 2010-2011, with two particularly sever events being on September 4, 2010 and February 22, 2011. The present work entails a statistical analysis of the data for 112 churches in the affected region, including in situ damage observed by the authors and the structural assessment classification assigned by the local authorities, allowing for the discussion of the seismic performance of these churches separated in three main typologies: i) stone masonry; ii) clay brick masonry churches; iii) timber churches.

A simplified method of assessment of large span masonry structures, which had already been applied to a database of 44 monuments in Italy, Portugal and Spain, was applied with the objective of validating the proposed thresholds. A set of fragility curves, with the objective of estimating damage as a function of the peak ground acceleration, is also provided.

It was concluded that the timber churches had an excellent seismic performance, registering only non-structural damage, while the stone and clay brick churches clearly performed unsatisfactorily. The simplified method, which includes three separated indexes, presented very good results for one index (plan area ratio), and acceptable results for the other two (weight ratio and base shear ratio).

KEYWORDS: masonry; churches; seismic behavior; damage assessment; indexes; fragility curves

1 INTRODUCTION

It is often stated that the history of a country can be told by its heritage buildings. These buildings have an invaluable cultural significance but because of their architectural characteristics, construction materials and deterioration resulting from ageing, these heritage buildings are often highly vulnerable to extreme hazard events such as earthquakes. The seismic vulnerability of heritage buildings is much relevant for New Zealand (NZ), as the country's indigenous Maori population did not employ durable construction materials [1] and the country was one of the last lands to be colonized by Europeans. Hence the country's identity is represented by a comparatively small heritage building stock dating from 1833, emphasizing the need for seismic retrofit implementation to ensure that these heritage buildings can be retained for use by future generations.

When a region is struck by an earthquake, a specific procedure is triggered by the local authorities who have two different types of objectives when undertaking post-earthquake emergency assessment of buildings [2]. The primary objectives are the protection of human life and property, while the secondary objectives are related to minimizing the number of people made homeless by rapidly assessing buildings as safe or unsafe, evacuating dangerous areas, and creating shelter sites. In addition to data collection to inform the above objectives, data are also sought for purposes such as: (i) for authorities to develop disaster mitigation policies and allocate funds based on reliable estimates; (ii) for identification of the causes of damage, so that rehabilitation plans can take these hazards into account; (iii) for research, so that standards and construction practices can be re-evaluated, along with the development of supplementary resources such as seismic hazard maps. The methodology used for the seismic safety evaluation of buildings needs to be clear and straightforward, so that flaws can be limited to a minimum and reliable data can be retrieved. This need for clarity is a core issue as a variety of activities will be based on this information, such as building demolition and provisional securing interventions in the short term and the publishing of standard updates in the long term. Strong aftershocks are common and for that reason building assessments must be undertaken as quickly and as safely as possible due to the risk of collapse of damaged structures.

The Canterbury Region in the South Island of NZ underwent two severe earthquakes on 4 September 2010 and 22 February 2011 [3] [4]. Chronologically separated by only five months, and with epicentres located close to urban areas, the region suffered considerable human and material losses [5] [6]. In additional to building damage, much of the civil infrastructure sustained damaged due to the geotechnical phenomena of liquefaction and lateral spreading [7] [8]. The Central Business District (CBD) of Christchurch, the largest city of NZ's South Island and the nation's oldest and second largest city, was partially destroyed in February 2011 and had to be evacuated. A large number of heritage buildings, mostly constructed using unreinforced clay brick masonry, partially collapsed or were damaged beyond repair [9].

In order to study the behavior of the masonry and heritage buildings in the region affected by the Canterbury sequence, an international team of post-graduate students was deployed in Christchurch soon after the 22 February 2011 earthquake with coordination provided by the University of Auckland and funding provided by the NZ Natural Hazards Research Platform. Statistical analysis of the damage data gathered for churches in the region is presented here, followed by safety evaluation data collected by NZ authorities, as well as data on the damage classification registered for each church by the NZ authorities. The above mentioned Italian survey form was used for each church inspected, and the results are compared with those registered by the authorities. Almost all churches of the Canterbury region built before 1938 were assessed [10], leading to a total of 112 church buildings being contained within the dataset (see **Erro! A origem da referência não foi encontrada.**). The exceptions were churches that were already demolished and churches that were damaged to such an extent that it was unsafe to perform the assessment.

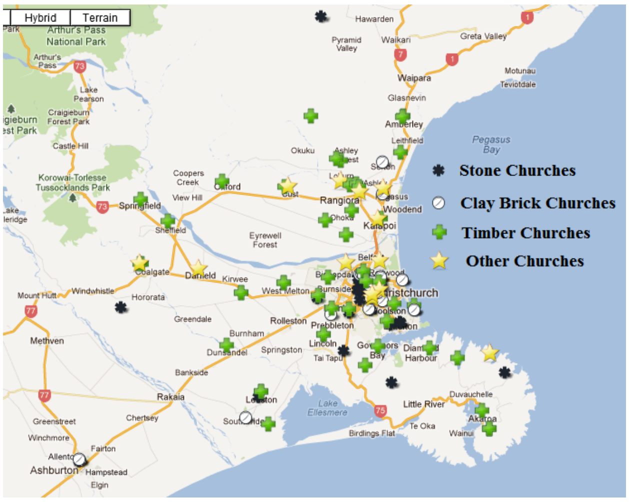


Figure 1. Location of surveyed churches in the Canterbury District of New Zealand.

2 DAMAGE INSPECTION

New Zealand legislation requires that immediately after the declaration of a state of emergency [11], a building safety evaluation process is activated. This procedure was followed in Christchurch City and surrounding districts after the earthquakes in September 2010 and February 2011. The process overview and guidelines are reported in [12] and were based on North American procedures developed by the Applied Technology Council [13] [14].

An immediate overall damage survey was performed by the Civil Defense and Territorial Authorities within hours of each event, with the objective of defining priority intervention areas and the human and technical resources required. Two levels of rapid assessments were next undertaken. Level 1 assessments were performed by structural and civil engineers, as well as by architects and other personnel from the building industry, with all buildings being assessed except for critical facilities and multi-story buildings. The survey form requires identification of the structural system, occupancy class and any structural damage that was visible by external observation. At the completion of the assessment the inspector assigned a placard (see Figure 2) to the building: *green* if there were no restrictions to use of the building; *yellow* if there were safety concerns, restricting use of the building to shorts periods of time for essential business; and *red* if the building was clearly unsafe and therefore re-entry of the building was prohibited (see Figure 3). Level 2 assessments were more thorough and therefore were only undertaken by experienced structural and geotechnical engineers, and were completed for critical facilities and multi-story buildings, as well as for all buildings that received *yellow* or *red* placards during the Level 1 assessments. For a Level 2 inspection an assessment was required of overall, structural, non-structural and geotechnical hazards.

The placards posted were valid during the state of emergency, superseding the Dangerous Buildings Notice posted under the Building Management Act 2004 [15]. The engineers that performed these assessments were mainly volunteers. After the first earthquake on 4 September 2010 nearly 100 engineers teamed up with NZ Fire Service Urban Search and Rescue (USAR) members, and the inspections started 12 hours after the shock [16].

(a) green for inspected and safe to re-enter | (b) yellow for restricted access | (c) red for unsafe

Figure 2. Placards given to buildings after Level 1 and Level 2 assessment.

New Zealand became a colony of the British Empire in 1840. After this date the immigrant population increased exponentially, as did the demand for residential and community buildings [17]. The first churches during this period were built mainly in timber because of the simplicity of construction, the wide availability of the material and a fast construction time. With growing prosperity stone and clay brick masonry started being used for the construction of important and public buildings, including churches, such that these three materials were the most common to be used for the construction of older NZ churches from the first quarter of the 20th century (see Figure 4). It is also possible to find a few churches built with reinforced concrete as well as churches constructed with a combination of the above mentioned materials.

(a) *green* tagged (St. Martin's, stone, Middleton) | (b) *yellow* tagged (St. Andrew's College, brick, Papanui) | (c) *red* tagged (Holy Trinity, stone, Lyttleton)

Figure 3. Examples of damaged churches.

As for all buildings affected by the Canterbury earthquakes, churches were assessed using Level 1 and, if necessary, Level 2 inspections. As stated above, almost all churches of the Canterbury region built before 1938 were assessed [10], leading to a total number of 112 church buildings contained within the dataset. This survey included the recording of the placard that was assigned to each church during the required safety evaluation, a visual inspection (exterior and interior when possible and safe) with photographic documentation of the damage and the geometrical measurement, in plan and height, using a distance meter laser. Further details can be found in [18] and [19].

As shown in Figure 5 (a), the three major construction typologies for churches in the Canterbury region are timber, stone masonry and brick masonry, with about 10% of the buildings using other materials. More than half of the surveyed churches (57%) received a green placard from the structural inspectors (see Figure 5 (b)).

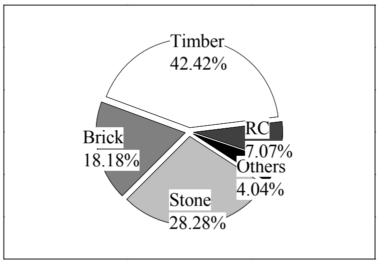
(a) timber church of St Andrews, Merivale, 1857

(b) stone church of St Peters, Upper Riccarton, 1876

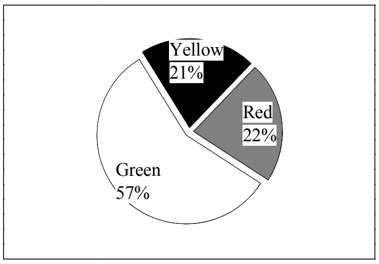
(c) clay brick church of Our Lady Star of the Sea, Sumner, 1912

Figure 4. Church typologies found in the Canterbury region.

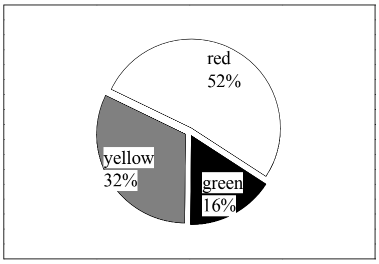
Given the different dynamic characteristics of the three principal church typologies found in the Canterbury region, the results are presented for each individual typology and comparing these findings with the overall results. More than half of the stone churches (52%) were assigned a red placard and only 16% of the churches had a green placard assigned (see Figure 5 (c)). The brick churches were less damaged than the stone churches, but also exhibited poor performance during the earthquakes. Figure 5 (d) shows that a red placard was assigned to 38% of the churches, while a yellow placard was assigned to 43% of the churches. The percentage of red placards assigned for this typology was smaller than the percentage assigned for the stone churches, but the sum of red and yellow placards was similar for both masonry typologies and exceeded 80%. The timber churches had the best overall performance (see Figure 5 (e)), with no structural damage and 94% of the churches were assigned a green placard. The single red placard assigned to a timber church was due to external cause while the yellow placard was due to non-structural damage like cracking of the inside or outside plaster.



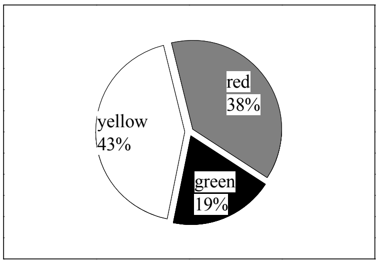
(a) Typology of all the churches



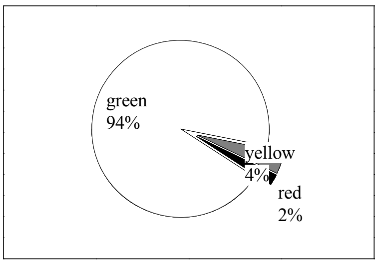
(b) Placard classification of all the churches



(c) Placard classification of stone churches



(d) Placard classification of brick churches



(e) Placard classification of timber churches

Figure 5. Damage statistics for the assessed churches.

3 SIMPLIFIED METHOD OF ASSESSMENT OF LARGE SPAN MASONRY STRUCTURES

The approach proposed in [20] aims at a simple, fast and low cost procedure analysis of the seismic risk, based on a simplified geometric approach for immediate screening of the large number of buildings at risk. The objective of this method is to compare simple geometrical data taking into account local seismic hazard (PGA), and to evaluate the possibility to adopt simple indexes (a numerical indicator deduced from observations and used as an indicator of a process or condition) related to geometrical data as a first (very fast) screening technique to define priority for further studies with respect to seismic vulnerability. These fast techniques are to be used without actually visiting the buildings, encompassing therefore a low accuracy. It is expected that the geometrical indexes could detect cases of serious risk and can define priority of studies in countries/locations without recent earthquakes.

The usage of simplified methods of analysis usually requires that the structure is regular and symmetric, that floors act as rigid diaphragms and that the dominant collapse mode is in-plane shear failure of the walls [21]. In general, these last two conditions are not met by ancient masonry structures, meaning that simplified methods should not be understood as a quantitative safety assessment but merely as a simple indicator of possible seismic performance of a building. The following simplified methods of analysis and corresponding indexes are considered as in-plane indexes (Index 1, Plan area ratio; Index 2, Area to weight ratio, Index 3, Base shear ratio) and out-of-plane indexes (Index 4, Slenderness ratio of columns; Index 5, Thickness to height ratio of columns; Index 6: Thickness to height ratio of perimeter walls). All indexes refer only to geometrical parameters. Factors that are not taken into account (albeit qualitatively) are the type of construction, the quality of the walls and the connections, and the presence of pushing structures. To address these factors would require in situ investigations, which are needed for the study of an individual building but can hardly be used for a first screening technique at territorial level, or for post-earthquake disasters, given the quantity of damage and the fact that access to the inside of many buildings is impossible due to safety reasons.

These methods can be considered as an operator that manipulates the geometric values of the structural walls and columns and produces a scalar value. As the methods measure different quantities, their application to a large sample of buildings contributes to further enlightenment of their application. As stated above, a more rigorous assessment of the actual safety conditions of a building is necessary to have quantitative values and to define remedial measures, if necessary.

3.1 In-plane indexes

The simplest index to assess the safety of ancient constructions is the ratio between the area of the earthquake resistant walls in each main direction (transversal x and longitudinal y, with respect to the church nave) and the total plan area of the buildings. According to Eurocode 8 [22], walls should only be considered as earthquake resistant if the thickness is larger than 0.35 m, and the ratio between height and thickness is smaller than nine. The first index $\gamma_{1,i}$ reads:

$$\gamma_{1,i} = {A_{wi}}/{S} \quad [-] \tag{1}$$

where A_{wi} is the area of the earthquake resistant walls in direction "i" and S is the total area of the building.

The non-dimensional index $\gamma_{1,i}$ is the simplest one, being associated with the base shear strength. Special attention is required when using this index as it ignores the slenderness ratio of the walls and the mass of the construction. Eurocode 8 [22] recommends values up to 5-6% for regular structures with rigid floor diaphragms. In cases of high seismicity, a minimum value of 10% seems to be recommended for historical masonry buildings [21]. For simplicity sake, high seismicity cases can be assumed as those where the design ground acceleration for rock-like soils is larger than 0.20g.

Index 2 provides the ratio between the area of the earthquake resistant walls in each main direction (again, transversal x and longitudinal y) and the total weight of the construction, reading:

$$\gamma_{2,i} = {A_{wi}}/{G} \quad [L^2F^{-1}] \tag{2}$$

where A_{wi} is the plan area of earthquake resistant walls in the direction "i" and G is the quasi-permanent vertical action. This index is associated with the horizontal cross-section of the building, per unit of weight. Therefore, the height (i.e. the mass) of the building is taken into account, but a major disadvantage is that the index is not non-dimensional, meaning that it must be analyzed for fixed units.

In cases of high seismicity, a minimum value of 1.2 m^2/MN seems to be recommended for historical masonry buildings [21], but on the basis of a more recent work [20], a minimum value of 2.5 m^2/MN is adopted for high seismicity zones.

Finally, the base shear ratio provides a safety value with respect to the shear safety of the construction. The total base shear for seismic loading ($V_{Sd,base} = F_E$) can be estimated from an analysis with horizontal static loading equivalent to the seismic action ($F_E = \beta G$), where β is an equivalent seismic static coefficient related to the design ground acceleration. It is recommended to use the value of PGA for β in historical masonry structures. The true value of β in a finer analysis depends on the failure mechanism. For local mechanisms a correction that takes into account the participation mass and the height of the center of gravity of the macro-block, together with a behavior coefficient, should be applied. The shear strength of the structure ($V_{Rd,base} = F_{Rd}$) can be estimated from the contribution of all earthquake resistant walls $F_{Rd,i} = \sum A_{wi} f_{vk}$, where, according to Eurocode 6 [23], $f_{vk} = f_{vk0} + 0.4\sigma_d$. Here, f_{vk0} is the cohesion, which can be assumed equal to a low value or zero in the absence of more information, σ_d is the design value of the normal stress and 0.4 represents the tangent of a constant friction angle, ϕ, equal to 22º. The new index γ_3 reads:

$$\gamma_{3,i} = {}^{F_{Rd,i}}/_{F_E} \quad [-] \tag{3}$$

If zero cohesion is assumed ($f_{vk0} = 0$), $\gamma_{3,i}$ is independent from the building height, reading:

$$\gamma_{3,i} = {}^{V_{Rd,i}}/_{V_{sd}} = {}^{A_{wi}}/_{A_w} \times {}^{\tan\phi}/_{\beta} \tag{4}$$

But for a non-zero cohesion, which is most relevant for low height buildings, $\gamma_{3,i}$ reads:

$$\gamma_{3,i} = {}^{V_{Rd,i}}/_{V_{sd}} = {}^{A_{wi}}/_{A_w} \times [\tan\phi + f_{vk0}/(\gamma \times h)]/\beta \tag{5}$$

where A_{wi} is the area of earthquake resistant walls in direction "i", A_w is the total area of earthquake resistant walls, h is the (average) height of the building, γ is the volumetric masonry weight, ϕ is the friction angle of masonry walls and β is an equivalent static seismic coefficient. Here, it is assumed that the normal stress in the walls is only due to their self-weight, i.e. $\sigma_d = \gamma \times h$, which is on the safe side and is a very reasonable approximation for historical masonry building, usually made of thick walls.

Equation 5 must be used rather carefully, since the contribution of the cohesion can be very large. Here, a cohesion value of 0.05 N/mm^2 will be assumed. This non-dimensional index considers the seismicity of the zone, which is taken into account in the factor β. The building will be safer with increasing ratio (earthquake resistant walls/weight), i.e. larger relation (A_w/A_{wi}) and lower heights. For this type of buildings and action, a minimum value of $\gamma_{3,i}$ equal to one seems acceptable.

The adopted indexes measure rather different quantities and cannot be directly compared. Index 2 is dimensional, which means that it should be used with particular care. Index 1 and index 2 are independent of the design ground acceleration. Therefore, assuming that the buildings must have identical safety, these indexes should be larger with increasing seismicity. For indexes 1 and 2, the seismicity is taken into account by considering that the threshold value given above is valid for a PGA/g value of 0.25 and a linear variation with PGA/g is assumed, as illustrated in Figure 6, see also Eurocode 8 [22]. In contrast, index 3 should be constant in different seismic zones, as it considers the effect of seismicity. This index format is close to the traditional safety approach adopted for structural design, being the threshold value equal to 1, see Figure 6.

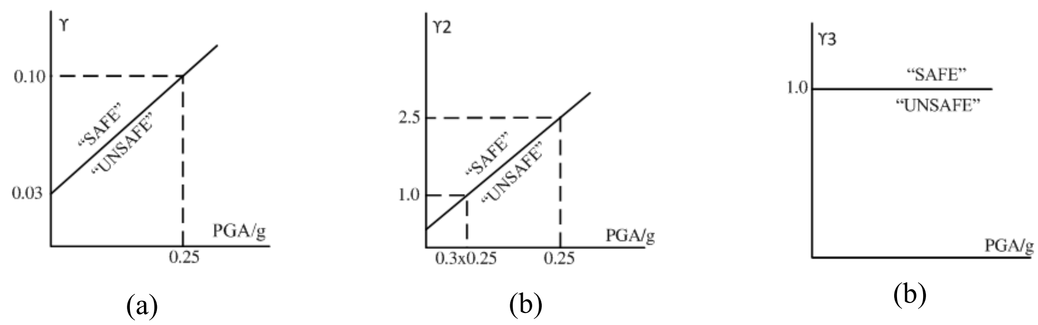


Figure 6. Assumed threshold for indexes 1, 2 and 3 as a function of PGA/g: (a) index 1; (b) index 2; (c) index 3.

3.2 *Out-of-plane indexes*

Besides the three indexes given above, other key indexes related with structural performance were computed for the database under analysis. It is well known that traditional masonry structures usually fail out-of-plane as observed in earthquakes, e.g. [24], and shaking table tests, e.g. [25]. Limit analysis using macro-blocks can be carried out to assess the seismic performance of partial collapses that occur due to seismic action, generally, with the loss of equilibrium of rigid bodies., see e.g. [25], but this detailed analysis is outside the scope of the present article.

Instead, three simple geometric ratios concerning the structural out-of-plane behavior of columns and walls were adopted, when applicable. Slenderness ratio γ_4, and thickness to height ratio of the columns γ_5, as well as thickness to height ratio of the perimeter walls γ_6, were analyzed, reading:

$$\gamma_4 = {h_{col}}/{(I/A)} \tag{6}$$

$$\gamma_5 = {d_{col}}/{h_{col}} \tag{7}$$

$$\gamma_6 = {t_{wall}}/{h_{wall}} \tag{8}$$

where h_{col} is the free height of the columns, I and A are the inertia and the cross section area of the columns, respectively, d_{col} is the (equivalent) diameter of the columns and t_{wall} and h_{wall} are the thickness and the (average) height of the perimeter walls, respectively. All of the out-of-plane indexes are dimensionless and do not consider the local seismicity. If identical safety factors for the monuments are assumed, these indexes should vary with increasing seismicity, namely index 4 should decrease and index 5 and index 6 should increase.

4 VALIDATION THRESHOLD WITH NZ DATA AND FRAGILITY CURVES

The assessment carried out after the CHC earthquake included the recording of the placard assigned to each church by the NZ National Authorities, a visual inspection (exterior and interior when possible and safe) with photographic documentation of the damage and the geometrical measurement, in plan and height, using a distance meter laser. The thickness of the walls was obtained rather easily in the clay brick churches because the same type of brick and construction techniques were used. As for the stone churches, the heavily damaged churches with large cracks or local collapses allowed an easy measurement, while in the less damaged churches the measurement was done at the openings (windows and doors). For furthers details see [19].

The weight, friction angle and cohesion of masonry are also needed in order to compute all three indexes. The same weight and friction angle values were considered for both typologies, 18 KN/m2 and 22° respectively, while the cohesion value assumed for f_{vk0} was 0.05 N/mm2. Index 3 was also computed considering zero cohesion, recognizing the particularly high level of recorded vertical acceleration.

The indexes related to the above mentioned simplified method of analysis were computed for all the stone and clay brick churches. The timber church typology was excluded since the performance was excellent and the proposed method is not applicable. The objective was to validate the proposed thresholds for each of the three in-plane indexes (in-plane area ratio, area to weight ratio, base shear ratio) by means of the PGA imposed to each church during the 22 February 2011 seismic event, acquired by the National Strong Ground Motion Network's equipment. Given the high number of instruments installed in structures and buildings [26], it was possible to associate the PGA recorded at a given location to a nearby church building. In most cases the distance between the church building and the accelerographs was less than 2 km, which ensures the quality of the produced data. Considering the latitude and longitude coordinates of each church and the horizontal PGA associated to it, it is possible to plot a contour map, see **Erro! A origem da referência não foi encontrada.**, which shows that the measured PGAs almost reach 1.4 g for the churches located near the CBD of CHC, and then decrease non-uniformly as the churches are located further away.

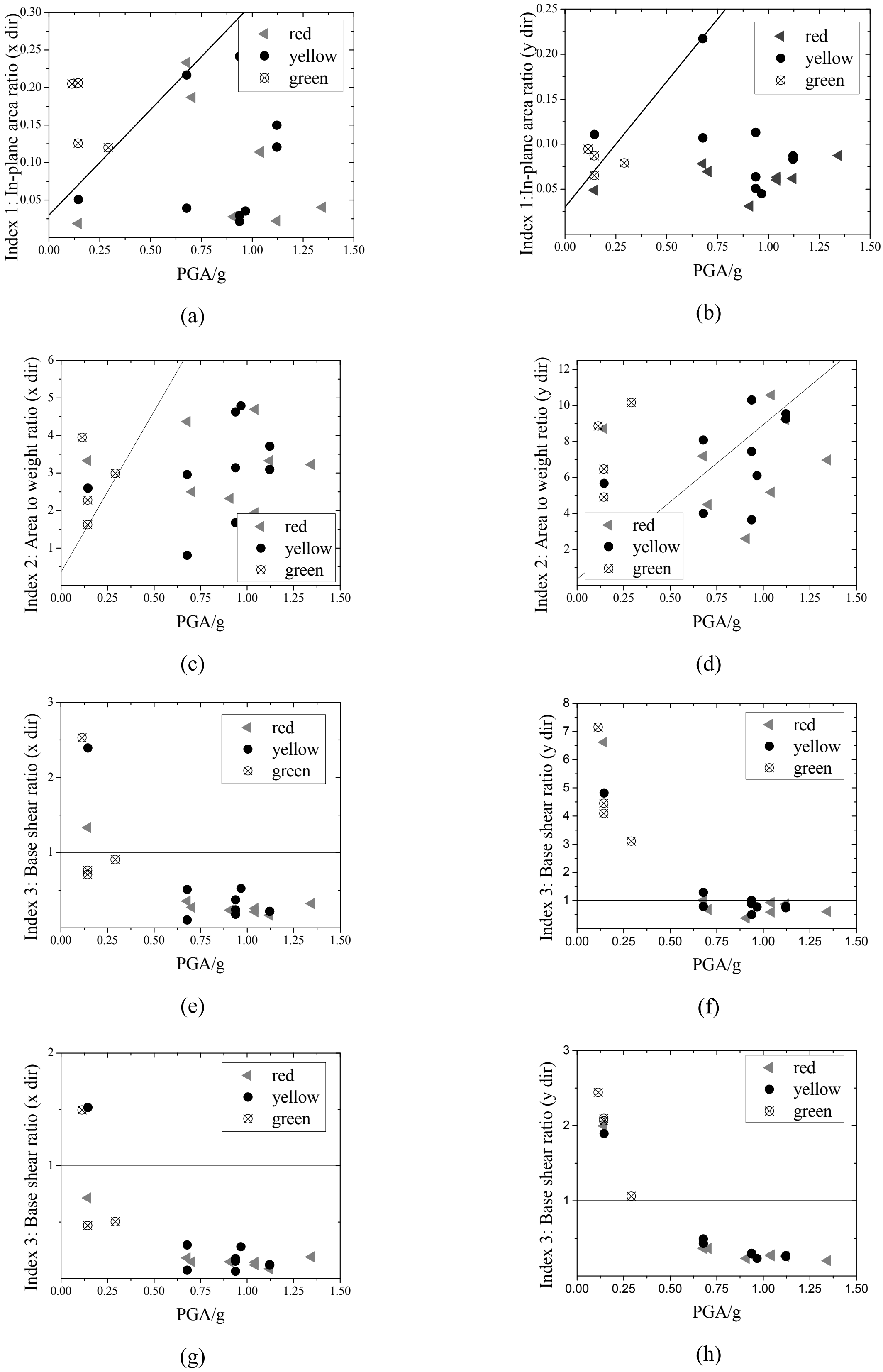


Figure 7. Clay brick churches: (a) index 1: In-plane area ratio in the x (transversal) direction; (b) index 1: In-plane area ratio in the y (longitudinal) direction; (c) index 2: Area to weight ratio in the x (transversal) direction; (d) index 2: Area to weight ratio in the y (longitudinal) direction; (e) index 3: Base to shear ratio, taking cohesion into consideration, in the x (transversal) direction; (f) index 3: Base to shear ratio, taking cohesion into consideration, in the y (longitudinal) direction; (g) index 3: Base to shear ratio, considering zero cohesion, in the x (transversal) direction; (h) index 3: Base to shear ratio, considering zero cohesion, in the y (longitudinal) direction.

Figure 7 presents the scatter plots of each index and the recorded horizontal PGA of the 22 February 2011 event for clay brick churches, as well as the proposed thresholds from Figure 6. Direction X and Y correspond, respectively, to the transversal and longitudinal directions regarding the main nave. The threshold for the first index is excellent, with all the green tagged churches above or near the line and only one yellow and one red church incorrectly identified. The yellow tagged church had only minor cracking with the exception of a large shear crack on one longitudinal wall of the main nave. There were also unstable nonstructural elements inside that could collapse during a stronger aftershock, which led the structural inspector to classify the church as yellow. The red tagged church was also a particular case, as it had pinnacles overhanging from the transversal walls and therefore was unstable even for a low PGA. These elements were severely damaged or partially collapsed compromising the connection between the transversal and longitudinal walls. The thresholds for index 2 and 3 also have acceptable results. The X (or transverse) direction provides better results in all three indexes, and this is the critical direction. The indexes are consistent even if they are not directly correlated. Index 3 exhibits the worse performance if cohesion is taken into consideration, with better results obtained for a zero cohesion, see Figure 7 (e) (f) (g) (h).

The thresholds for the stone churches are not as good as those for the clay brick churches, see Figure 8. For all indexes, and in both directions, there are green tagged churches subjected to a PGA equal or higher to 1g under the threshold, and red tagged churches subjected to a PGA lower than 0.125g above the threshold. The lack of homogeny of the stone churches justifies the lack of agreement with the thresholds, as the seismic behavior of these churches is rather different. Monumental good quality stone churches can present a seismic behavior similar to clay brick churches, while rubble weak stone masonry lacks interlocking and disaggregates, even for low PGA values. Redefining the thresholds is not a solution and the stone church typology would possibly have to be divided in sub-categories, according to more specific construction details. As it will be shown next with the fragility curves, the response of the stone churches is rather peculiar. As for the clay brick churches, there is a better agreement with the threshold of index 3 if cohesion is not taken into consideration.

Finally, it is noted that indexes 4 and 5 are related to the columns and these structural components rarely exist in the church typologies in New Zealand. These indexes are therefore non-applicable. Index 6 is also hardly applicable for these churches as they have rather small spans and many buttresses.

5 FRAGILITY CURVES

After the above computations, the assessed buildings were sorted following the damage index i_d assigned to each one [18] and the PGA that the buildings were subjected to on the 22 February event. The damage Index i_d, is based on the concept of *macroelements* [28]. These *macroelements* are subdivisions of the church based on architectural elements (such as facade, lateral walls, chapel, bell tower) which have an almost independent seismic behavior at collapse, therefore simplifying the complex structure of most churches into several smaller and simpler elements. The concept is based on experience acquired from past earthquakes, and was later revised and applied to the inspection forms [29] [30] used by the Italian Civil Protection [31].

Each data plot was divided into three areas obtained using the average PGA of the group and one standard deviation, iteratively defined to include 70% of the data. This clustering provided two empirical fragility curves which would be inadequate to estimate the global losses in the case of important damage. For this purpose the lognormal distribution was fitted to the observed data [32]. Several attempts were made but it was observed that the fit was far from perfect, providing unrealistic values for the extensive-complete damage, with low percentages of failure at higher PGAs. Finally, the following procedure was used, adopting the first two points in the fragility curves: (a) a lognormal distribution was fitted to the data points; (b) the lognormal cumulative distribution function was set to pass in the average quantile; (c) the standard deviation (measured by β) was obtained by the least square method for the slight-moderate sample, being the same β used for the extensive-complete damage. The fitted fragility curves are plotted in Figure 9, where a β equal to 1.3, 0.8 and 1.1, was found respectively for stone masonry, clay brick masonry and the entire sample. The higher vulnerability of stone churches is again demonstrated by the fitted fragility curves. Further details about this process can be found in [19].

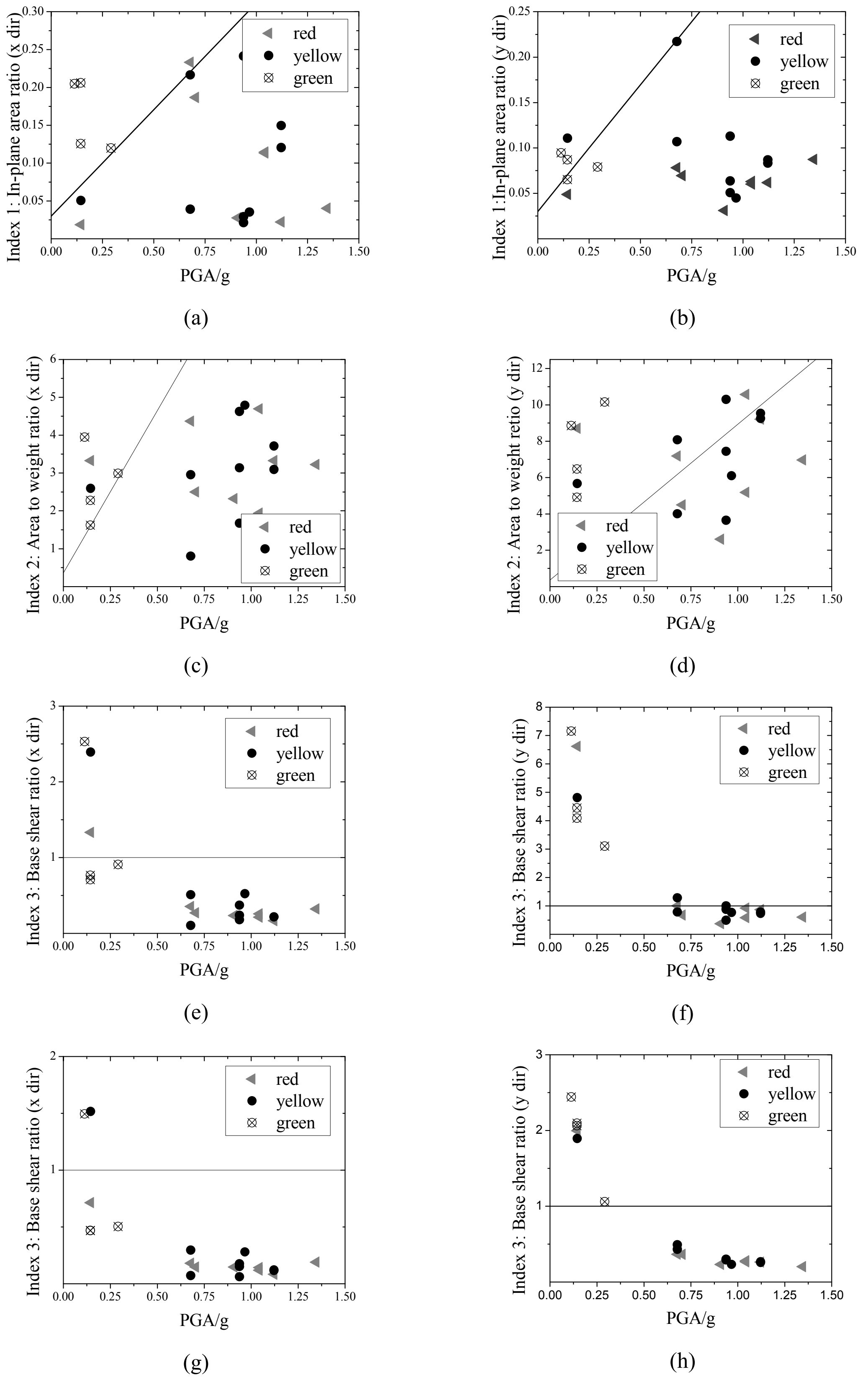


Figure 8. Stone: (a) index 1: In-plane area ratio in the x (transversal) direction; (b) index 1: In-plane area ratio in the y (longitudinal) direction; (c) index 2: Area to weight ratio in the x (transversal) direction; (d) index 2: Area to weight ratio in the y (longitudinal) direction; (e) index 3: Base to shear ratio, taking cohesion into consideration, in the x (transversal) direction; (f) index 3: Base to shear ratio, taking cohesion into consideration, in the y (longitudinal) direction; (g) index 3: Base to shear ratio, considering zero cohesion, in the x (transversal) direction; (h) index 3: Base to shear ratio, considering zero cohesion, in the y (longitudinal) direction.

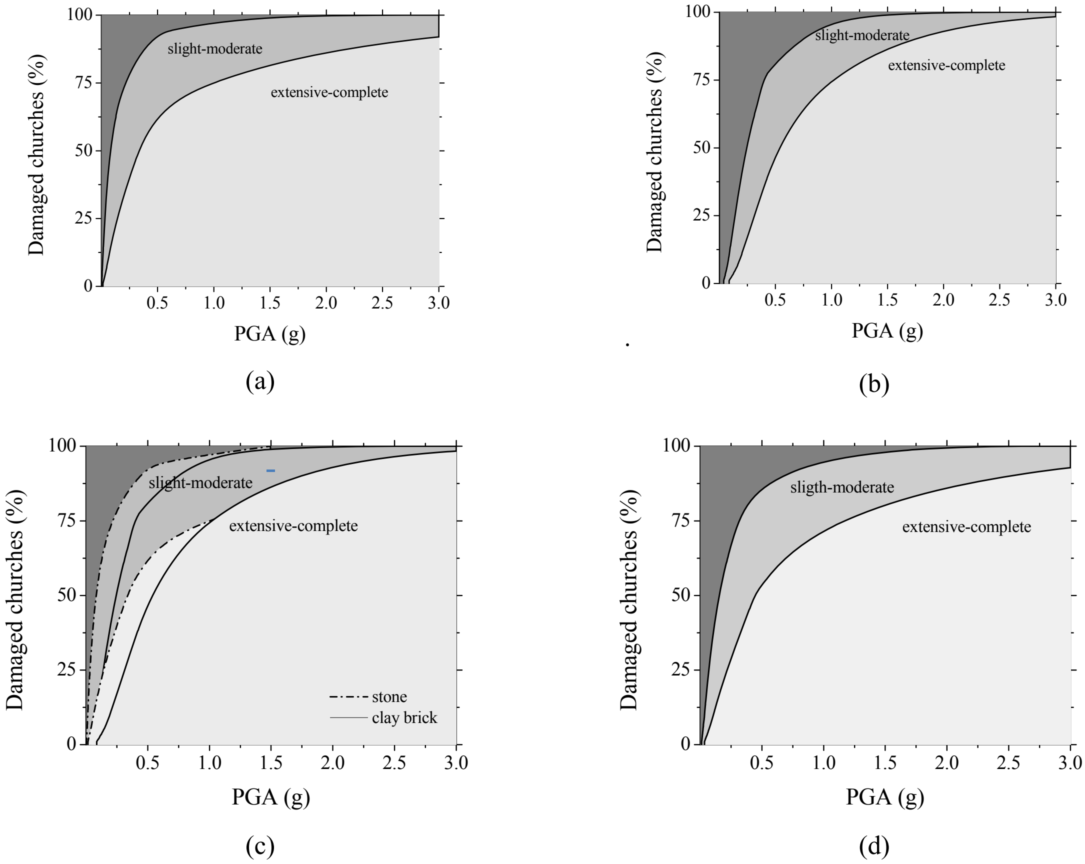


Figure 9. Fragility curves: (a) stone churches; (b) clay brick churches; (c) stone and brick; (d) all churches.

It is now assumed that the values of 5%, 50% and 95% quantiles are the lowest expected, the average, and the highest expected damage. For stone churches, it is expected that 50% receive a *yellow* tag for a PGA of 0.1g and a *red* tag for a PGA of 0.35g. For clay brick churches, it is expected that 50% receive a *yellow* tag for a PGA of 0.25g and a *red* tag for a PGA of 0.55g. For stone churches, it is expected that they all receive a *yellow* tag for a PGA of 0.65g and a *red* tag for a PGA of 3g. For clay brick churches, it is expected that they all receive a *yellow* tag for a PGA of 0.95g and a *red* tag for a PGA of 2.2g. Finally, for stone churches, it is expected that *yellow* tags appear for any seismic event and *red* tags appear for PGAs over 0.05g. For clay brick churches, it is expected that *yellow* tags appear for PGAs over 0.07g and *red* tags appear for PGAs over 0.15g.

In general for masonry churches, *yellow* tags are not expected for PGAs lower than 0.05g and *red* tags are not expected for PGAs lower than 0.1g. For PGAs of 0.15g and 0.5g, half of the churches are expected to be *yellow* and *red* tagged, respectively. For PGAs of 1g and 3g, all churches are expected to be *yellow* and *red* tagged, respectively.

6 CONCLUSIONS

From the statistical analysis of the obtained data it was established that a general comment on the overall performance of the churches was potentially misleading due to the existence of three main church typologies (stone, clay brick and timber), that exhibited different seismic characteristics. When analysing the typologies separately, the timber churches were found to have had an excellent seismic performance, while the stone and clay brick churches clearly performed unsatisfactorily. Only non-structural damage such as damaged plaster in the interior was registered during the assessment of timber churches, with 94% of these churches having received a *green* placard. The inverse scenario was found in the stone and clay brick churches, with 80% of those churches being assigned either a *yellow* or *red* placard.

As for the simplified indexes, the first index, being the plan area ratio, seems to provide very good results for clay brick churches, while the second index, being the area to weight ratio, and the third index, being the base shear ratio, also have acceptable results. The third index exhibits acceptable results only if the cohesion of masonry is not taken into consideration. The results for stone churches are inadequate, mainly due to their lack of homogeneity, since the database includes both monumental good quality masonry churches and rubble weak stone masonry with poor bond.

The present work also provides fragility curves for masonry churches based on the structural classification obtained during the safety evaluation process and the recorded PGA of each church. According to the results obtained, in general, *yellow* tags are not expected for PGAs lower than 0.05g and *red* tags are not expected for PGAs lower than 0.1g. For PGAs of 0.15g and 0.5g, half of the churches are expected to be *yellow* and *red* tagged, respectively. For PGAs of 1g and 3g, all churches are expected to be *yellow* and *red* tagged, respectively.

REFERENCES

[1] Russell, A., and Ingham, J. [2010] "Prevalence of New Zealand's Unreinforced Masonry Buildings'." Bulletin of the New Zealand Society for Earthquake Engineering 43 (3, Sept.), 182-201.

[2] Anagnostopoulou, M., Bruneau, M., and Gavin, H. P. [2010] "Performance of Churches During the Darfield Earthquake of September 4, 2010," Bulletin of the New Zealand Society for Earthquake Engineering 43 (4), 374-381.

[3] Bradley, B. A., and Cubrinovski, M. [2011] "Near-Source Strong Ground Motions Observed in the 22 February 2011 Christchurch Earthquake," Bulletin of the New Zealand Society for Earthquake Engineering 44 (4), 181-194.

[4] Gledhill, K., Ristau, J., Reyners, M., Fry, B., and Holden, C. [2010] "The Darfield (Canterbury) Earthquake of September 2010: Preliminary Seismological Report," Bulletin of the New Zealand Society for Earthquake Engineering 43 (4), 215-221.

[5] NZ Police. "List of deceased" [2011] Available from http://www.police.govt.nz/list-deceased

[6] NZ Treasury. "Government Budget" [2011] Available from http://www.treasury.govt.nz/economy/reports/econbrief-eice-10sep10.pdf

[7] Allen, J., Ashford, S., Bowman, E., Bradley, B., Cox, B., Cubrinovski, M., Green, R. A., Hutchinson, T., Kavazanjian, E., Orense, R., Pender, M., Quigley, M., and Wotherspoon, L. [2010] "Geotechnical Reconnaissance of the 2010 Darfield (Canterbury) Earthquake," Bulletin of the New Zealand Society for Earthquake Engineering 43 (4), 243-320.

[8] Cubrinovski, M., Bradley, B., Wotherspoon, L., Green, R., Bray, J., Wood, C., Pender, M., Allen, J., Bradshaw, A., Rix, G., Taylor, M., Robinson, K., Henderson, D., Giorgini, S., Ma, K., Winkley, A., Zupan, J., O'Rourke, T., DePascale, G., and Wells, D. [2011] "Geotechnical Aspects of the 22 February 2011 Christchurch Earthquake," Bulletin of the New Zealand Society for Earthquake Engineering 44 (4), 205-226.

[9] Dizhur, D., Ingham, J., Moon, L., Griffith, M., Schultz, A., Senaldi, I., Magenes, G., Dickie, J., Lissel, S., Centeno, J., Ventura, C., Leite, J., and Lourenco, P. [2011] "Performance of Masonry Buildings and Churches in the 22 February 2011 Christchurch Earthquake," Bulletin of the New Zealand Society for Earthquake Engineering 44 (4), 279-296.

[10] Hamilton, D., and Hamilton, J. (2008). *Early churches in and around Christchurch*, Christchurch: Judith and Derek Hamilton.

[11] New Zealand Legislation. "Civil Defence Emergency Management Act 2002" [2002] accessed 27/01/2012 Available from http://www.legislation.govt.nz/act/public/2002/0033/latest/DLM149789.html

[12] New Zealand Society for Earthquake Engineering [2009] "Building Safety Evaluation During a State of Emergency - Guidelines for Territorial Authorities."

[13] Applied Technology Council (ATC). (1989). "ATC-20: Procedures for Post-earthquake Safety Evaluation of Buildings ATC-20" Redwood City, CA, pp. 152.

[14] Applied Technology Council (ATC). (1995). "ATC-20-2: Addendum to the ATC-20 Post-earthquake Building Safety Procedures" Redwood City, CA, pp. 95.

[15] New Zealand Legislation. "Building Act 2004 No 72" [2004] accessed 27/01/2012 Available from http://www.legislation.govt.nz/act/public/2004/0072/latest/viewpdf.aspx

[16] Wood, P., Robins, P., and Hare, J. [2010] "Preliminary observations of the 2010 Darfield (Canterbury) earthquakes: an introduction.," Bulletin of the New Zealand Society for Earthquake Engineering 43 (4), i-iv.

[17] Russell, A., and Ingham, J. [2010] "Prevalence of New Zealand's Unreinforced Masonry Buildings'." Bulletin of the New Zealand Society for Earthquake Engineering 43 (3, Sept.), 182-201.

[18] Leite, J., Ingham, J. M., and Lourenço, P. B. [2013] "Statistical Assessment of Damage to Churches Affected by the 2010-2011 Canterbury (New Zealand) Earthquake Sequence," Journal of Earthquake Engineering 17 (1), 73-97.

[19] Lourenço, P.B., Oliveira, D.V., Leite, J.C., Ingham, J.M., Modena, C., Porto, F.da, [2013] "Simplified indexes for the seismic assessment of masonry buildings: International database and validation," Engineering Failure Analysis, December 2013 (34), pages 585-605.

[20] Lourenco, P. B., and Roque, J. A. [2006] "Simplified indexes for the seismic vulnerability of ancient masonry buildings", Construction and Building Materials 20 (2006), 200-208.

[21] Meli, R. [1998] *Structural engineering of historical buildings (in Spanish)*, Mexico-City: Fundación ICA.

[22] Eurocode 8 [2004] "Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings (EN 1998-1), European Committee of Standardization " Brussels, Belgium.

[23] Eurocode 6 [2006] "Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures (EN 1996-1-1), European Committee of Standardization" Brussels, Belgium.

[24] Doglioni, F., Moretti, A., Petrini, V., and Angeletti, P. [1994] *Le chiese e il terremoti: dalla vulnerabilità constatata nel terremoto del Friuli al miglioramento antisismico nel restauro, verso una politica di prevenzione*, Trieste, Italy: Edizioni Lint.

[25] Mendes, N., Lourenço, P.B. [2010], Seismic assessment of masonry "Gaioleiros" buildings in Lisbon, Portugal, Journal of Earthquake Engineering, 14: 80-101.
[26] Canterbury Regional Strong-Motion Network. "Canterbury Seismic Instruments" [2003] Available from http://www.csi.net.nz/profile.html
[27] Doglioni, F., Moretti, A., Petrini, V., and Angeletti, P. [1994] *Le chiese e il terremoti: dalla vulnerabilità constatata nel terremoto del Friuli al miglioramento antisismico nel restauro, verso una politica di prevenzione*, Trieste, Italy: Edizioni Lint.
[29] Angeletti, P., Ferrini, and Lagomarsino, S. [1997] "Survey and evaluation of the seismic vulnerability of the churches: and application in Lunigiana and Garfagnana (in Italian)", Proc. of the 8th National Conference: Earthquake Engineering in Italy, Palermo, 1077-1084.
[30] Lagomarsino, S., and Podestà, S. [2004] "Seismic Vulnerability of Ancient Churches: I. Damage Assessment and Emergency Planning," Earthquake Spectra 20 (2), 377-394.
[31] Civil Protection Department [2006] "Scheda per il rilievo del danno ai beni culturali - Chiese."
[32] Singhal, A., and Kiremidjian, A. S. [1996] "Method for probabilistic evaluation of seismic structural damage.," Journal of Structural Engineering 122 (12), 1459-1467.