GEOMETRIC CHARACTERISATION AND OUT-OF-PLANE SEISMIC STABILITY OF LOW-RISE UNREINFORCED BRICK MASONRY BUILDINGS IN AUCKLAND, NEW ZEALAND

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SUMMARY

The 2010-2011 Canterbury earthquakes and corresponding Royal Commission reports have resulted in changes to the legislative environment and led to increased public awareness in New Zealand of the earthquake performance of unreinforced masonry (URM) buildings. As a result, building regulators, owners, tenants, users and heritage stakeholders will be facing a unique challenge in the near future where assessments, improvements and demolitions of URM buildings are expected to occur at an unusually high rate. Auckland is the largest city in New Zealand and because of the relative prosperity of Auckland during the period 1880-1935 when most URM buildings were being constructed in New Zealand, the city has the largest number of URM buildings in the country. Identifying those buildings most at seismic risk in Auckland's large and varied building stock has warranted a rapid field assessment program supplemented by strategically chosen detailed assessments. Information that can be procured through rapid field inspections includes the building geometric typologies (e.g., heights, building footprint geometry and isolated versus row configuration), elevation type (e.g., perforated frame versus solid wall), wall construction (e.g., solid versus cavity, number of leaves) and basic construction material type (e.g., clay brick versus stone). Furthermore, investigation into the architectural history, heritage status and functional usage of Auckland's URM buildings will affect the direction of retrofit strategies and priorities. As the owner of a large and varied portfolio of URM buildings as well as the local organisation responsible for assessing building safety, Auckland Council is developing exemplar inspection, assessment, prioritisation and retrofit strategies that will target the seismic risks associated with URM buildings, in particular, so as to preserve and enhance safety and the economic and community value of these special buildings. Collaboration amongst Auckland Council, The University of Auckland and GNS Science has resulted in a state-of-the-art rapid quantitative assessment program applied to a sampling of typologically representative URM buildings in Auckland.

INTRODUCTION

As previously noted [1], the apparent seismic hazard in Auckland is relatively low within New Zealand, especially in Auckland Central where pluralities of population and unreinforced masonry (URM) buildings reside. However, the vulnerability of Auckland's built infrastructure is relatively high within New Zealand, as would be the consequences of earthquake-related damage. As of 2012, Auckland's economy accounted for an estimated 37% of New Zealand's GDP and the region's economic growth has outpaced New Zealand's national economic growth in 7 of the past 11 years [2]. Auckland's regional population of about 1.3 million in 2006 [3] accounted for 32.4% of the nation's population. Hence, a major natural disaster in Auckland would be detrimental to much of New Zealand's economy and people.

Cousins [4] suggests that URM buildings are 5.4 times more vulnerable (i.e., empirically-based mean ratio of repair cost to replacement cost is 5.4 times higher) than are post-1980 reinforced concrete (RC) buildings. By comparison, the second-most vulnerable building type is pre-1980 RC buildings at 2.3 times the vulnerability of post-1980 RC buildings [4]. Hence, as building performance in the Canterbury earthquakes has confirmed [5, 6, 7], URM buildings warrant special attention in a discussion on building typologies in Auckland for purposes of seismic hazard modelling. Furthermore, other historic New Zealand earthquakes have demonstrated that the first damage to a community of buildings is typically damage to the appendages and ornaments of URM buildings within that community, specifically chimneys and parapets. Chimney and parapet damage was widespread, for example, in the 2007 Gisborne earthquake [8] and the 2010 Darfield earthquake [9]. For higher levels of shaking intensity, significant out-of-plane

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failures of the walls of URM buildings are experienced [5]. From these observations and in response to the proposed "earthquake-prone buildings" amendment to the New Zealand Building Act [10] requiring all existing commercial buildings in New Zealand to be assessed for their earthquake capacity, it follows that a question of particular relevance is what level of forecasted shaking intensity is expected to initiate damage to unreinforced masonry buildings. The question has specific relevance for Auckland, which contains the largest stock of URM buildings in New Zealand and for which the level of seismic loading necessary to initiate damage will have a longer return period than for URM buildings located elsewhere throughout New Zealand.

INVENTORY OF URM BUILDINGS IN AUCKLAND

Auckland Council (AC) released its updated *Earthquake-Prone, Dangerous & Insanitary Buildings Policy* [11] in response to the Building Act [10] requirements that territorial authorities adopt such policies (Section 131 of the Building Act) to identify if buildings are considered "earthquake-prone" (Section 122 of the Building Act), dangerous, or otherwise unsuitable for human occupation. The 2011 policy was an update to the 2006 policies of the former Auckland territorial authorities. Under the current policy, a process has been outlined for identifying earthquake-prone buildings and, where applicable, working with building owners to establish a scope and timetable for any building improvements.

In response to these regulations, Auckland Council Building Control has worked with contracted professional engineers to inspect and document commercial buildings in the Auckland region most likely to be vulnerable to earthquakes. All Council-affiliated engineers follow inspection guidelines to procure data, records and photographs to aid the council in prioritising buildings for further assessment and, potentially, seismic retrofitting. The inspectors have been utilising a program based largely on the Initial Evaluation Procedure (IEP) per NZSEE [12].

As of May 2014, the amalgamated database of commercial unreinforced masonry (URM) buildings in Auckland includes 901 buildings. The authors expect that this list represents the vast majority of commercial URM buildings in Auckland. The term "commercial buildings" is defined as including industrial buildings and multi-unit, rent-tenanted residential buildings in addition to those traditionally considered commercial. However, low-unit residential (e.g., single-family houses and condominiums) are excluded.

Estimates of the total number or percentage of URM buildings are considered as if 1026 is the total number of commercial URM buildings in Auckland, based on an estimate by Russell and Ingham [13] and corresponding to a percentage of the entire commercial building stock proposed by Cousins [4] of about 5% - 6%. However, not all of the documented 901 URM buildings have been inspected and not all of the estimated 1026 URM buildings have been identified. Where appropriate, a distinction has been made between whether quantitative values represent "documented" or "estimated" buildings as well as whether percentages represent proportions including or excluding unknown building attribute data. Furthermore, a bias exists in the documented data. Investigators have prioritised buildings most likely to be vulnerable to earthquakes, producing a partiality in the data pool to older, taller buildings located close to the city centre. These biases have been accounted for and corrected as much as possible, as noted in the following sub-sections. Non-occupied unreinforced masonry monuments, kilns, chimneys, free-standing walls and ruins are not considered in this study. Finally, it is recognised that the total stock of URM buildings will continuously decline due to building demolition without replacement, and hence the current estimate of 1026 buildings may diminish over time.

Primary Lateral Load-Resisting Material and System

In the amalgamated database, all 901 identified URM buildings have been documented with the primary lateral load-resisting system (LLRS) recorded as either unreinforced brick masonry (URBM) or unreinforced stone masonry (URSM). Auckland Council Building Control has provided the vast majority of this information, so the structure type nomenclature used in the Auckland Council / University of Auckland (AC / UoA) study has been matched to the precedent set by AC Building Control. These categories are listed in Table 1 along with their presumed equivalents in the Global Earthquake Model [14] and RiskScape [15] taxonomies. The number of documented buildings within each AC / UoA structure type category, as well as the estimated total number and percentage of buildings within each taxonomic construction type category, is also listed in Table 1.

University of Auckland researchers identified 19 occupiable buildings in Auckland with above-grade, load-bearing unreinforced stone masonry (URSM) walls. As URSM buildings represent such a small portion of the Auckland building stock, these buildings have been neglected from the scope of the quantitative assessment program described later, and the focus has been applied to URBM buildings.

Number of Storeys

In the amalgamated database, 879 URM (out of 901) buildings have been documented with a particular number of storeys above grade and the associated percentage groupings are illustrated in Figure 1. As discussed later, for typological groupings to include multiple attributes (e.g., structure type, number of storeys and age of construction), buildings have been grouped into categories of 1 - 3 storeys and 4 - 7 storeys, consistent with previous typological groupings used in New Zealand [16]. Two buildings were identified as having eight storeys above grade, but both buildings have frame systems supporting most or all of the gravity loads while the masonry is expected to provide the primary lateral load resistance.

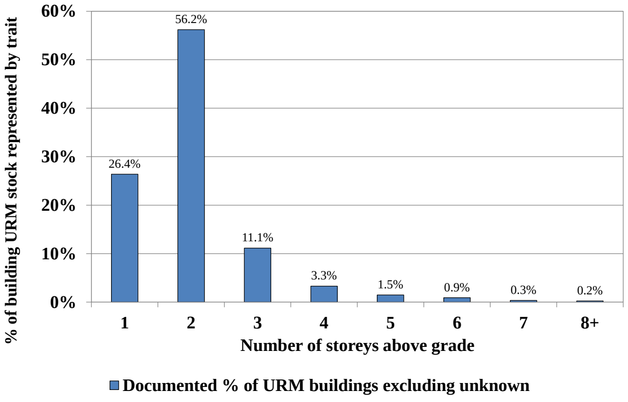


Figure 1: Proportions of documented commercial URM buildings in Auckland by number of storeys above grade.

Table 1. Summary of primary lateral load-resisting material and structural system attributes for commercial URM buildings in Auckland

AC / UoA		GEM (2013)					RiskScape (2010)		
Structure type category	**Documented # bldgs**	**Material of the LLRS system level 1**	**Material of the LLRS system level 2**	**LLRS Level 1**	**Estimated #**	**Est. % of all commercial bldgs**	**Constr. type category**	**Estimated #**	**Est. % of all commercial bldgs**
URBM (unreinforced brick masonry)	882	MUR (Masonry, unreinforced)	CLBRS (Fired clay solid bricks) or CLBRH (Fired clay hollow bricks) and/or RCB (Reinforced concrete bands)	LWAL (Wall)	1007	5.8%	Brick masonry (to include stone masonry)	1026	5.9%
URSM (unreinforced stone masonry)	19		STRUB (Rubble (field stone) or semi-dressed stone) or STDRE (Dressed stone)	LWAL (Wall)	19	0.1%			

The bias in the documented data for number of storeys above grade likely over-represents taller buildings given the initial emphasis on inspecting buildings near the city centre as well as the inherent higher profile of taller buildings and the higher risk consequence associated with them. Hence, the total number of estimated buildings with 4 - 8 storeys was capped at the current number of documented buildings with 4 - 8 storeys (totalling 55 buildings) and proportional extrapolations were made only for buildings with fewer than 4 storeys.

Year of Construction or Retrofit

In the amalgamated database, 830 URM buildings (out of 901) have been documented with a particular or approximate year of construction, reconstruction, or implementation of seismic retrofit to the primary lateral load-resisting system. For typological groupings to include multiple attributes (e.g., structure type, number of storeys and age of construction), buildings have been grouped into ranges of years consistent with major updates to the loading standard and with previous typological groupings used in New Zealand [12, 16, 17, 18]. The proportions associated with these simplified year groupings are illustrated in Figure 2. For breakdowns of URM building construction by decade, please refer to Walsh and Ingham [1] and Russell and Ingham [13].

Most URM buildings in New Zealand were originally constructed before 1940, although a few were constructed in Auckland as late as the 1950s [13]. In the 1965 Standard Building By-Law [19], URM buildings with more than two storeys were prohibited as new construction in Auckland and no URM buildings of any height have been identified as having been constructed after 1965. Hence, the buildings represented in Figure 2 (and in Table 2 as follows) as being constructed post-1965 denote major reconstruction or retrofit having occurred to an existing URM building. The specific type of reconstruction or seismic retrofit has not yet been documented for most buildings.

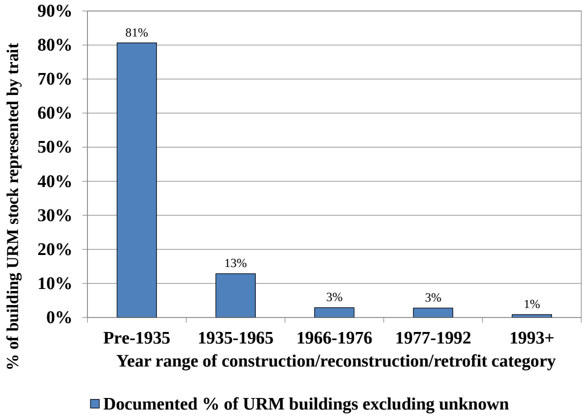


Figure 2: Proportions of documented commercial URM buildings in Auckland by year of construction, reconstruction, or seismic retrofit.

Typological Groupings by Structure Type, Number of Storeys and Year of Construction

To facilitate typological groupings conducive to assigning accurate building fragility functions within seismic hazard models, the three primary structural attribute categories of structure type, number of storeys and year of construction were combined in a hierarchical fashion. Note that while such hierarchies within building taxonomies are common and perhaps most useful regionally, they can be restrictive within more encompassing taxonomies and are not favoured in GEM [20]. However, the information is presented here in such a fashion for simplicity. The five most prominent URM building typological groupings by estimated percentage of the total commercial URM building stock are listed in Table 2, representing 96% of all of Auckland's commercial URM buildings. Note the dominance of low-rise, pre-1935 URM buildings in this list, consistent with the types of buildings considered in the quantitative analysis described later.

Occupancy / Usage Type and Importance Level

The third component of risk that must be quantified in models, along with hazard and vulnerability, is consequence. Hence, both GEM [14] and RiskScape [15] consider attributes related to the functional use or occupancy of the buildings being considered. GEM and RiskScape have different applications for the word "occupancy," but both taxonomies accommodate building usage type in some fashion.

In the amalgamated database, 831 documented URM buildings (out of 901) have been assigned a particular use category. AC Building Control has provided the vast majority of this information, so the use category nomenclature used in the AC / UoA study has been matched to the precedent set by AC Building Control. The ten most prominent AC / UoA use categories by average importance level [21] and estimated percentage of the total commercial building stock for each of the taxonomies are listed in Table 3. The AC / UoA occupancy terms have been converted into their presumed counterparts in Table 3 for each of the taxonomies with re-rankings based on the different nomenclature and groupings of buildings by such nomenclature.

The importance level (IL) can be indicative of the number of people within a building as well as its post-disaster pertinence. Table 4 summarises information from building standards and seismic hazard assessments most relevant to Auckland's buildings. Most buildings in Auckland will likely be considered to have 50-year design working lives for assessment and retrofit design purposes, but some buildings of particular significance to the community could be considered for 100-year design working lives. For design and assessment purposes, most buildings will be assigned IL 2 or IL 3. The return periods listed in Table 4 correspond with ultimate limit state (ULS) design parameters to include strength, ductility, serviceability and durability.

The Modified Mercalli (MM) intensity scale is used to describe the damage and intensity experienced by people at a particular location. MM7 and MM8 intensities approximate the range of maximum credible seismic hazards relevant to the Auckland region [4], as shown in Table 4. MM7 intensity is associated with cracked unreinforced brick and stone walls with minor masonry falls, while MM8 intensity is associated with heavy structural damage with partial collapse of some unreinforced masonry walls [22].

OUT-OF-PLANE SEISMIC ASSESSMENT OF EXTERIOR URBM WALLS

Auckland Council Civil Defence and Emergency Management (CDEM) commissioned researchers from the University of Auckland to assist CDEM in documenting and assessing a selection of URBM buildings in the Auckland region most likely to be vulnerable to earthquakes. On that basis, a survey of the geometric characteristics of 206 URBM buildings located in Auckland (whose locations are marked in Figure 3) was conducted in order to get a representative distribution of the geometric attributes that are inputs into the assessment procedure, as well as a representative distribution of expected performance thresholds against varying peak ground acceleration (PGA) intensities. Given the observations of URM wall failure in Canterbury [5, 7] as illustrated in Figure 4, along with the relatively low (for New Zealand) seismic hazard in Auckland, it is expected that most URBM buildings in Auckland will have adequate in-plane strength to sustain moderate earthquakes and to avoid global failure, but that out-of-plane (OOP) capacity and the associated local failure mode is critical. As a result, recent research has been performed on modelling the OOP performance of URM walls [23, 24, 25] and retrofitting URM load-bearing and infill walls for OOP capacity [26, 27, 28, 29]. The procedure proposed by Derakhshan *et al.* [24, 25] for the numerical assessment of URM parapets and walls subjected to out-of-plane loading forms the basis of the procedure advocated by NZSEE for updates to the assessment guidelines expected in 2014. This procedure is largely based on the geometric attributes of URM buildings. The Derakhshan *et al.* [24, 25] model for vertical flexure (i.e., flexure about a horizontal axis) was coupled with two-way flexure assessment criteria derived from the design criteria of AS 3700:2011 [30] in order to assess the 206 URM buildings in this study.

Table 2. Ranking of top 5 building typological categories by number of estimated buildings

Rank	AC / UoA structure type category	Storeys category	Year of construction/reconstruction/retrofit category	Estimated # buildings	Estimated % of commercial URM buildings
1	URBM	1-3	Pre-1935	764	74.5%
2	URBM	1-3	1935-1965	129	12.6%
3	URBM	4-7	Pre-1935	41	4.0%
4	URBM	1-3	1966-1976	27	2.6%
5	URBM	1-3	1977-1992	24	2.4%
Total	-	-	-	**986**	**96%**

Table 3. Ranking of top 10 building occupancy / usage type attributes for each taxonomy scheme by percentage of estimated total.

Rank	Importance level [21]		GEM [14]		RiskScape [15]	
	AC / UoA Use category	Avg. importance level	Occupancy category	% of total incl. unknown	Use category	% of total incl. unknown
1	Emergency services facility	4.0	COM1 (Retail trade)	49.6%	Commercial - Business	62.6%
2	Prison	4.0	COM3 (Offices, professional/technical services)	12.4%	Other	7.8%
3	Hospital	3.7	OC99 (Unknown occupancy type)	7.9%	Residential Dwellings	5.2%
4	Recreation centre	3.0	RES2 (Multi-unit, unknown type)	5.2%	Lifestyle	4.3%
5	Utility station	3.0	COM11 (Recreation and leisure)	4.3%	Community	4.1%
6	Cinema	2.7	COM6 (Public building)	4.1%	Religious	3.6%
7	Education	2.6	ASS1 (Religious gathering)	3.6%	Industrial - Manufacturing, Storage	2.7%
8	Community facility	2.5	OCO (Other occupancy type)	2.2%	Commercial - Accommodation	2.2%
9	Government	2.5	COM4 (Hospital/medical clinic)	1.9%	Education	2.0%
10	Church	2.5	EDU99 (Education, unknown type)	1.8%	Hospital, Clinic	1.9%
Total	-	-	-	**93%**	-	**96%**

Table 4. Building design criteria and associated return periods and hazard intensities for Auckland [4, 21].

Design life	Import. level	Importance level assignment	Annual prob. of exceedance for EQ ULS	Approx. MM intensity
50 years	2	Normal structures	1/500	MM6.8
	3	Crowds or valuable assets	1/1000	MM7.2
100 years	2	Normal structures	1/1000	MM7.2
	3	Crowds or valuable assets	1/2500	MM7.6

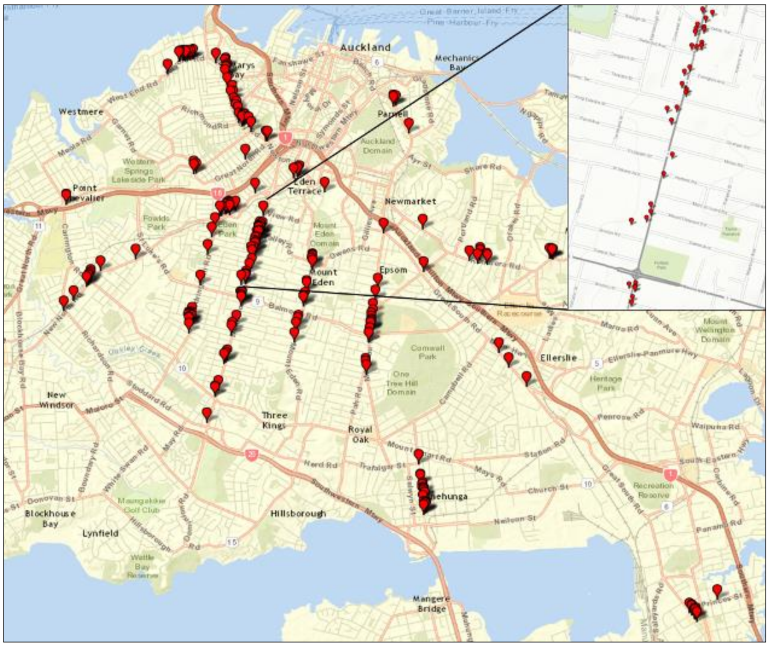


Figure 3: Locations of URBM buildings in Auckland Central assessed for CDEM study (inset – zoom on Dominion Road).

(a) Building A parapet condition in October 2010. (b) Building A parapet condition in March 2011. (c) Building B facade condition in October 2010.

(d) Building B condition in March 2011 – collapse of façade. (e) Building C post-22nd February 2011 – collapse of outer leaf of cavity wall. (f) Building C post-13th June 2011 – collapse of inner leaf of cavity wall.

Figure 4: Post-earthquake observations of out-of-plane failures to URBM buildings in Christchurch.

URBM Typologies

Russell and Ingham [13] have reported on the characteristics of URM buildings in New Zealand, established a set of URM building typologies and proposed that the stock of URBM buildings across the country is reasonably homogeneous, with a majority of buildings being defined by a narrow range of material and geometric characteristics. Prototypical examples of the most prominent sub-typologies documented from the CDEM study as classified by the Russell and Ingham [13] scheme are illustrated in Figures 5 and summarised in Figure 6(a). While one-storey row buildings are deemed most prevalent across New Zealand, the data illustrated in Figure 1 and Figure 6(a) indicate that Auckland has a greater proportion of multi-storey row buildings, especially in the older parts of town, such as the central business district (CBD) and suburbs nearest the CBD. Approximately half of the buildings surveyed for the CDEM study, as shown in Figure 6(b), have cavities, indicating that an air gap exists between leaves (i.e., horizontal layers in a wall cross-section) of bricks. While most brick cavity walls have been traditionally tied during original construction as described in Tasligedik *et al.* [26], the original ties have often corroded to the point that they are no longer functional and the individual wall leaves are expected to respond to lateral loads independently (i.e., non-compositely). Almost all (92%) of the buildings inspected in the CDEM study were documented as having parapets on at least one side of the building.

Exterior Wall Materials and Vertical Geometry

Russell and Ingham [13] determined that most URBM buildings in New Zealand were constructed of clay brick masonry (approximately 230 x 110 x 76 mm bricks) with solid walls being three leaves thick. Wall thicknesses in the base storey were typically increased by one leaf per every two storeys in height, such that the lower wall of a six-storey URM building may be six leaves thick. Mortar used between foundations and the walls in Auckland URM buildings was likely to include red scoria ash, sand and hydraulic lime that was ground in a mortar mill [31]. Mortar between bricks was likely to include either lime or cement as the adhesive agent, with a wide variety of sand particles depending on how close any given building was to ocean beaches and river banks, as the sand was usually taken from nearby the construction site [13]. Lime-based mortars do not perform as well over time compared to cement-based mortars, especially in buildings near the ocean and exposed to higher concentrations of sea salt spray.

In the present study, wall thickness measurements at the ground storey of the buildings, including the cavity where applicable, were found to be between 230 mm and 470 mm, with the vast majority of solid walls documented as being 350 mm thick (i.e., three leaves thick). The typical construction of a two-storey wall includes an inset ledge (and hence, reduced thickness) at the upper storey floor levels to permit floor bearing. A cross section of typical cavity wall construction (reproduced from AS 3700:2011 [30]) is included in Figure 7, although diaphragm-to-wall ties were rarely observed within the study group. The distributions of wall and parapet heights across the portfolio of 206 inspected buildings are illustrated in Figure 8.

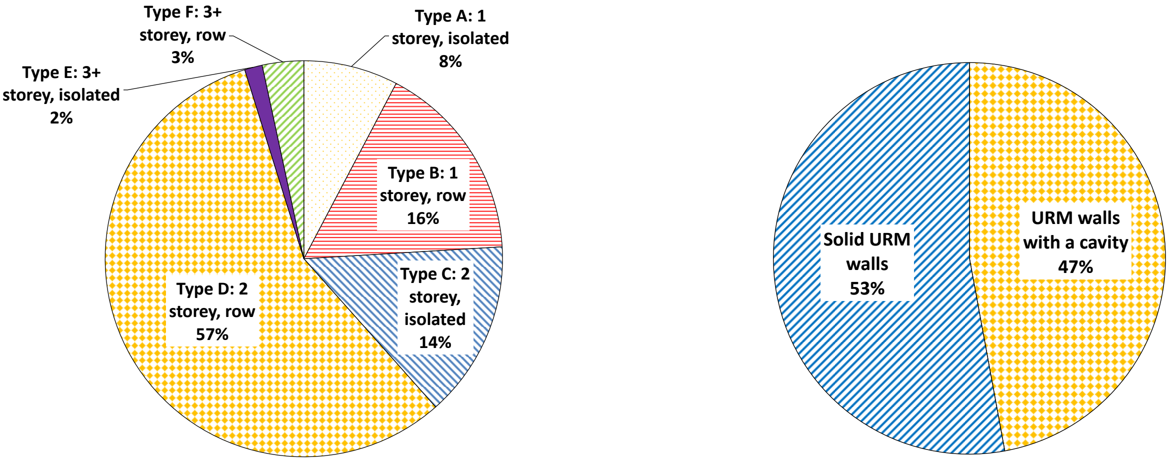
(a) Typology A - one-storey isolated building.

(b) Typology B - one-storey row building.

(c) Typology C – two-storey isolated building.

(d) Typology D – two-storey row building.

Figure 5: Most prominent sub-typologies documented from the CDEM study as classified by Russell and Ingham [13].



(a) Proportions of URBM buildings documented for the CDEM study within each sub-typology per Russell and Ingham [13].

(b) Proportions of URBM buildings documented for the CDEM study by presence of a cavity within the load-bearing wall.

Figure 6: Summary of typological proportions from CDEM study.

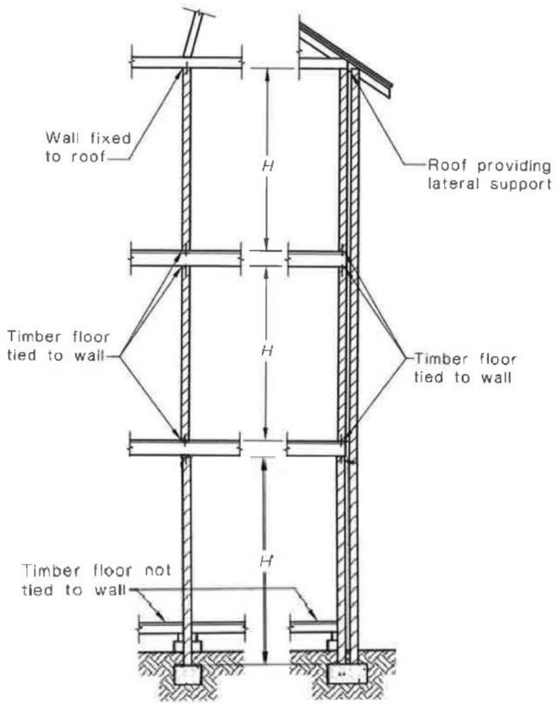
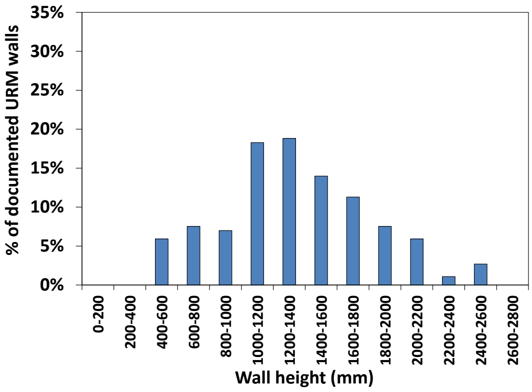


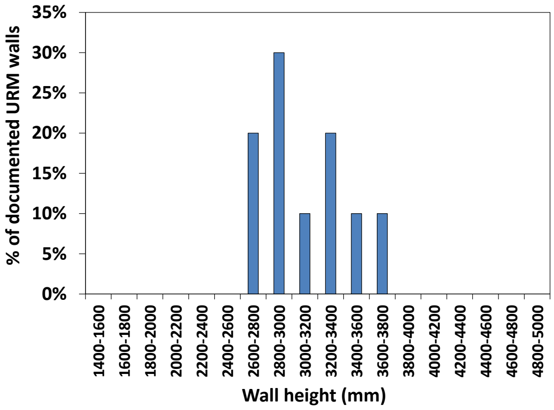
Figure 7: Typical cross-section of URBM building with exterior cavity wall, timber diaphragms and inset diaphragm restraints but without parapet [30].

The following notions are relevant to how the wall geometries were measured:

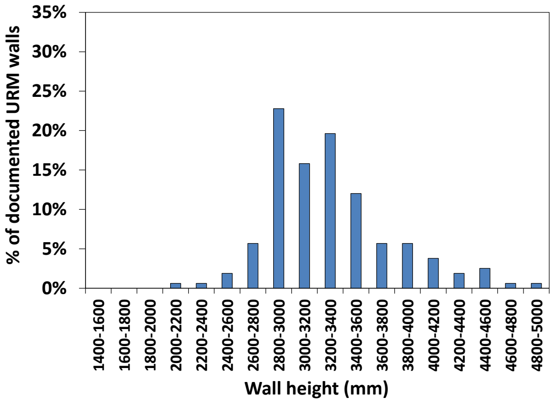
- Buildings were measured using a laser measurer. If the laser was unable to measure a certain height, reference heights were measured such that the desired height could be determined by estimate from photographs and proportionality;
- Typical URM buildings have front walls that are open at ground level (with windows above in two storey buildings). These openings disrupt the unit length method of analysis utilised in this study. Hence, for purposes of assessing vertical flexural capacity, the walls were divided into vertical "strips" that were not interrupted by windows and these "strips" were assessed. Generally, the side walls of the buildings inspected did not have openings, so "strips" did not need to be considered;
- Previous observations and studies of the effects of earthquakes on URM buildings have led researchers to determine that more concentrated, intense forces are located at the corners of buildings which are different to those experienced by walls and are a result of the interaction between in-plane and out-of-plane loading. However, in the assessment methods utilised for out-of-plane loading, the behaviour of corners was not explicitly considered beyond the rotational restraint condition for two-way flexure; and
- The base of the parapet relative to the roof level generally cannot be viewed from the outside of the building. To find the base, an aerial satellite view from Google Maps^{TM} was utilised as it provides images of the roof. These aerial images were then referenced to measurements from the ground in order to determine parapet heights.



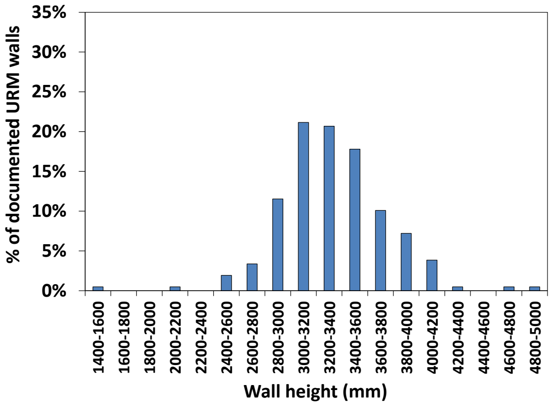
(a) Parapets.



(b) Walls of second elevated floor.



(c) Walls of first elevated floor.



(d) Walls of ground floor.

Figure 8: URBM wall heights and corresponding proportions of those walls documented for the CDEM study.

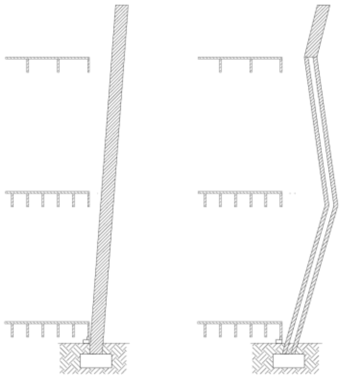
Vertical Exterior Wall Conditions and Assessment Parameters

The measured walls were assessed for different restraint conditions in order to gauge the effects of different retrofit solutions on the inspected building stock. The different restraint cases considered are illustrated in Figure 9 and described as follows:

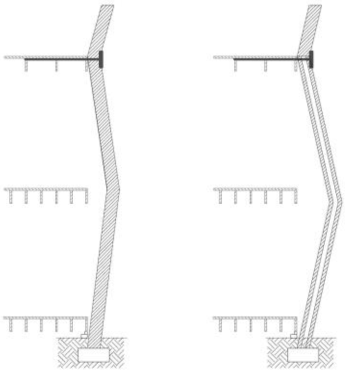
- Case 1: No above-ground diaphragm restraints - the wall is restrained at ground level by nature of the shear friction provided by the foundation and/or the lateral resistance of the soil itself, but the wall has no restraints along its height above-grade. This condition effectively enables the full height of the wall to act as a cantilever. Note that the exterior leaves of cavity walls were assumed to be restrained at the roof level by either roof framing or by the presence of solid parapets. Case 1 is the base case for all buildings;
- Case 2: Diaphragm restraint at roof level only - the wall was simply supported at its ends and the top of the parapet would act as a cantilever. In this situation, parapets were more likely to be the critical, governing element compared to the wall below the parapet. Note that, for cavity walls, Case 1 and Case 2 were treated identically in the analysis as the outer leaf below the parapet was assumed to be simply supported in both cases;
- Case 3: Diaphragm restraint at roof level and parapet restraint – the wall below the parapet was more likely to be the critical element, especially in multi-storey buildings, because both the wall and the parapet were assumed to be simply supported and the parapet was generally smaller in height;
- Case 4: Diaphragm restraint at roof level and intermediate floor levels and parapet restraint - this condition is applicable to multi-storey buildings only; and
- Each of the restraint cases (e.g., 1 - 4) was also considered without cavity restraint ties to physically connect the two cavities together (noted as sub-case “A”) and with cavity restraint ties (noted as sub-case “B”), respectively. For example, Case 4B is Case 4 with cavity restraints. Sub-case B was assumed to represent fully composite action where the outside-to-outside thickness was assumed to represent the cavity wall, which is a slightly liberal assumption. However, given the inability of the researchers to accurately measure most cavity widths and the variance observed when cavity widths were measured, a conservatively small cavity width of 10 mm was assumed in most cases (e.g., a cavity wall with two 110-mm wide bricks was assumed to be 110-mm thick in sub-case A and 230-mm thick in sub-case B).

The following assumptions were made as a basis for analysis of the portfolio of 206 buildings:

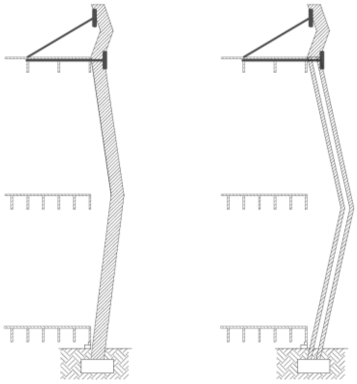
- For purposes of the assessment considered in the CDEM study, the in situ (i.e., existing) condition of all buildings was assumed to be Case 1A. Any original cavity restraints were assumed to be corroded so as to be ineffective. Existing diaphragm restraints were evident at roof levels in some walls, but were neglected due to uncertainties in condition, and no multi-storey walls were observed to have restraints at the inter-storey levels. The diaphragms themselves might provide limited lateral restraint to solid walls through bearing friction, but typical construction practices in row buildings preclude diaphragm beams



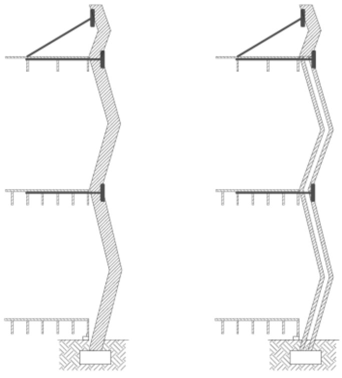
(a) Case 1 – No above-ground wall restraints.



(b) Case 2 – Roof diaphragm-to-wall connections.



(c) Case 3 - Roof diaphragm-to-wall connections and parapet restraints.



(d) Case 4 –Diaphragm-to-wall connections at all levels and parapet restraints.

Figure 9: ***URBM wall out-of-plane collapse mechanisms in vertical flexure according to different lateral restraint conditions (solid wall left and cavity wall right).***

resting on the short-length front and rear walls (relative to the street location) as shown in Figure 9. The majority of inspected buildings were observed to have unrestrained parapets. Furthermore, because pre-2011 parapet restraints were observed to be unreliable in Christchurch [6], any parapet restraints observed in the CDEM study were considered unreliable;

- The site hazard coefficient per NZS 1170.5:2004 [34] for a period equalling zero seconds, *C(0)*, effectively represents the peak ground acceleration (PGA), such that the performance of the building stock at different PGAs can be efficiently considered;
- Masonry density was assumed as 1700 kg/m^3, which is a medium-weight density per Lumantarna *et al.* [32, 33]. Assumptions regarding other material properties are summarised in Table 5;
- In accordance with NZS 1170.5:2004 [34], the part risk factor (R_p) was assumed to equal 1.0, representing a part posing a potential hazard to life outside the structure; and
- Vertical wall strips were assumed to deform through rocking mechanisms and these mechanisms are sensitive to dynamic inputs. Derakhshan *et al.* [25] prescribe capacity reduction factors of 0.50 for simply supported vertical wall strips and 0.25 for cantilevered vertical wall strips that account for the generic vertical wall strip models remaining stable for seven dynamic loading scenarios as prescribed for the Auckland region by Oyarzo-Vera *et al.* [35]. Hence, these capacity reduction factors represent a lower-bound "characteristic" performance (in this case, effectively, a conservative, 99th-percentile characteristic based on the limited number – i.e., seven - of the selected ground motion records). A review of the relationship between potential capacity reduction factors and the percentage of the models that remain stable under the various dynamic loading scenarios resulted in 50th-percentile (i.e., 50% of the models remained stable) capacity reduction factors being estimated as 0.75 for simply supported vertical wall strips and 0.55 for cantilevered vertical wall strips. The considered capacity reduction factors are summarised in Table 5.

Building Plan Geometry and Two-Way Flexure

In order to develop accurate fragility curves representative of Auckland's typological URBM building stock, additional capacity limit states were considered in addition to the vertical flexure "wall strip" method proposed by Derakhshan *et al.* [25]. Recognition for the need for these additional analyses arose from the unrealistically low seismic capacities initially obtained when considering three storey cavity walls having no reliable cavity restraint, and hence a higher effective slenderness ratio (e.g., 10,500 mm / 110 mm = 95). A virtual work-based, two-way flexural analysis, to include weighted components of horizontal flexure (i.e., flexure about a vertical axis) and diagonal flexure, was conducted by implementing the design procedure of AS 3700:2011 [30]. All walls were assessed considering both vertical flexure and two-way flexure. The capacity limit state with the higher value between vertical flexure and two-way flexure was presumed to govern for any specific wall panel given the inherent conservativeness in the idealised failure modes presumed by each method. Two-way flexure was considered for the parapet with the top edge considered laterally restrained for Cases 3 and 4.

Griffith and Vaculik [36] validated the accuracy of the AS 3700:2011 method with empirical testing, provided that return walls were assumed to provide only partial moment restraint such that the vertical edge restraint factor, R_f, equalled 0.5. For the Auckland CDEM study, assumed masonry unit dimensions were consistent with those used by Griffith and Vaculik [36] and with the design examples of Think Brick Australia [37]. Material properties were determined from either the AS 3700 design standard [30, 37] or with reported empirical testing results from New Zealand URBM buildings [38]. The material properties assumed for the Auckland CDEM study are summarised in Table 5.

Table 5. Assumed material properties and capacity reduction factors

Component	50th percentile value [ref.]	Lower-bound characteristic value [ref.]
Capacity reduction factors		
Vertical flexure, simply supported	0.75 [25, 35]	0.5 [25]
Vertical flexure, cantilevered	0.55 [25, 35]	0.25 [25]
Two-way flexure	1.0 [36]	0.6 [30, 37]
Material properties		
Brick compression strength, f_b (MPa)	17.0 [38]	-
Brick lateral modulus of rupture, f_{ut}, *MoR* (MPa)	2.0 [38, by equation]	0.8 [30, 37]
Mortar compression strength, f_j (MPa)	-	0.5 [38]
Masonry flexural tensile/bond strength, f_{mt} or f_{fb} (MPa)	0.2 [30, 37]	0.02 [38, by equation]

The masonry flexural tensile (or bond) strength was assumed to have a lower-bound characteristic of 0.02 MPa as derived from the empirical study [38], but the 50th percentile value was assumed to be 0.2 MPa as recommended by the AS 3700 standard [30, 37]. The material strengths recommended by the design standard for new construction would normally be assumed to represent the lower-bound characteristic strengths, but it was assumed in this case that contributions from overburden and bed joint shear to the flexural bond strength were not appropriately represented by the lower value for flexural tensile strength from the empirical study [38] when considered in the context of an in situ building rather than an isolated masonry assembly. For comparison, Griffith and Vaculik [36] determined a mean flexural tensile strength of 0.61 MPa for their test specimens.

Note that the results of the analysis procedure are sensitive to the assumed material strengths. Considering Case 1A, if no other values but the assumed material strengths are altered in the analysis (including maintaining the capacity reduction factors as their 50th percentile values) and the assumed material strengths are changed from their characteristic values to 50th percentile values, the percentage of buildings for which two-way flexure governs over vertical flexure greatly increases (from 31% to 82%). The buildings with assessment scores most improved by considering the two-way flexural capacity are row buildings with relatively short end walls (i.e., *L2* in Figure 10).

Plan dimensions were not recorded during the inspection of the 206 assessed buildings, so plan dimensions were assumed to correlate with the mean values for each typology as recorded by Russell [39]. The plan and elevation for the most prominent typology within the inspected building stock, Typology D, is included in Figure 10 and mean plan dimensions for all typologies are summarised in Table 6.

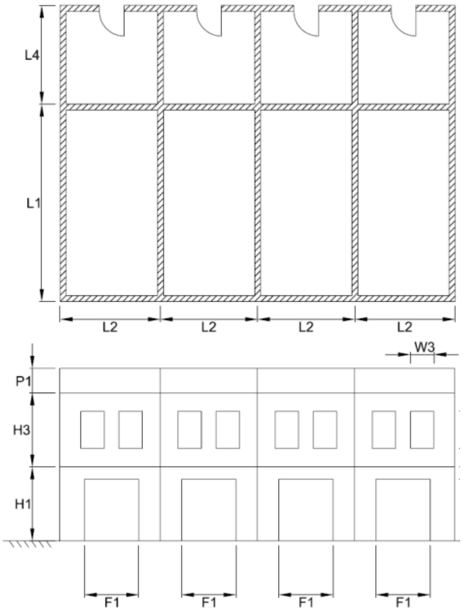


Figure 10. Plan and elevation of Typology D building [39].

Table 6. Mean plan dimensions based on building typology [39]

Type	Overall length between return walls (*L1*)	Width between return walls (*L2*)	Assumed L_d for horizontal flexure [30]
A	8000	6000	4000
B	8000	8000	4000
C	15000	7000	7500
D	12000	6000	3000
E	20000	16000	10000
F	20000	16000	8000

The symbol L_d in Table 6 represents the design length of the wall in two-way flexure per AS 3700:2011 [30]. For isolated buildings (types A, C and E), L_d was assumed to equal the maximum of the plan dimensions (*L1* and *L2*) divided by two (per AS 3700:2011 [30]), assuming a wall without perforations that is laterally supported at its vertical edges. For row buildings (types B, D and F), L_d was assumed equal to *L2* divided by two, assuming that the perpendicular walls would be restrained against out-of-plane collapse by neighbouring buildings. Assumptions for restraint conditions at the top edges of the walls corresponded with the same assumptions for vertical flexure restraints (i.e., Cases 1, 2, 3 and 4) per Figure 9.

Assumed Seismic Hazard Conditions for Assessment

Two sources were considered for determining the seismic hazard for this analysis. The standards-based hazard is prescribed by NZS 1170.5:2004 [34] and is derived from the 2002 National Seismic Hazard Model (NSHM) proposed by Stirling *et al.* [40], but with a conservativeness associated with the national uniform hazard spectral shapes utilised by the NZS 1170.5 standard. The other, generally less conservative, source for hazard information is the 2010 NSHM [41, 42]. The PGAs associated with the 2010 NSHM are based on the Reasenberg declustering method [41]. The difference between the models is largely explained by changes to the b-value (defining the relationship between earthquake magnitudes and return periods) and assumed focal depth related to the distributed seismic hazard model assumed for New Zealand, which account for a large portion of Auckland’s aggregated hazard.

PGAs for both models are included in Table 7 along with the associated return periods and annual probabilities of exceedance. Note that the PGAs associated with the 2010 NSHM are applicable to the Auckland CBD and may be higher for South Auckland. In both cases, the hazard intensities are based on shallow soil sites (site subsoil class “C” per NZS 1170.0:2002 [21]).

Table 7. Return periods and associated intensities for Auckland assuming shallow soil [34, 41, 42]

Return period (years)	10	50	150	500	1000	2500
Annual probability of exceedance	9.5%	2.0%	0.66%	0.20%	0.10%	0.04%
NZS 1170.5:2004 *C(0)* ≈ PGA (g)	-	0.061	0.101	0.173	0.225	0.311
NSHM 2010 PGA (g)	0.007	0.021	0.038	0.075	0.103	0.157

Analytical Fragility Curves for URBM Exterior Wall Out-of-Plane Seismic Stability

Seismic assessment procedures formally utilised in New Zealand entail a scoring system of percent New Building Standard (%NBS) as proposed by NZSEE [12], which indicates the expected capacity of the building as a percentage of the ultimate limit state (ULS) demands prescribed by current standards [34]. The phrase “new building standard” is indicative of the intent of the scoring system - a building that is assessed as having a resistance exceeding 100%NBS is expected to withstand the current ULS “design basis earthquake” (DBE) demands, whereas a building assessed at 33%NBS is expected to withstand only one-third of the DBE. A building with a score of less than 33%NBS is deemed “earthquake-prone” and is potentially subject to regulatory measures per the Building Act [10] and current Auckland Council policy [11], warranting further assessment and possibly structural retrofits. A building with a score less than 67%NBS is deemed “earthquake risk” and is potentially subject to the provisions of the Health and Safety in Employment Act [43]. Note that the earthquake defined by the NZS 1170.5 loadings standard [34] as the DBE for any particular building is influenced by a number of factors, including the location, site conditions and functional purpose of the building being considered. Note also that the correlation between %NBS ratings determined for existing, older buildings and those determined for newly designed buildings can be skewed by, amongst other factors, differences in characteristic strengths presumed and factors of safety utilised [44].

%NBS is widely used in New Zealand as a term to generically represent the capacity/demand ratio. Because %NBS is named in reference to the "standard" [34], whereas Derakhshan *et al.* [25] recommend different spectral demands, the term "capacity/demand ratio" is used here in place of %NBS where appropriate in order to avoid misrepresenting results as directly applicable to the standard. While the NZS 1170.5:2004 [34] spectrum represents the current standard in New Zealand, it is based on the following assumptions and scenarios [45]:

- structure comprised of steel and/or concrete;
- buildings 3, 10, or 20 storeys in height;
- ductility factor range of 3 - 6;
- inelastic behaviour included; and
- structural performance factor of 0.7.

These assumptions do not align well with the properties of a URBM building. Hence, Derakhshan *et al.* [25] offer an alternative response spectrum for "parts and components" that is specific to unreinforced masonry buildings. In particular, Derakhshan et al. include the effect of lower building heights and hence smaller natural period. Derakhshan *et al.* also assume out-of-plane loaded wall failures occurring prior to the return walls exhibiting inelastic behaviour, precluding any advantage from potential building ductility. The part spectral shape factor, $C_p(T_p)$, was determined for two-way flexure in the same manner as for one-way vertical flexure with the exception that the component period of vibration for walls in two-way flexure, T_p, was assumed to be less than 0.5 seconds in all cases due to the increased stiffness of walls behaving in two-way flexure.

The effect of the influential geometric factors to the out-of-plane stability of URM walls can be seen in Figure 11 in terms of capacity/demand ratio. The base building for this parametric study was a two-storey row URBM building (Typology D) with wall heights of 3280 mm on each level, parapet height of 1,300 mm, solid wall thickness of 350 mm on the ground level, solid wall thickness of 230 mm on the first floor level and a solid parapet thickness of 230 mm. These values represent the approximate averages for the buildings surveyed (where such dimensions were applicable).

For this sensitivity study the hazard model associated with NZS 1170.5:2004 [34] was utilised and it was assumed that the buildings were located on shallow soil sites (site subsoil class "C" per NZS 1170.0:2002 [21]), the seismic hazard factor was Z = 0.13, the return period was 1/500 (see Table 4), the return period factor was R = 1.0 and the near-fault factor was $N(T,D)$ = 1.0 (NZS 1170.5.2004 [34]). These assumptions result in $C(0) \approx$ PGA = 0.173g, as shown in Table 7. However, this PGA was scaled up the building height using the Derakhshan *et al.* [25] response spectrum for "parts and components".

50th percentile reduction factors and material strengths (see Table 5) have been assumed in the analysis, producing the results shown in Table 8 and Figure 11, such that these results are most likely to represent the actual capacities of the base building in response to an applied PGA of 0.173g. The capacity/demand ratios associated with subjecting the base building to a PGA of 0.173g are summarised in Table 8 for each restraint case (see Figure 9). Only the results from Cases 1 and 2 are illustrated in Figure 11. The governing capacity/demand ratio in Table 8 is determined by the logical hierarchy as previously described wherein the maximum of vertical flexure and two-way flexure is assumed to represent the capacity of each wall panel between lateral restraints, and the minimum capacity of all wall panels in the building governs overall.

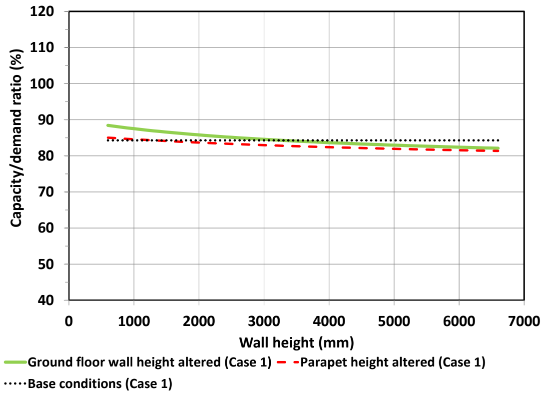
Table 8. Capacity/demand ratio for base building in sensitivity analysis

Case	Vertical flexure of parapet	Vertical flexure of walls below parapet	Two-way flexure of parapet	Two-way flexure of walls below parapet	Governing ratio
1	19%	-	84%	-	84%
2	37%	104%	103%	108%	103%
3	131%	104%	183%	108%	108%
4	131%	123%	183%	202%	183%

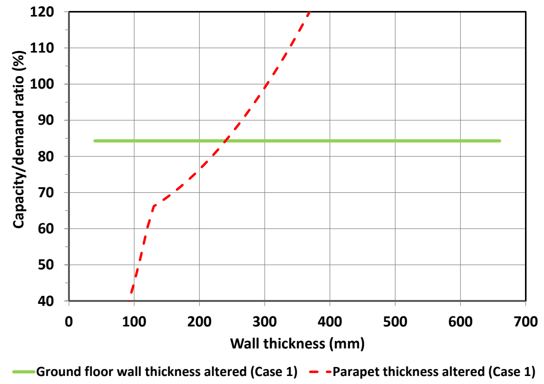
For Case 1 in Figures 11(a) and 11(b), the most influential geometric characteristic is the parapet thickness (positive correlation) which is the critical component in the calculation of parapet capacity for two-way flexure, and two-way flexure governs in Case 1 more often than does vertical flexure. Ground floor wall height and parapet height are slightly less influential (negative correlation) and affect the applied demand more so than the capacity, and ground floor wall thickness is non-influential within this range for the base building in Case 1. Note that these relative sensitivities are particular for this restraint case, base building geometry and assumed material properties. Changes to any of these variables may affect the relative sensitivities.

For Case 2 in Figures 11(c) and 11(d), the most influential geometric characteristics are parapet height (negative correlation) and parapet thickness (positive correlation). Ground floor wall height is only slightly influential (negative correlation) and ground floor wall thickness is non-influential within this range. Given that the wall below the roof level is restrained laterally top and bottom in Case 2, whereas the parapet still acts as a cantilever, these sensitivities are appropriate. Once again, these relative sensitivities are applicable only to buildings similar in geometry to the base building. A building with unusually high or thin (i.e., unrestrained cavity) walls below the parapet will be more sensitive to changes in wall dimensions. Furthermore, relative differences in geometries between wall and parapet may alter the governing limit state (i.e., whether the wall or the parapet collapses firstly).

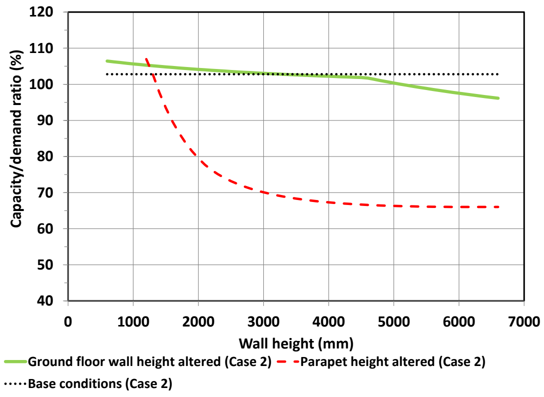
Summaries of the expected performance of the URBM buildings inspected in the CDEM study considering the two most extreme restraint cases (Case 1A with no restraints and Case 4B with all restraints, including cavity restraints) are included in Figures 12(a) and 12(b). Results produced using both response "parts and components" spectra [25, 34] are included in Figures 12(a) and 12(b) as a reference for engineers completing detailed seismic assessments of URBM buildings in Auckland with reporting intended to be used for regulatory purposes.



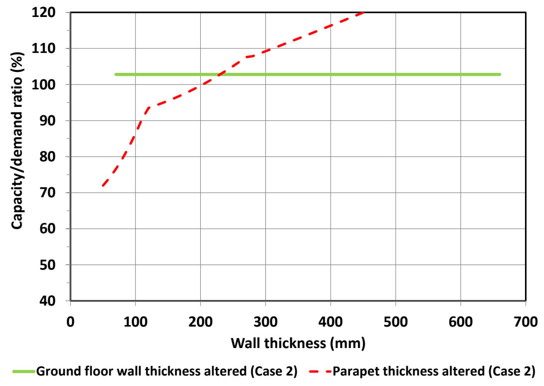
(a) Influence of wall height on wall stability (Case 1).



(b) Influence of wall thickness on wall stability (Case 1).



(c) Influence of wall height on wall stability (Case 2).



(d) Influence of wall thickness on wall stability (Case 2).

Figure 11: Sensitivity of capacity/demand ratio to different conditions (50th percentile performance, PGA = 0.173g, Derakhshan et al. 2014 [25] "parts and components" spectrum).

The 206 buildings inspected in the CDEM study are considered to be representative of the majority of Auckland's URBM building stock as they are all comprised of brick masonry, presumed to be unretrofitted and are 1 to 3 storeys in height. The cases shown in Figures 12(a) and 12(b) represent the bounds of expected performance across the portfolio, as cases representing intermediate extents of restraints will produce distributions where the percentage of buildings failing will be somewhere between the percentages determined assuming Case 1A and Case 4B. A selection of such cases and their expected performance distributions are shown in Figures 12(c) - 12(f). Note that there is a limit to the effectiveness of the retrofit strategy to increase the extent of lateral restraints to prevent out-of-plane wall failure, and even fully restrained buildings (Case 4B) may be susceptible to out-of-plane wall collapse at high intensities (e.g., PGAs associated with return periods of 2500 years or more). In order to more effectively improve buildings to sustain high levels of earthquake intensity, more invasive retrofit procedures would need to be considered [26, 27, 28, 29].

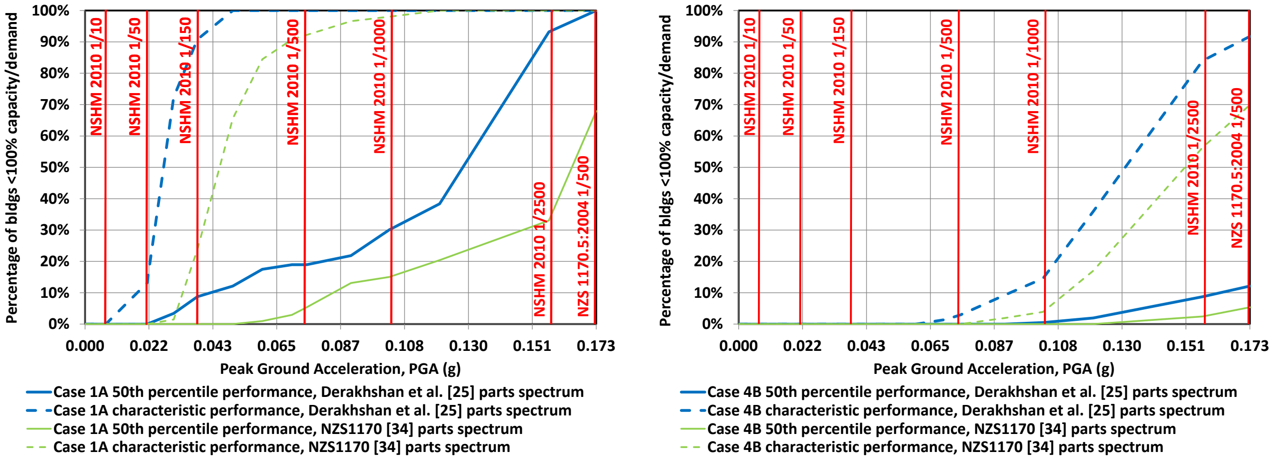
SPATIAL ANALYSIS AND ESTIMATED DISTRIBUTION OF OUT-OF-PLANE URBM WALL COLLAPSES IN AUCKLAND

The AC / UoA database of documented URM buildings was filtered based on the following criteria:

- URBM;
- 1 - 3 storeys; and
- Construction year either pre-1960 or unknown (presuming that anything known to be newer would represent a building that was retrofitted).

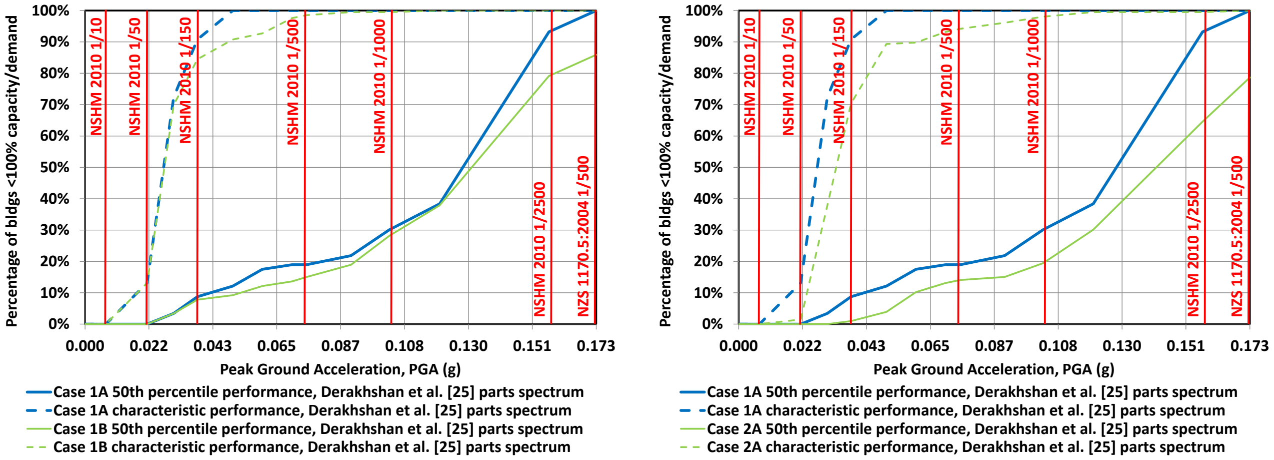
This filtering produced a total of 763 documented buildings located in the Auckland region (mapped in Figure 13) whose performance could be estimated using the fragility curves of Figure 12. In a spatially-based intensity assignment using ESRI's ArcMap^{TM} and proprietary software from GNS Science, MM intensities were determined for each of the 763 buildings in each of four earthquake scenarios as follows:

- 1891 Waikato Heads earthquake with magnitude M_w 6.2 which caused the highest intensities on record in Auckland's CBD. (Note that this event pre-dates the construction of most Auckland URBM buildings);
- Hypothetical earthquake caused by the rupture of the Wairoa North fault with magnitude M_w 6.7 normal fault rupture at 6 km depth and a return period of 12,600 years, representing the maximum credible earthquake originating from the nearest major active fault to Auckland's CBD [46];
- Hypothetical earthquake caused by the rupture of the Wairoa North fault with magnitude M_w 6.49, causing 100%NBS demands per NZS 1170.5 [34] in the Auckland CBD (PGA = 0.173g on a shallow soil site at 12 Wyndham Street); and
- Hypothetical earthquake caused by the rupture of the Wairoa North fault with magnitude M_w 5.36, causing 33%NBS demands per NZS 1170.5 [34] in the Auckland CBD (PGA = 0.058g on a shallow soil site at 12 Wyndham Street).



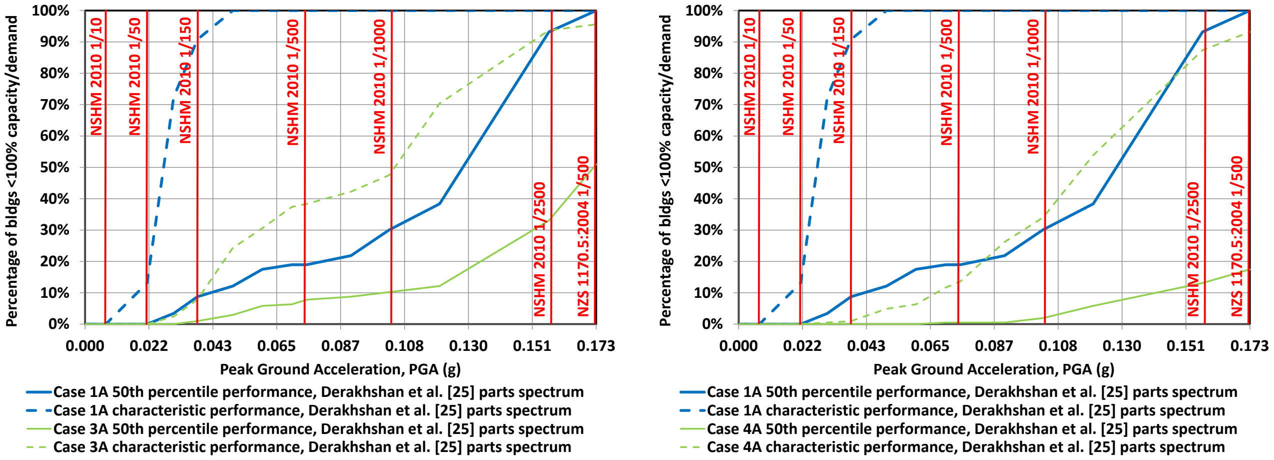
(a) Case 1A (no restraints) comparing results from two response spectra [25, 34].

(b) Case 4B (roof, inter-storey diaphragm, parapet and cavity restraints) comparing results from two response spectra [25, 34].



(c) Case 1A (no restraints) versus Case 1B (cavity restraints) (includes all buildings, with and without cavities).

(d) Case 1A (no restraints) versus Case 2A (roof diaphragm restraints, no cavity restraints).



(e) Case 1A (no restraints) versus Case 3A (roof diaphragm and parapet restraints, no cavity restraints).

(f) Case 1A (no restraints) versus Case 4A (roof and inter-storey diaphragm and parapet restraints, no cavity restraints).

Figure 12: ***Percent of URBM buildings failing (i.e., capacity/demand < 100%) when subjected to varying PGAs, considering different restraint cases, capacity reduction factors and return periods (latter indicated by vertical red lines).***

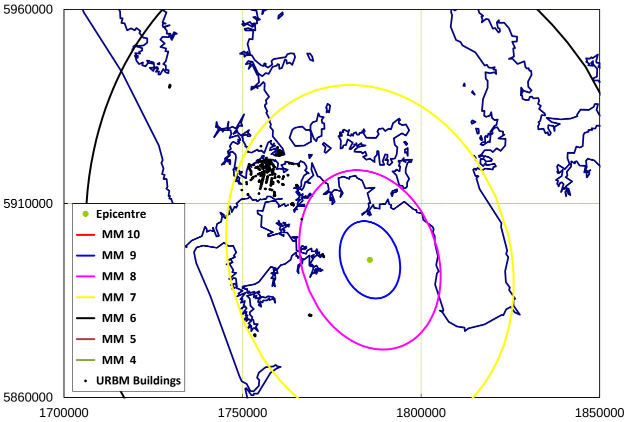


***Figure 13:** Mapped locations of 763 documented URBM buildings in Auckland and intensity contours from maximum credible earthquake originating from the Wairoa North fault.*

The proprietary software from GNS Science accounted for soil amplifications in specific geographic locations, particularly for buildings sitting on landfill near the waterfront in the CBD. The MM intensities were converted into PGAs using the conversions in Table 9, per the recommendations of GNS Science.

Table 9. Modified Mercalli intensities and assumed corresponding peak ground accelerations

MM intensity	Approximate PGA (g)
4.0	0.01
5.0	0.03
6.0	0.08
7.0	0.17
8.0	0.30

For each earthquake scenario, each individual building was assigned a percentage chance of collapse by correlating its site-specific PGA intensity (based on proximity to the source and local soil characteristics) to the appropriate fragility curve in Figure 12. The percentages for all 763 buildings were then averaged in order to determine the percentage of low-rise (i.e., 1 - 3 storeys) URBM buildings in Auckland expected to experience out-of-plane wall collapses in each earthquake scenario. Three different fragility curves were considered, all representing 50th percentile performance using the Derakhshan *et al.* [25] "parts and components" spectrum but each with different extents of lateral restraint conditions. The results are summarised in Table 10.

Table 10. Earthquake scenarios considered and corresponding estimated proportions of out-of-plane (OOP) wall collapses in low-rise URBM buildings in Auckland

Earthquake scenario	Earthquake scenario relevance	Estimated percentage with OOP collapses	Estimated number with OOP collapses
Unretrofitted buildings (Case 1A)			
1891 Waikato Heads, M_w 6.2	Maximum recorded	20%	154
Wairoa North, M_w 6.7	Maximum credible	98%	749
Wairoa North, M_w 6.49	100%NBS in CBD	84%	645
Wairoa North, M_w 5.36	33%NBS in CBD	15%	115
Roofs and parapets laterally restrained (Case 3A)			
1891 Waikato Heads, M_w 6.2	Maximum recorded	6%	47
Wairoa North, M_w 6.7	Maximum credible	67%	512
Wairoa North, M_w 6.49	100%NBS in CBD	46%	350
Wairoa North, M_w 5.36	33%NBS in CBD	4%	32
Roofs, floors and parapets laterally restrained (Case 4A)			
1891 Waikato Heads, M_w 6.2	Maximum recorded	0.6%	5
Wairoa North, M_w 6.7	Maximum credible	27%	209
Wairoa North, M_w 6.49	100%NBS in CBD	18%	138
Wairoa North, M_w 5.36	33%NBS in CBD	0.4%	3

CONCLUSIONS AND RECOMMENDATIONS

The general findings from this study are as follows:

- The consideration of two-way flexure in addition to one-way vertical flexure may aid in improving estimated URBM capacity/demand ratios. Consequently, further research attention towards effective assessment procedures that account for two-way flexure is merited;
- Approximately 20% of URBM buildings in Auckland that are 1 - 3 storeys in height and unretrofitted would be expected to experience at least partial out-of-plane collapse when subjected to the peak ground acceleration intensities associated with a NHSM 2010 return period of 1 in 500 years (see Figure 12(a), Case 1A with 50th percentile performance);
- The installation of lateral restraints may significantly raise the capacities of buildings at most considered levels of intensity (see Figure 12, all cases);
- A capacity limit at high intensities is reached by the diaphragm restraint retrofit solutions considered in this study, particularly when lower-bound "characteristic" performance is assumed, as walls between restraints at each storey will still fail out of plane at high lateral load demands (see Figure 12, all cases). In order to more effectively improve buildings to sustain high levels of intensity (and achieve acceptable levels of %NBS), more invasive retrofit procedures may need to be considered [26, 27, 28, 29]; and
- An earthquake originating from the Wairoa North fault causing 33%NBS demands in the Auckland CBD on shallow soil sites would be expected to cause out-of-plane wall collapse of 15% of low-rise, unretrofitted URBM buildings in Auckland. The estimated percentage of low-rise URBM buildings experiencing out-of-plane collapses in this 33%NBS earthquake scenario drops to 4% if a comprehensive roof and parapet restraint program were to be instituted and drops to 0.4% if adequate lateral

restraints are added at the parapet, roof and floor levels. Buildings in the CBD near the waterfront are expected to be subjected to increased ground intensities due to the assumed amplifications of the soft soils there.

The capacities as reported here have been determined using either a displacement-based assessment procedure [24, 25] or a virtual-work based assessment procedure [30]. Most practitioners find simplified force-based procedures to be more convenient. The capacity/demand ratios determined from a force-based approach may be more conservative, particularly for the assessment of unrestrained parapets which behave as cantilevers. Furthermore, parapet restraint capacities are often limited by the shear capacity of the in-situ bricks and mortar.

Finally, the authors recommend that seismic hazard modellers provide guidance to engineers and civil defence officials regarding the use of NZS 1170.5:2004 [34] seismic demands compared to the updated NSHM 2010 [41, 42] seismic demands in order to most accurately consider the return periods of events, such that seismic risks can be compared to other risks (e.g., cyclones, tornadoes, tsunamis, etc.).

FUTURE WORK

Other data relevant to URM building typological classifications in Auckland are also being procured. Information pertaining to the shape of the building plan, structural irregularities, foundation system, roof material and geometry and floor system is being accrued through structural inspections. Consequence-related information such as building condition, floor area, footprint area, number of occupants and replacement costs is available through other sources (e.g., Quotable Value NZ).

Further work on URM out-of-plane assessments will involve performing further spatially-oriented analyses to include additional buildings expected to be documented in more rural areas of the Auckland region. An extrapolation of these results to the URM building population across New Zealand is also intended in order to answer the following questions:

- If a nationwide effort were instituted to seismically restrain all URM parapets, what overall performance improvement would be expected?
- If a nationwide effort were instituted to provide wall-diaphragm anchorages to all URM buildings across New Zealand, then what overall performance improvement would be expected?

Other relevant projects are currently underway within Auckland Council and at the University of Auckland as follows (in no particular order):

- Continuing taxonomical recording of Auckland commercial building typologies for all structure types;
- URM infill OOP detailed assessment methodology and empirical case study tests;
- Shake table testing of masonry parapets; and
- Unreinforced masonry exemplary seismic retrofit design and implementation into typological URM heritage buildings.

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DISCLAIMERS

The peak ground accelerations (PGAs) referenced in this study are interpreted from the 2010 NSHM [41, 42] and are intended specifically for research purposes. The results based on these intensities (as compared to those currently in use by the engineering community per NZS 1170.5:2004 [34]) do not represent a currently understood assessment of seismic building risk, as the seismic hazard is explicitly defined by NZS 1170 - the industry loadings standard representing the implementation of the Building Act [10] for these purposes. Changes to the standard regarding design seismic hazard intensities have not yet occurred and are not necessarily going to occur in the manner in which the authors are presuming here. Note also that if changes do occur to the hazards prescribed by the standard, that South Auckland may have a higher design intensity than the rest of the Auckland region.

Capacity/demand ratios reported here for all buildings are based on an importance level of 2 and a design life of 50 years, which result in a non-conservative estimation of seismic hazard per the NZS 1170 Standard for some buildings. Less conservative capacity reduction factors and material strengths have been assumed where noted in order to predict the 50th percentile performance of the building stock. However, individual buildings should be assessed by a competent structural engineer using characteristic capacity reduction factors and material strengths prescribed by the standards.

NOTATION

AC	Auckland Council;
Avg.	average;
C(0)	site hazard coefficient, assumed to represent PGA;
CBD	central business district (city centre);
CDEM	Civil Defence and Emergency Management;
CERC	Canterbury Earthquakes Royal Commission;
DBE	design basis earthquake;
Est.	estimated;
GEM	Global Earthquake Model;
IEP	Initial Evaluation Procedure;
IL	importance level;
LLRS	lateral load-resisting system;
MM	Modified Mercalli (intensity scale);
%NBS	percent New Building Standard;
NSHM	National Seismic Hazard Model;
OOP	out-of-plane;
PGA	peak ground acceleration;
URBM	unreinforced brick masonry;
UoA	University of Auckland;
ULS	ultimate limit state (design parameters);

URBM	unreinforced (clay) brick masonry;
URM	unreinforced masonry; and
URSM	unreinforced stone masonry

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