The role of in-plane strengthening within a proposed non-specific design approach to seismic improvement for URM buildings

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ABSTRACT

Recent research shows that a significant number of earthquake-prone unreinforced brick masonry buildings may not receive seismic strengthening within the deadlines mandated by the Building (Earthquake-prone Buildings) Amendment Act 2016, with at-risk buildings clustered disproportionately in provincial centres. Conventional strengthening approaches appear to be uneconomic for many of these buildings, as does from-scratch replacement of the social and cultural amenity they provide. Many URM buildings conform to standard typologies and detailing, creating an opportunity to lower the cost of strengthening for a subset of these buildings via a non-specific design methodology mapped to a set of standard details. One challenge in defining a comprehensive non-specific design scheme for URM buildings is the assessment of the in-plane capacity of perforated walls and open shopfronts. This paper examines the frequency and severity of observed in-plane damage from reconnaissance reports of recent earthquakes. It presents a structural analysis of the in-plane capacity of a representative open-fronted URM commercial building. The analysis uses displacement-based assessment to show that many New Zealand buildings of common type may be able to withstand moderate in-plane seismic demand without dedicated in-plane structural intervention. The analysis results are shown to be in good agreement with ASCE 41-17 provisions for URM buildings with open shopfronts. We propose that it may be pragmatic to omit in-plane strengthening for many typical URM building of modest dimensions which have undergone a well-formulated non-specific strengthening design and received appropriate measures competently installed.

1 INTRODUCTION

In a 2011 presentation to the Canterbury Earthquakes Royal Commission, Ingham (2011a) noted that a "do-nothing" approach to earthquake-prone unreinforced brick masonry (URM) buildings would likely lead to fatalities in future earthquakes. Ingham laid out two alternatives to the status quo: demolition of earthquake-prone URMs, which he described as "pragmatic, but the impact to the character of NZ towns and cities will be massive"; or widespread earthquake strengthening, noting that the cost of such strengthening "may be greater than the value of the current building[s]" (Ingham, 2011a).

In the years following the Royal Commission, research has been carried out supporting Ingham's predictions, both in terms of the probable impact of demolition upon communities and the deterrent effect of the cost of strengthening (for a summary see Tocher and Cutfield 2021a, 2021b). A key finding by researchers is that, for many owners of earthquake-prone URM buildings, inaction on earthquake strengthening followed by building demolition appears the most likely outcome (Aigwi et al 2019, Fillipova and Noy 2020, Nakhies 2014).

Faced with the loss of significant numbers of URM buildings particularly in regional centres, it is worth considering an alternative strategy for reducing the cost and complexity of the earthquake strengthening process. Many of the buildings in the shopping streets of New Zealand conform to standard typologies, with most being one- and two-storey standalone and row buildings (Russell and Ingham 2010). With the limited goal of preventing catastrophic structural collapse under moderate seismic loading, standard-detail-type improvements would likely be sufficient to adequately strengthen most of these buildings, particularly in regions of lower seismic hazard. Given that URM buildings demonstrate relatively limited structural variation throughout the country, it may not be necessary to carry out building-by-building analyses to determine the extent of remediation required, but instead to define a non-specific design process which can be mapped to a set of standard-detail remediations. For a subset of buildings, determined to be suitable for this process, the outcome could be a "deemed-to-adequately-comply" exemption from further seismic strengthening requirements of the EPB Act. We propose that this approach may be a pragmatic way to reduce the seismic risk from New Zealand's URM buildings without resorting to wholesale demolition. It would also promote more immediate reduction in seismic risk, noting that, under current settings, it is predicted that many URM buildings will not be strengthened at all (Aigwi et al 2019, Fillipova and Noy 2020, Nakhies 2014). For URM buildings that require a higher standard of performance improvement, or for eligible buildings for which future market drivers provide incentives for improvement, bespoke strengthening would still be available.

In prior work (Tocher and Cutfield 2021a, 2021b), we described some possible outlines of a non-specific design method for URM buildings. We proposed a system containing **baseline** measures, applied to all buildings, dealing with parapets, floor-wall and roof-wall connections, cavity walls, and canopies. For some buildings, **conditional** measures would also be applied, dealing with walls out-of-plane and with diaphragms. We have termed the proposed non-specific design process and standard detail approach **simple strengthening** (Tocher and Cutfield 2021a, 2021b).

The non-specific design process separates potentially eligible URM buildings into three categories. Firstly, buildings which are clearly suitable for standard-detail simple strengthening, for example those located in low-hazard areas. Secondly, buildings which are clearly unsuitable for simple strengthening, a determination which may be made on the basis of building geometry, for example buildings with unfavourable diaphragm aspect ratios or excessive interstorey heights. The third category contains buildings for which **many** elements of strengthening can be done with standard details, (e.g., connection strengthening) but which may require structural engineering input for the design of specific aspects of a simple strengthening scheme. An outline of the process (including triage into the categories above) is given in Tocher and Cutfield (2021b).

The proposed non-specific design scheme omits in-plane strengthening from the package of structural improvements required under simple strengthening, with the proviso that some buildings (depending on, for example, seismic hazard, building arrangement, and the length of wall in each direction) would not be eligible. This approach is conceptually similar to that adopted in the "Bolts-Plus" strengthening approach that was implemented in San Francisco (Paxton et al. 2015). In this paper, we briefly discuss engineering justifications for omitting in-plane strengthening. This is based on a review of observed earthquake damage, and on numerical calculations in accordance with the New Zealand Guidelines for the Seismic Assessment of Existing Buildings (NZSEE 2017).

2 FREQUENCY AND SEVERITY OF REPORTED IN-PLANE FAILURES

2.1 Observations from the Canterbury Earthquake Sequence

Reconnaissance reports recording damage to URM buildings from the Canterbury earthquakes show that in-plane failures occurred at a lower rate than out-of-plane failures, and that in-plane failures seldom resulted in collapse. Presenting to the Canterbury Earthquakes Royal Commission, Ingham (2011b) noted that "failures were almost entirely attributable to out-of-plane deformation mechanisms." Similarly, Moon et al. (2014) reported "relatively infrequent" in-plane failures, observing that while 56% of URM buildings showed signs of in-plane wall damage, "extreme" in-plane damage occurred in only 32 of 627 of the URM buildings studied. Ingham (2011b) itemised observed failure modes of masonry buildings. Out-of-plane failures included partial or total wall collapses, whereas observed in-plane failures were limited to cracking through lintels and spandrels. Recorded in-plane cracking damage to spandrels and piers appeared unlikely to lead to significant loss of gravity support or to the ejection of masonry units. An addendum report (Ingham and Griffith 2011b), containing summary data from observations by reconnaissance teams, noted only one in-plane partial collapse (84 Lichfield St), attributed to "weak piers". This failure occurred in the upper floor of a three-storey URM building, a typology which would not be eligible for simple strengthening.

It is noted that the Canterbury Earthquake Sequence subjected buildings to demands greater than the "moderate" (<0.34 × ULS) earthquake envisaged under the non-specific design proposal, with the 4 September 2010 event being 0.7 × ULS, the 22 February 2011 event 1.5-2 × ULS, and the 13 June event 0.9 × ULS (Clifton 2011). The limited in-plane damage reported above should be considered in light of these higher demand levels.

It should be noted that URM buildings that are strengthened against out-of-plane failure but not in-plane failure would, by design, be more likely to exhibit in-plane failure modes. That said, as described above observed in-plane failures tend towards modes which, while damaging to the structure, do not often result in collapse or the ejection of large quantities of masonry units, suggesting that in-plane failure could be considered a more "desirable" mode from a life-safety standpoint. Ingham and Griffith (2011b) analysed the performance of unstrengthened URM buildings in Christchurch against buildings having various levels of strengthening. Their report records a significant reduction in the damage index for buildings having only limited strengthening. Though many buildings with limited strengthening were seriously damaged by the high accelerations they experienced, this strengthening "did result in a significant reduction from major damage to moderate damage" (Ingham and Griffith 2011b). Taken in combination with the recorded in-plane failure types, their analysis provides some evidence of a reduction in overall hazard resulting from failures being driven towards in-plane modes.

2.2 Observed in-plane failures from international literature

The predominance of severe out-of-plane failures over severe in-plane failures reported in Canterbury can also be observed in international post-quake reconnaissance surveys (Lizundia et al 2016, Ismail and Khattak 2015, Penna et al 2013).

A significant study from Italy on the 6.3M_w 2009 Abruzzo Earthquake provides quantification of the relative severity of in-plane and out-of-plane failures. Observed damage was recorded by failure mode type and ranked for severity on the five-step EMS98 damage scale for masonry. In-plane failure modes "present a maximum peak at the level d_3 [moderate structural damage] and a drastic reduction at the level d_4 [heavy structural damage]…. In contrast, the out-of-plane mechanisms appear to be more frequent for damage level

d_4, with a frequency larger than two times the in-plane ones." As with other earthquakes, recorded in-plane damage is primarily in the form of diagonal shear cracks in the masonry piers and local crushing of the masonry (Indirli et al. 2013). The quantified damage results of the study appear to support the argument that in-plane failure modes cause damage but are less likely to result in significant structural collapse.

3 INSIGHTS FROM NUMERICAL ANALYSIS

3.1 Representative example building

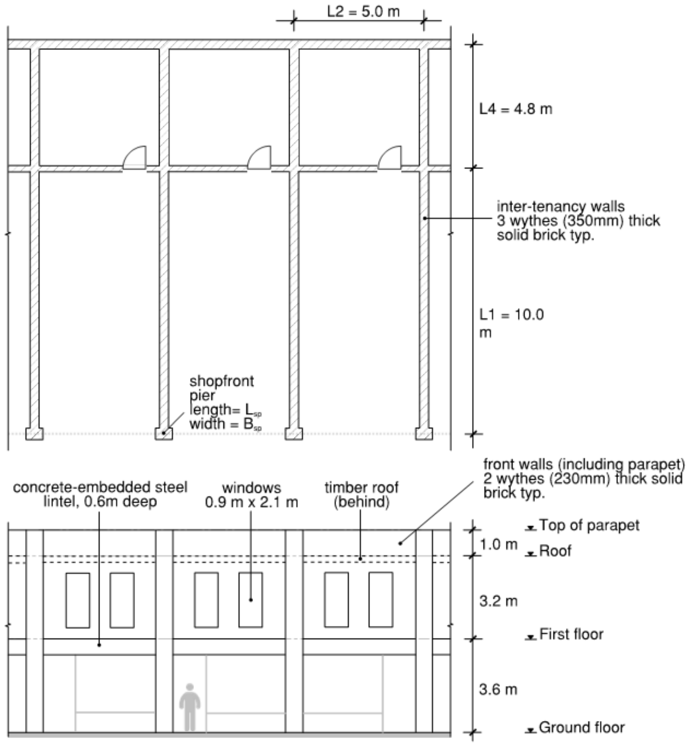

Figure 1. An example unreinforced masonry building generally representative of "Type D" New Zealand unreinforced masonry buildings with open fronts.

To understand the seismic risk associated with in-plane failure mechanisms in New Zealand's URM buildings in moderate earthquakes, it is useful to first work through analysis of some typical buildings that are representative of commonly observed building typologies. For example, consider the building shown in Figure 1. This building is typical of a Type D two storey row building as described in Russell and Ingham (2010). Russell (2010) determined the mean geometry for various typologies of URM buildings on the basis of a nationwide survey. The example building analysed in this paper uses the mean Type D geometry determined by Russell, with some values adjusted to suit an actual unreinforced "Type D" masonry building from Feilding. The representative building geometry was also checked for general consistency against another survey of URM buildings by Walsh et al. (2014). The diaphragm at the first floor level was taken as timber sheathing with a shear stiffness of 135 kN/m and a seismic weight of 1.0kPa.

Considered in-plane, the building contains long unperforated walls, long and relatively squat piers at the upper storey, and slender piers on the ground floor. These wall and pier scenarios are considered below.

3.2 Unperforated walls and squat piers

The in-plane response of unreinforced masonry walls is characterized by higher shear capacities for longer, squatter walls, and lower shear capacities for shorter, more slender walls. For example, consider a 230mm

(2- leaf, f'_m = 8.5 MPa) solid unreinforced masonry pier supporting 3.0m of solid masonry wall on the storey above and a 5.0m tributary width of light (1.0kPa) floor. Table 1 lists the in-plane shear capacities of the wall at various increasing wall lengths, with calculated capacities based on the New Zealand Guidelines for the Seismic Assessment of Existing Buildings (hereafter "the Guidelines") (NZSEE, 2017). The shorter, more slender wall piers have significantly lower capacities, both on an absolute basis and on a "per unit length" basis.

***Table 1**. Lateral capacity of a 230mm thick solid unreinforced masonry pier supporting 3.0m of masonry wall and a 5.0m tributary width of light (1.0kPa) floor.*

		Wall length (m)					
		0.8	1.2	1.6	2.0	3.0	4.0
		Lateral capacity of the pier (kN)					
Wall height, h (m)	3.0	4.4	9.9	17.5	27.4	61.6	109.5
	4.0	3.6	8.0	14.3	22.3	50.2	89.3
	5.0	3.1	6.9	12.3	19.3	43.4	77.1
		Lateral capacity of the pier per m length (kN/m)					
Wall height, h (m)	3.0	5.5	8.2	11.0	13.7	20.5	27.4
	4.0	4.5	6.7	8.9	11.2	16.7	22.3
	5.0	3.9	5.8	7.7	9.6	14.5	19.3
		Lateral capacity normalized by weight					
Wall height, h (m)	3.0	0.25	0.34	0.42	0.48	0.59	0.67
	4.0	0.19	0.25	0.30	0.34	0.41	0.46
	5.0	0.16	0.20	0.24	0.26	0.31	0.34

The weight-normalised capacities listed in Table 1 can also be compared against seismic demands from NZS 1170.5:2004. For example, the 1/500 year spectral accelerations for Auckland (Z=0.13), Whanganui (Z=0.25) and Wellington (Z=0.40) (Importance Level 2, Site Class C, 5% damped, Sp=μ=1.0) are 0.31g, 0.59g and 0.94g, respectively. Adjusting for the K_R factor from the Guidelines section C8.10.2.2 reduces these to 0.10g, 0.20g and 0.31g, respectively – with further reductions applicable if the assessment is targeted to a "moderate earthquake" i.e., to one third design level earthquake. We note that the capacities listed in Table 1 exceed the demand except in higher hazard regions for walls that are particularly slender. This result can be shown to hold over a variety of different masonry wall pier arrangements. It follows that very slender masonry wall piers represent a critical case that warrants careful consideration in the construction of non-specific strengthening schemes.

3.3 Slender piers and open shopfronts

Slender wall piers commonly appear along street frontages, particularly in shops with large open fronts such as those shown in Figure 2. To draw general conclusions around the requirements for in-plane seismic strengthening of such buildings, it is necessary to first understand how these slender wall piers behave in the context of the overall building response. We therefore present an analysis of the street-facing wall line of the representative example building shown in Figure 1.

The second storey of the example building (along the front façade) was observed to be relatively stiff compared to the slender piers below. A simple single degree of freedom approximation for the front façade using a strong spandrel-weak pier approach was therefore adopted. This allowed a pushover curve for the front façade wall line to be determined based on the response of the shopfront piers. The tributary seismic mass to the front façade was then calculated. This enabled plotting of the pushover curve against ADRS plots for Auckland, Whanganui and Wellington. An example is as shown in Figure 3 for a shopfront pier of width 350mm and length 450mm. It is noted that this pushover curve is a relatively simple approximation– it does not, for example, consider modal interactions between the façade wall and adjacent diaphragm.

Figure 2. Examples of two-storey row masonry buildings with large openings along shopfronts from (a) Oamaru and (b) Ashburton. Reproduced with permission from Russell (2010).

Figure 3 indicates that, while the elastic capacity of the shopfront piers is likely to be exceeded even in some moderate earthquakes, they have considerable displacement capacity post-yield. The necessity for in-plane seismic strengthening of these piers therefore needs to be determined with consideration of the spectral displacement resulting from the seismic demand, and its likelihood to cause lateral instability. Moment-curvature analysis suggests a stable rocking response where gravity load-carrying capacity can be achieved to drifts as high as 2.2% (eg per the Guidelines section C8.8.6.2), provided that the diaphragms are in good condition and well-connected to the perimeter wall structure. Connection enhancements and selective enhancements to the diaphragm for some buildings, forming part of a simple strengthening scheme, can be seen therefore to add in-plane resilience to the overall structure. Additional resilience is provided by the redundancy created by multiple neighbouring diaphragms in row buildings.

It should be noted that the indicative example presented here does not consider flange effects (due to the short pier lengths). Flange effects have the potential to "hold down" shopfront piers against rocking, which can reduce the piers' displacement capacity (for example, by forcing a bed joint sliding failure mode). A more detailed investigation would be needed to fully quantify the impacts of flange effects on the proposed simple strengthening approach.

Examining the results with reference to the limited objectives of simple strengthening, it can be seen that the sample pier has sufficient displacement capacity to satisfy the 34% spectral displacement demand in Auckland, Whanganui, and Wellington. This supports the premise that, for many buildings of this type, slender shopfront piers may be able to meet the regulatory benchmark of avoiding collapse in a moderate earthquake without dedicated in-plane structural intervention.

3.4 Secondary gravity support for open shopfronts

An approach to in-plane structural enhancement in keeping with the simple strengthening methodology is suggested by engineering practice from the USA. The Los Angeles Division 88 Ordinance requires "beams other than rafters or joists" to be supported on "independent secondary columns" (LA Municipal Code 1985). This approach was put to the test in the 5.9M_w 1987 Whittier Narrows earthquake. A report, written by engineers who had carried out post-disaster reconnaissance and had also examined collated damage reports from the recovery period, concluded that “the installation of gravity framing systems…apparently prevented collapse of many buildings” (Moore et al 1988). The case study buildings in the 1988 report use timber posts for gravity support. A possible addition to the simple strengthening measures might be the installation of timber posts to support existing concrete or steel lintel beams in the open shopfronts of URM buildings. It is not intended that timber gravity posts would contribute to the stiffness and shear strength of the wall line, but that they would have sufficient robustness to provide reliable displacement capacity and create a secondary gravity load path, if masonry elements were seriously damaged.

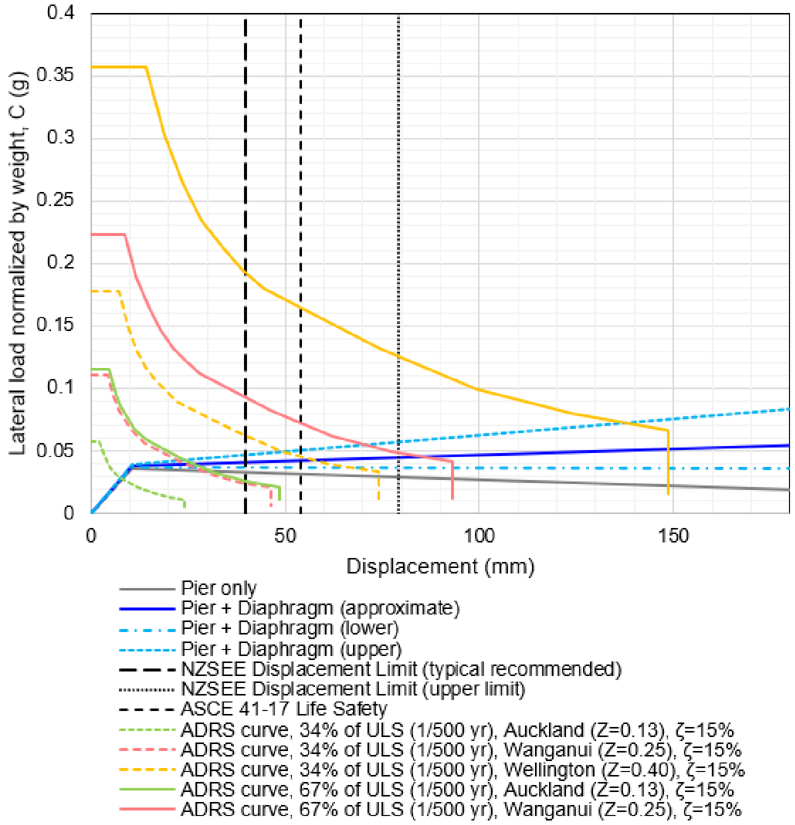

***Figure 3**. A plot of the pushover curve for the front façade of the simple example building*

3.5 Comparison to assessment methods from the USA

ASCE 41-17 Chapter 11 and Section 16.2 provides a simple design methodology to assess masonry buildings with open fronts against life safety criteria (ASCE, 2017). The ASCE approach was applied to the example building shown in Figure 2, using a number of different sizes of shopfront pier geometries with widths ranging between 230mm and 450mm and lengths ranging between 350mm and 680mm. Applied to the example problem noted above, based on the requirement to achieve a low probability of collapse in a “moderate” earthquake (one third as strong as the design level earthquake), it followed that seismic strengthening of piers was not required for Auckland or Christchurch. In-plane strengthening was similarly not required for Wellington – however, for this case, the building did require plywood diaphragm strengthening. These calculations to ASCE 41-17 rely critically on the rear amenity block being constructed of at least two lines of strong, sturdy masonry wall, and on well-connected timber diaphragms at all levels. The authors note that criteria for simple strengthening eligibility around these points would need to be clearly stated and explained.

3.6 Notes towards sorting criteria for URM buildings not requiring in-plane measures

A non-specific design system requires methods for triaging buildings. While further analysis of representative buildings of varying typologies is required, it is proposed that sorting criteria could be defined and used to identify URM buildings which do not require in-plane strengthening under the non-specific design rubric. The eligibility criteria could be adapted from those used for the Bolts Plus strengthening scheme that was adopted in San Francisco roughly 30 years ago (Paxton et al. 2015). Probable criteria are likely to include minimum pier lengths for the lower floor (which may vary with geographic location), diaphragm material and condition (noting that connection strengthening is mandatory), maximum distances between wall lines, and the presence of solid wall lines parallel to open shopfronts.

Buildings failing to meet the triage criteria would not be eligible for unconditional standard-detail strengthening; however, we suggest that bespoke structural engineering solutions could be devised for elements which do not fall within triage parameters, and that these solutions could sit alongside standard measures for connection strengthening, out-of-plane wall restraint, etc. A comparison with NZS3604 “SED” measures is discussed in Tocher & Cutfield (2021b).

4 CONCLUSIONS

This paper examines the need for in-plane strengthening in the context of a proposed non-specific design approach to seismic retrofit for certain URM buildings, with intent to prevent collapse in a moderate earthquake. In summary, we find that:

- Reconnaissance reports and post-disaster analyses show that in-plane failures occur with lower frequency and lower severity than out-of-plane failures. The majority of reported in-plane failures do not result in damage that threatens the overall stability of the structure. In-plane collapse appears to be rare and to occur mainly in masonry structures larger than those which would be eligible for non-specific design.
- Using conventional analysis approaches, masonry piers which are longer and squatter appear to frequently possess sufficient capacity to withstand design levels of seismic demand. The critical wall lines for common NZ URM buildings are therefore likely to be those which contain significant penetrations and taller, more slender piers.
- Examining the response of open shopfronts containing tall slender piers, and giving consideration to the effect of connection enhancement in activating load-sharing mechanisms, it can be shown that wall lines with low shear strength can nevertheless withstand significant displacement. Comparing the predicted displacements to the spectral demand, it appears that URM buildings of moderate size in many NZ locations may be able to resist moderate seismic demands without requiring specific in-plane structural enhancement to prevent collapse. Some further investigations to quantify the impacts of flange effects on a range of representative example buildings will be necessary.
- A comparison to URM analysis methods from ASCE 41-17 supports the above conclusion.

We propose that geometric parameters can be used to determine whether a building requires specific in-plane intervention as part of a comprehensive standard-detail strengthening scheme. We suggest that many NZ buildings may not require dedicated in-plane strengthening in order to survive moderate seismic demand without excessive risk to life safety.

REFERENCES

Aigwi, I., Phipps, R., Ingham, J. and Filippova, O. (2019). *Urban transformation trajectories of New Zealand's earliest cities undergoing decline – identifying links to the newly enforced Building (Earthquake Prone Buildings) Amendment Act 2016*. Proceedings of the 43rd Australasian Universities Building Education Association (AUBEA) Conference, Noosa, Australia, 6 – 8 November 2019.

ASCE, 2017. *ASCE 41-17: Seismic Assessment and Rehabilitation of Buildings*. American Society of Civil Engineers.

Clifton, C., Bruneau, M., McRae, G., Leon, R., and Fussell, A. (2011). *Steel building damage from the Christchurch Earthquake Series of 2010 and 2011*. Technical report. Retrieved 9 January 2022 from: https://canterbury.royalcommission.govt.nz/documents-by-key/20111104.759

Filippova, O. and Noy, I. (2020). *Earthquake-Strengthening Policy for Commercial Buildings in Small-Town New Zealand*. Disasters 44(1), pp. 179-204.

Nahkies, B. (2014). *Mandatory Seismic Retrofitting – A Case Study of the Land Use Impacts on a Small Provincial Town*. Proceedings of the 20th Annual Pacific Rim Real Estate Society Conference, Christchurch, New Zealand, 19-22 January 2014.

Indirli, M., Kouris, L., Formisano, A., Borg, R., and Mazzolani, F. (2013). *Seismic Damage Assessment of Unreinforced Masonry Structures After The Abruzzo 2009 Earthquake: The Case Study of the Historical Centers of L'Aquila and Castelvecchio Subequo*. International Journal of Architectural Heritage: Conservation, Analysis, and Restoration, 7:5, 536-578

Ingham, J. (2011a). *Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquakes - Part 1*: Presentation by Jason Ingham, University of Auckland, November 2011. Retrieved 6 January 2022 from: https://canterbury.royalcommission.govt.nz/documents-by-key/20111103.716

Ingham, J. (2011b). *Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquakes - Part 2*: Presentation by Jason Ingham, University of Auckland, November 2011. Retrieved 5 January 2022 from: https://canterbury.royalcommission.govt.nz/documents-by-key/20111103.724

Ingham, J and Griffiths, M. (2011a). *The Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquake Swarm*. Technical report. Retrieved 5 January 2022 from: https://canterbury.royalcommission.govt.nz/documents-by-key/20110920.46

Ingham, J and Griffiths, M. (2011b). *The Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquake Swarm*. Addendum Report to the Royal Commission of Inquiry. Retrieved 5 January 2022 from: https://canterbury.royalcommission.govt.nz/documents-by-key/20111026.569

Ismail, N. and Khattak, N. (2015). *Reconnaissance report on the Mw 7.5 Hindu Kush earthquake of 26th October 2015 and the subsequent aftershocks*. United Arab Emirates University. Retrieved 4 January 2022 from: www.uaeu.ac.ae

Lizundia, Bret, et al. (2016). *EERI Earthquake Reconnaissance Team Report: M7.8 Gorkha, Nepal Earthquake on April 25, 2015 and its Aftershocks*. Retrieved 4 January 2022 from: http://www.eeri.org

Los Angeles Municipal Code (1985). *Division 88: Earthquake Hazard Reduction in Existing Buildings*. Retrieved 25 January 2022 from: https://up.codes/viewer/los_angeles/ca-building-code-2016/chapter/new_88/earthquake-hazard-reduction-in-existing-buildings#new_88

Moon, L., Dizhur, D., Senaldi, I., Derakhshan, H., Griffith, M., Magenes, G., and Ingham, J. (2014). *The Demise of the URM Building Stock in Christchurch during the 2010–2011 Canterbury Earthquake Sequence*. Earthquake Spectra 30(1). Retrieved 9 January 2022 from: https://www.researchgate.net/publication/269519605

Moore, T., Kobzeff, J., Diri, J., and Arnold, C. (1988). *The Whitter Narrows, California Earthquake of October 1, 1987—Preliminary Evaluation of the Performance of Strengthened Unreinforced Masonry Buildings.* Earthquake Spectra 4(1).

NZSEE. (2017). *The Seismic Assessment of Existing Buildings*. New Zealand Society for Earthquake Engineering, Wellington, New Zealand.

Paxton, B., Turner, F., Elwood, K., Ingham, J. (2015). *URM Bearing Wall Building Seismic Risk Mitigation on the West Coast of the United States: A Review of Policies and Practices*. Bulletin of the New Zealand Society for Earthquake Engineering 48(1).

Penna, A., Morandi, P., Rota, M., Manzini, C., da Porto, F. and Magenes, G. (2013). *Performance of masonry buildings during the Emilia 2012 earthquake*. Bulletin of Earthquake Engineering 11(4). Retrieved 7 January 2022 from: https://www.researchgate.net/publication/256799917

Russell, A. (2010). *Characterization and seismic assessment of masonry buildings*. PhD Thesis, The University of

Auckland, Auckland, New Zealand.

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).

Tocher, H. and Cutfield, M. (2021a). *A Simpler Approach to Strengthening URM Buildings in New Zealand*. Proceedings of the Structural Engineering Society Conference, Hamilton, New Zealand, 5-6 July 2021.

Tocher, H. and Cutfield, M. (2021b). *Simple Strengthening: An outline of a non-specific design approach for unreinforced masonry buildings in Aotearoa/New Zealand*. Proceedings of the Australasian Engineering Heritage Conference, 10-14 November 2021.

Walsh, K., Dizhur, D., Almesfer, N., et al. (2014). *Geometric characterization and out-of-plane stability of low-rise unreinforced brick masonry buildings in Auckland, New Zealand*. Bulletin of the New Zealand Society for Earthquake Engineering 47(2).