Recent developments in the seismic assessment of masonry buildings

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Abstract

Significant research has been conducted to establish the seismic capacity of unreinforced clay brick masonry buildings in Australia.
In this paper, we consider modern and older existing construction typical of Australia. The paper presents results of laboratory tests on large scale walls, in-situ tests of walls in houses, and analytical predictions to estimate the level of damage that could be expected in a range of earthquake scenarios in Australia to support our conclusion that even a modest M6.0 earthquake within 10km of any capital city in Australia poses a significant life safety hazard to the public and a financial exposure to the nation in the order of $10 Billion.

Keywords: seismic, design, unreinforced masonry, displacement, performance, static pushover analysis

INTRODUCTION

Unreinforced masonry is used throughout Australia for the construction of houses, flats, and low rise commercial premises. Vertical loads in these structures are carried by the unreinforced masonry elements and lateral loads are resisted by the in-plane shear in masonry walls. Many of these buildings were constructed prior to the introduction of seismic design requirements (nationally in 1995) so their seismic capacity is not well known. As the 1989 Newcastle Earthquake showed, unreinforced masonry (URM) is extremely vulnerable to earthquake forces with significant damage observed in URM during the M5.6 earthquake. Since then a small but significant amount of research has been conducted to better understand how Australian URM buildings will respond under earthquake loading and what, if anything, can be done to reduce the hazard that such buildings pose to the public.

Initial research by Klopp (1994) indicated that few existing URM buildings would satisfy the design requirements in the 1993 version of AS 1170.4 on the basis of the default characteristic design strengths for masonry in AS 3700. Lam et al (1995) and Doherty (2000) followed this up with investigations into the displacement capacity of URM walls under out-of-plane inertia loading. Their work helped explain why the seismic performance of URM buildings in Australian earthquakes has been better than predicted (although not adequate) by simple force-based assessments. This led to development of capacity spectrum design methods that consider both strength and displacement capacity of URM buildings which has made its way into the latest version of AS 1170.4 as an 'alternative' method.

In the following sections of this paper, we will report on two recent areas of research to further our understanding of the seismic behaviour of URM buildings – namely, in-situ testing of masonry walls in Adelaide houses scheduled for demolition and improved static pushover analysis methods to more accurately predict the load-deflection capacity curves for URM buildings.

IN-SITU TESTS

In late 2015, the South Australian Department of Planning, Transport and Infrastructure (DPTI) made available to us for testing a number of unreinforced clay brick masonry (URM) houses that the state had purchased and scheduled for demolition in order to create a widened north-south road corridor through the greater Adelaide region. This afforded an ideal opportunity to quantify what material properties and wall strengths were achieved in actual construction so that we might better estimate the seismic capacity of existing URM buildings. Eleven walls and 3 chimneys were tests across 4 different houses that were located in Darlington, a suburb approximately 15 km south of the Adelaide GPO. Importantly, the buildings were all constructed (1960 – 1980) before earthquake force design requirements were in place.

(a) 1960s URM, 3 walls tested

(b) 1960s URM, 5 walls & 1 chimney tested

(c) 1960s URM, 2 chimneys tested.

(d) 1980s URM, 3 walls tested.

Figure 1. Darlington, SA URM houses

Masonry samples were taken from each building/chimney to establish the actual in-situ engineering properties for comparison to the characteristic design values given in the masonry structures code, AS 3700 (). We conducted tests to determine flexural tensile bond strength, compressive and shear strength, and elastic modulus of the masonry. Due to space limitations, only the values from the bond wrench tests (Table 1) and compression tests (Table 2) are reported here (Table 1). See () full details. What can be seen that (i) there is a large variation in values between houses as well as within as evidenced by the CoV values and (ii) the average 'characteristic' bond strength without assistance from the plaster is five times smaller than the default value given in AS 3700.

Table 1. Bond strength test results

Bldg	Location	No.	Mean Bond Str. f_{mt}, MPa	CoV	Char. Bond Str. f'_{mt}, MPa
A	External cavity wall	8	0.15	0.30	0.04
	Plaster in tension	4	0.54	0.36	0.18
B	Outbuilding, no plaster	10	0.15	0.40	0.04
	No plaster in tension	16	0.15	0.73	0.02
	Plaster in tension	2	0.59	0.30	0.26
C	Outer cavity leaf, no plaster	10	0.26	0.44	0.09
D	Outer cavity leaf, no plaster	10	0.10	0.43	0.02
All	Plaster in tension	6	0.56	0.27	0.21
	No plaster in tension	54	0.16	0.61	0.04

AS 3700 (2011): *"… Under actions resulting from wind, earthquake loads or similar forces of a short-term, transient nature, … For clay, concrete and calcium silicate masonry (except special masonry), a (f'_{mt}) value* ***not greater*** *than* ***0.20 MPa****."*

Table 2. Compression test results

Bldg	No.	Compressive strength, f_{mc}, Mpa Mean (CoV) Characteristic	Young modulus, GPa Mean (CoV)		
			Unit, E_u	Mortar, E_j	Masonry, E_m
A	6	4.5 (0.33) 1.7	4.9 (0.75)	1.1 (0.98)	2.5 (0.64)
B	12	5.0 (0.23) 2.6	4.7 (0.90)	0.47 (0.25)	1.9 (0.55)
D	3	15.8 (0.10) 11.2	27.6 (0.10)	1.15 (0.09)	11.6 (0.09)
All	21	6.5 (0.64) 1.8	9.1 (1.17)	1.2 (0.91)	2.99 (0.80)

Given the bond wrench test results, it was expected that the walls would fail at relatively low air bag pressures. The test setup for each wall test is typical of those shown in Figure 2. In short, a reaction frame was created for each wall test and either braced back to the floor or adjacent walls after which an airbag was positioned between the test wall and backing frame. Wall displacements were recorded as the airbag pressure was increased. Each test stopped once the wall's maximum strength had been reached. The wall dimensions and test results are noted in Table 3 with the load-deflection response for each of the walls shown in Figure 3. What is surprising is that the actual wall strengths were greater than the code demand for a single storey building with Z = 0.14 in spite of the fact that the material strengths were less than code assumptions. More promising though is that the predicted wall strengths using the average material properties from Table 1 (not the characteristic, design values) which include the effects of the plaster, agree reasonably well, being slight under predictions of the actual strength for most of the walls.

(a) External test setup

(b) Internal wall test setup, adjacent to window opening

(c) Internal wall test setup w/o openings

(d) Steel support for reaction frame dynabolted to side wall

Figure 2 – typical wall test setups.

Table 3. Wall bending test results

Wall	Length	Height	Thickness(1)		L/h	h/t		Supported Edges(2)		Applied Overburden	
	L(mm)	h (mm)	t (mm)							(kPa)	
			N(3)	F(4)		N	F	N	F	N	F
A-1	2430	2750	76	110	0.9	32.0	22.1	3	4	0	9.5
A-2	4075	2750	76	110	1.5	53.6	37.0	3	4	0	9.5
A-3	3020	2750	76	N/A(5)	1.1	39.7	---	3	---	4.1	N/A
B-1	4050	2530	110	N/A(5)	1.6	36.8	---	3	---	6.8	N/A
B-2	3795	2720	110	76	1.4	34.5	49.9	4	2	13.6	0
B-3	3260	2720	76	N/A(5)	1.2	42.9	---	3	---	4.1	N/A
B-4	3030	2720	76	110	1.1	39.9	27.5	3	4	0	13.6
B-5	3040	2720	76	N/A(5)	1.1	40.0	---	4	---	4.1	N/A
D-1	3980	2415	110	110	1.6	36.2	36.2	3	4	38	38
D-2	1990	2520	110	110	0.8	18.1	18.1	3	3	20	20
D-3	1790	2400	110	110	0.8	21.8	21.8	2	2	38	38

(1) excluding plaster (2) see also description and relevant figures for support details (3) Near (airbag-loaded) leaf (4) Far leaf (not in contact with airbag) (5) not a cavity wall

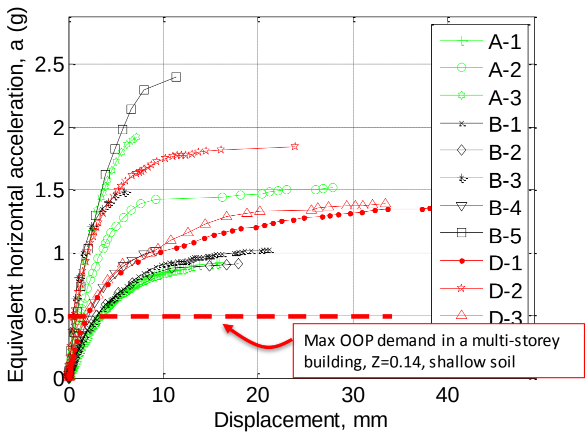


Figure 3 – load-deflection test results

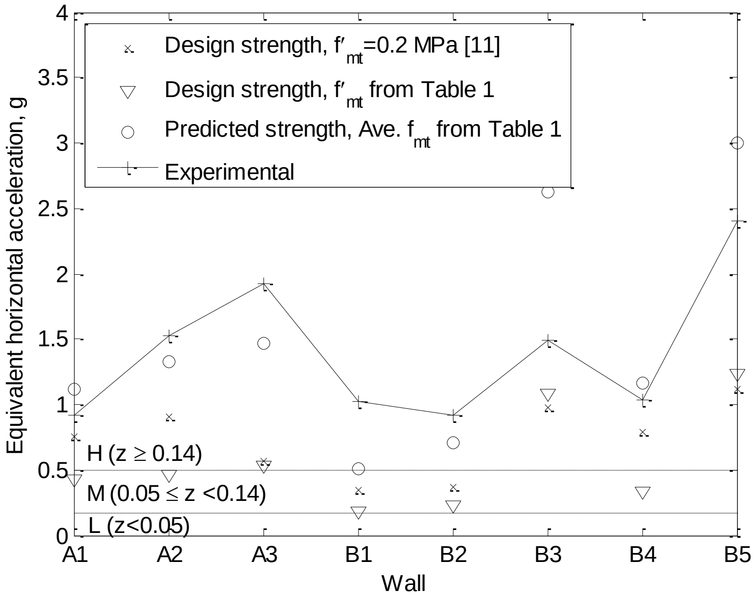
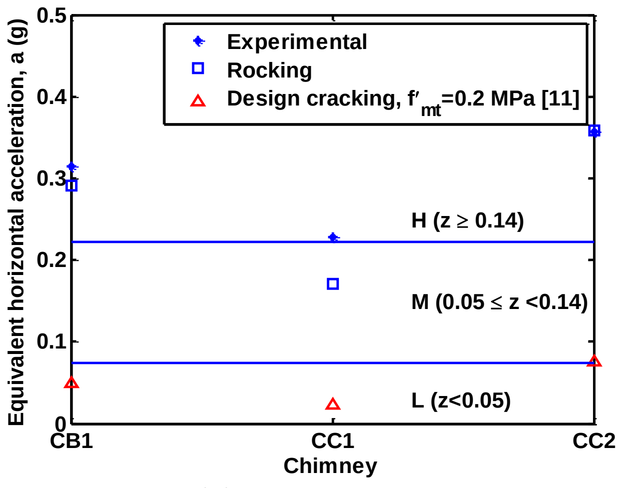


Figure 4 – Experiment vs theory comparisons.

Three chimneys were also tested to failure by attaching a collar around the top of the chimney to which a cable with a load cell was connected. The cable was pulled horizontally by the operator of an excavator as shown in Figure 5. The tests showed that the chimneys cracked at loads well below the design accelerations (< 0.05g) but their rocking strength agreed extremely well with the experimental load that actually pulled them off.

(a) Chimney test method



(b) Chimney test results

Figure 5. Chimney testing and results

IMPROVED STATIC PUSHOVER ANALYSIS

Static pushover methods for analysis of seismically loaded masonry buildings with rigid (i.e. concrete) floor slabs is being promoted for use with URM buildings. However, its accuracy for buildings with flexible floor diaphragms is still questioned. This paper presents a brief summary of the PhD research by Nakamura (2016) to develop static pushover analysis techniques that can cater for flexible diaphragms. The software Tremuri (Lagomarsino et al 2013) was developed specifically for URM buildings by representing each masonry wall as an 'equivalent frame' (Figure 6) where each masonry pier and spandrel can simulate shear, flexural and rocking response and are connected with rigid end offsets. However, in its static pushover analysis, it does not account for diaphragm mass and the corresponding inertial flexibility. In this research, dummy frames were inserted into the building model so that mass and corresponding displacement degrees of freedom were generated in the model to simulate diaphragm flexibility (Figure 7).

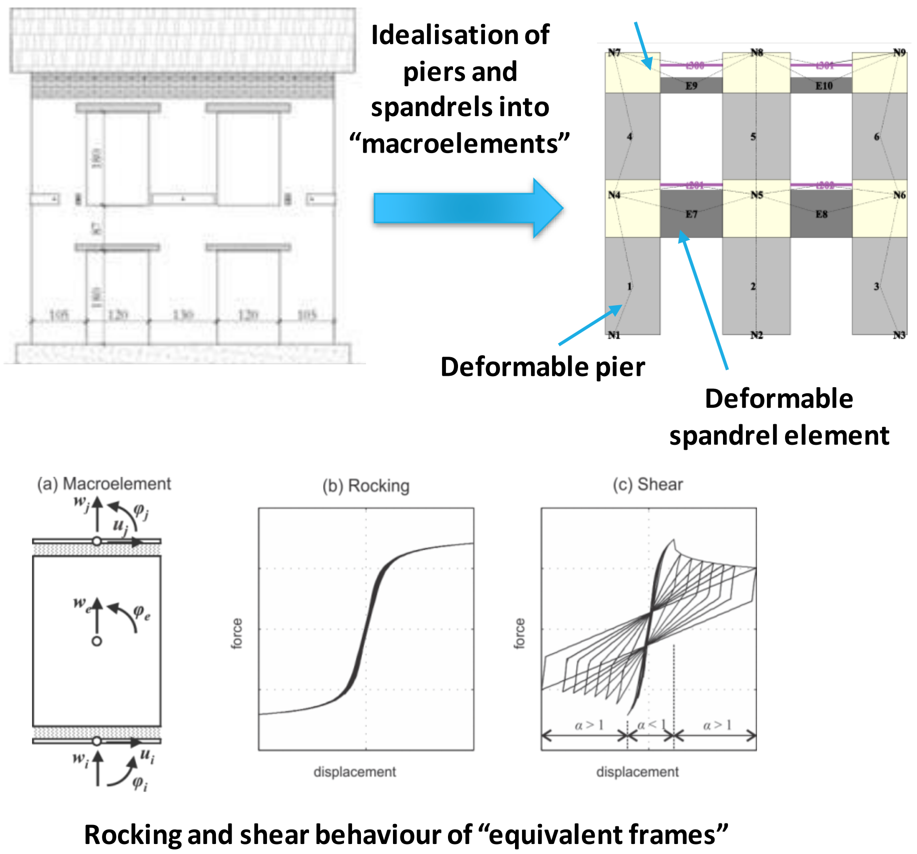


Figure 6. Equivalent frame model

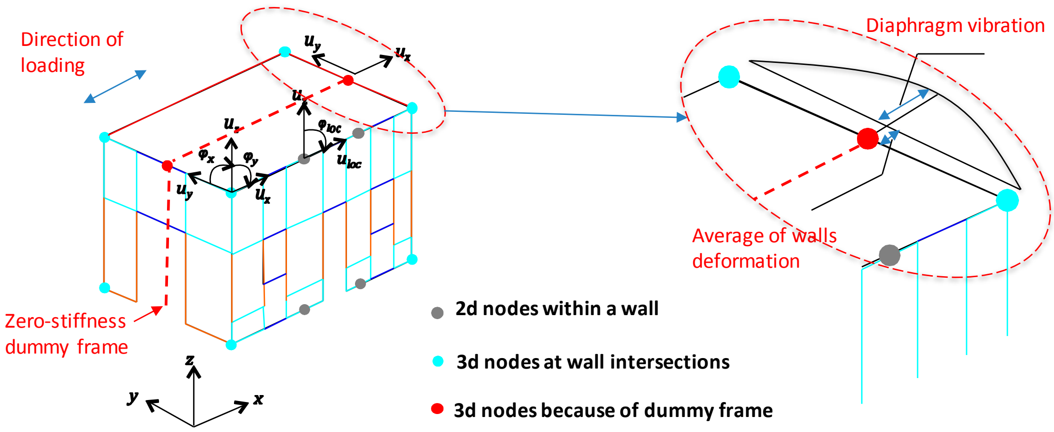
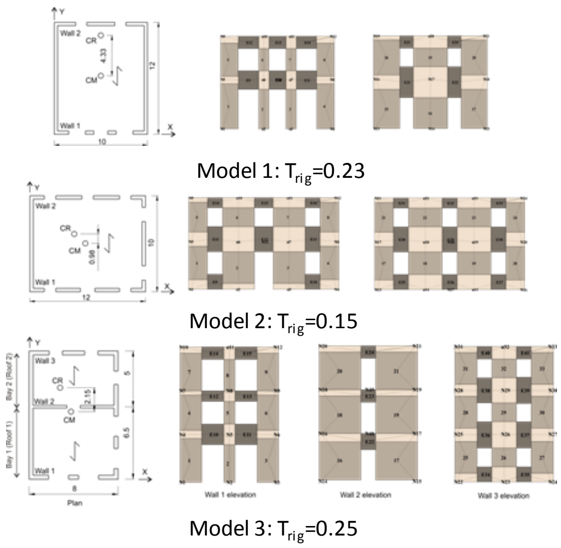


Figure 7. Dummy frame element for URM building

Three URM buildings were considered in this study (Figure 8) and in each building a range of diaphragm stiffnesses were simulated; from very stiff to very flexible. The results of this investigation are presented below.



Range of diaphragm stiffness and indicative periods

Diaph.	Gd (kN/m)	Description	Reference period, sec	Ratio T_d/T_{rig}
D1	350	Single straight sheathing	0.48~0.85	2~3.5
D2	1400	Single diagonal sheathing, chorded	0.24~0.42	1.7~2
D3	2625	Double straight sheathing, chorded	0.18~0.3	1.2~0.8
D4	3150	Double layered panels, chorded	0.15~0.26	1~0.6
D5	7000	Hypothetical retrofit 1	0.11~0.19	0.8~0.5
D6	35000	Hypothetical retrofit 2	0.03~0.05	0.2~0.1

Figure 8. Building models studied

The earthquake response of the three buildings were investigated using NTHA and three different pushover methods – N2 (Fajfar, P. and Gašperšič, P. 1996), Modal Pushover (MPA) and Adaptive pushover. 12 earthquake records from the PEER website, conforming to Australian seismicity were selected and 3 different levels of shaking were assumed, sufficient to introduce both elastic and inelastic response. This resulted in a total of 12 (records) x 3 (shaking level) = 36 nonlinear time-history analyses (NTHA) for each of the 3 building models.

The N2 Pushover calculations used 4 different hysteresis rules (Takeda, Origin-centre, Bilinear elastic, EPP), 3 different lateral load patterns (uniform, linear, SRSS) and a Control node placed at diaphragm mid-span, top of flexible wall, and top of rigid wall. The Modal Pushover considered different combination rules, e.g. SRSS, CQC were investigated. While the Adaptive pushover analyses considered three approaches to representing the progressive deformations of the building in the dynamic properties of the SDOF analyses (pushover forces obtained based on instantaneous deformation pattern by Galasco et al. 2006 and a virtual work approach from Hernández-Montes et al. 2004)

It can be seen in Figure 9 that the Takeda rule gives the most accuate comparsions to NTHA across the full range of displacement magnitudes.

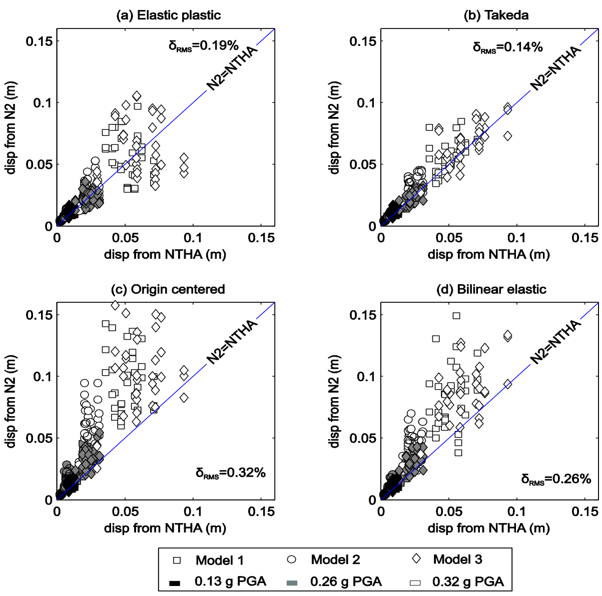
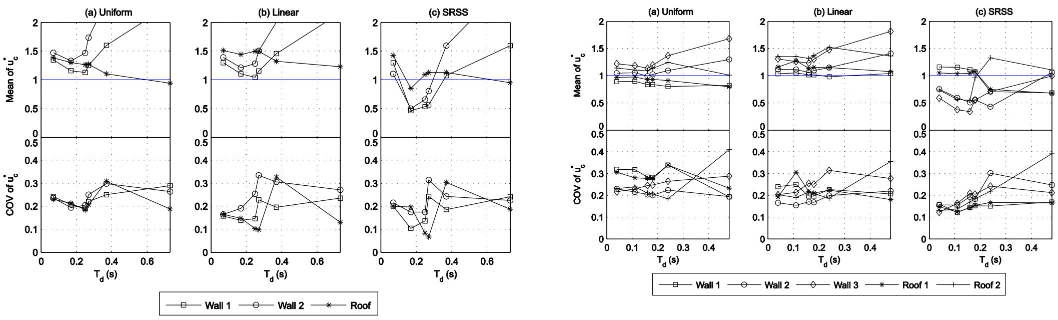


Figure 9. Hystersis rule influence.

It can be seen in Figure 10 that for the N2 method the most consistent results were obtained with the control node at the roof level in the diaphagm midspan. It was also seen that the uniform and linear varying load patterns gave displacements that consistently envoloped the NTHA results with the observation that the mean of the uniform and linear results was generally conservative for all diaphragm flexiblities. In constrast, the adaptive push over method was seen to be unconservative for flexible diaphagms (Figure 11).



(a) Building Model 2 (b) Building model 3

Figure 10. N2 Pushover results

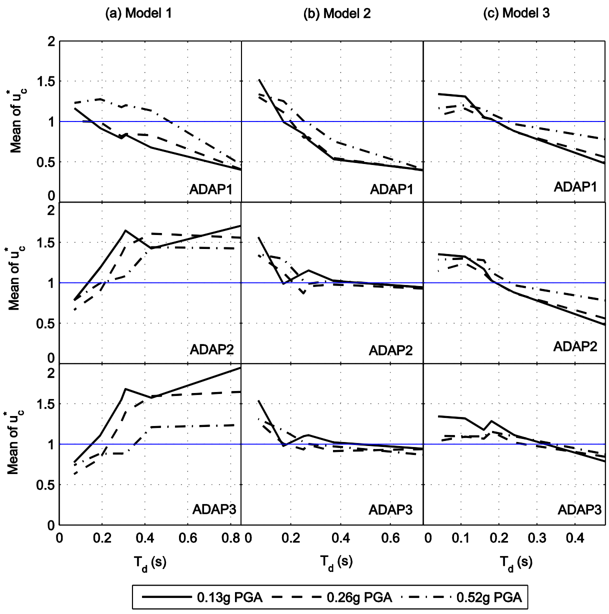


Figure 11. Adaptive PO results

In summary, the following recommendations have come out of this study:

(a) Modified Takeda should be used as hysteresis rule in N2;
(b) The mid-span of the most flexible diaphragm at roof level should be taken as the control node;
(c) The accuracy of the MPA was primarily dependent on the failure mechanism; unsuitable for shear-dominated response;
(d) Accuracy of N2 and the adaptive method were influenced mainly by the level of diaphragm flexibility; other parameters had secondary importance;
(e) In the adaptive method, the conversion to the equivalent SDOF system to be based on the equal work done on the MDOF structure by the pushover forces using the procedure of Hernández-Montes et al. [2006];
(f) The adaptive method is most accurate in predicting roof displacements and inter-storey drifts ratios for relatively stiff diaphragm; for more flexible diaphragms the method becomes non-conservative and N2 becomes more suitable; and
(g) For the buildings with overly flexible diaphragm, conservative results can be obtained by using the N2 method, by taking the envelope of the results obtained using the pushover force distributions corresponding to the uniform and linear displacement shapes

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