

SEISMIC DESIGN OF MASONRY BUILDINGS IN AUSTRALIA

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SUMMARY

This paper presents an overview of the design and performance of Australian masonry structures under earthquake loading. In particular, the current design requirements are considered and whether they are adequate or not. The question of the seismic resistance of existing structures and the need for future research is also touched upon.

INTRODUCTION

Contrary to popular opinion, Australia has a long history of earthquakes that have caused significant damage to buildings. For example, South Australia has experienced several earthquakes of moderate magnitude including the 1897 earthquake (estimated Richter magnitude M 6.5) near Beachport which caused some out-of-plane bending failures to masonry buildings (Figure 1). The 1954 earthquake in Adelaide (M 5.6) is probably the largest earthquake to have its epicentre within 15 km of the central business district of a capital city that has occurred in Australia. At that time, Adelaide had a population of about 480,000 and the damage bill was estimated to be, in today's dollars, over \$1 billion, most of it to do with unreinforced masonry construction. As shown in Figures 2 and 3, out-of-plane bending wall failure was common as were chimney failures. Had that earthquake affected present day Adelaide, with a much higher population density, the damage bill would be significantly greater. A magnitude M 5.6 earthquake in December 1989 caused similar damage in Newcastle (Figure 4) and notably the first deaths (12) due to earthquake-induced structural collapse in Australia. It should be noted that much of the Newcastle earthquake damage occurred in badly deteriorated buildings and/or buildings of poor construction (eg, corroded or missing wall ties) on soft soil sites. Nevertheless, the Newcastle earthquake magnitude (M5.6) was not large and buildings in other regions of a similar vintage would be expected to display similar characteristics and damage patterns.



Figure 1. Beachport, South Australia post office after 1897 (M 6.5) earthquake.
(Photo courtesy of David Love, PIRSA)



Figure 2. House in Adelaide, South Australia after 1954 (M5.6) earthquake.
(Photo courtesy of David Love, PIRSA)



Figure 3. Damage to Adelaide, South Australia buildings in 1954 (M5.6) earthquake.
(Photo courtesy of David Love, PIRSA)



Figure 4. Newcastle, New South Wales buildings after 1989 (M5.6) earthquake.

As suggested by the photos presented in Figures 1 - 4, the vast majority of earthquake induced damage in Australia has occurred in unreinforced masonry construction. There have been even larger earthquakes in Australia during this time but by virtue of the country's sparsely distributed population, none have occurred near a major city. Where unreinforced masonry construction has been subjected to large earthquakes such as the M6.9 earthquake in Meckering, West Australia, the damage to masonry buildings has been virtually complete (Figure 5). The immediate response in Meckering to this earthquake was that no unreinforced masonry construction could be used unless it was fully designed by an engineer; now recognised as being an over-reaction.

Nevertheless, it appears that it is just a matter of time before an earthquake of Richter magnitude 6 or greater occurs in one of Australia's capital cities. How structural engineers deal with such 'low probability but high consequence' natural hazards is still a matter of much debate. Suffice to say that while the earthquake hazard in Australia is modest by international standards, the low resistance of unreinforced masonry construction to severe earthquake induced shaking makes the seismic risk significant, especially in large Australian cities where masonry construction is prevalent. It is of interest, and perhaps a bit surprising, that a *Catastrophic Disasters Review* conducted recently by Emergency Management Australia (Pearce 2006) estimated that the highest risk to Australia posed by any hazard, natural or man-made, is that due to earthquake. Indeed, the risk to the insurance industry of a moderate-to-large earthquake in the Sydney region is such that this risk is amongst the highest globally for the world insurance market owing simply to the low seismic resistance for Sydney's large built environment, much of which is unreinforced masonry construction such as low-rise apartment buildings and small commercial premises.



Figure 5. Damage to hotel in Meckering, West Australia in 1968 (M 6.9) earthquake.
(Photo by Ian Everingham, courtesy of Kevin McCue)

EARTHQUAKE DESIGN REQUIREMENTS

The first Australian earthquake code, AS 2121, was published by Standards Australia in 1979 (Standards Australia 1979), largely as a consequence of the M6.9 Meckering earthquake in 1968. The underlying design philosophy of this and all subsequent earthquake loading codes in Australia was to control damage in a 500 year return period earthquake which corresponds to a 10 % probability of exceedence within 50 years. By limiting the drift limits to 1.5% in this so-called design magnitude earthquake it is expected that no buildings will collapse and so achieve the actual design objective of providing life safety in the design earthquake.

Adelaide city council, in 1983, was the first capital city to require all new construction to be designed for earthquake induced loads in accordance with AS 2121. By all accounts, it seems that no other capital cities required earthquake design for buildings in their jurisdictions until after the introduction of the Australian Earthquake Loading Code, AS 1170.4. This was first published in 1993 (Standards Australia 1993) and was first 'called up' by the Building Code of Australia in 1995. Since that time, all new buildings in Australia have been required to consider earthquake loading in their design although in practice many structures, by virtue of their form of construction, material, or occupancy, are deemed-to-satisfy the earthquake requirements and so are exempted from formal seismic design. For unreinforced masonry construction, the 1995 call-up of AS 1170.4 has had the most profound effect in that under much closer scrutiny by practising engineers typical construction details that have long been used for this form of construction have been found wanting with respect to earthquake loads. Put another way, engineers were finding it impossible to justify through calculations that typical construction details that have been widely used prior to 1995 have the necessary capacity to transmit the forces induced by the design magnitude earthquake. For example, the use of damp proof course joints as a barrier to moisture at the base of walls and slip joints to prevent damage due to differential creep and shrinkage at the interface between clay brick masonry walls and concrete slabs severely restricts the seismic capacity of these elements to transmit the storey shear forces expected from the design magnitude earthquake. An amendment was issued to the 1998 edition of the Masonry Structures Code, AS 3700 that allows engineers to use friction at these joints for the transmission of seismic forces, based on research at the University's of Newcastle and Adelaide (Griffith and Page, 1998). An update of AS 1170.4 was recently published (Standards Australia 2007) in which further restrictions in the form of height limits have been placed on unreinforced masonry construction. The immediate implications of this new code for design of typical masonry buildings in Australia is that unreinforced masonry construction is restricted to buildings less than 15m in height.

VULNERABILITY STUDY

In parallel with the revision of the Australian earthquake loading code, AS 1170.4 (2007), the Clay Brick and Paver Institute (CBPI) of Australia commissioned a study to determine the likely range of design parameters for which typical unreinforced masonry construction is likely to satisfy the earthquake loading requirements of AS 1170.4 without any special/extra detailing for seismic forces. The form of construction considered was regarded as being broadly representative of modern multi-storey construction for commercial office buildings and domestic apartment buildings. The basic construction considered was clay brick cavity wall construction with concrete floor slabs. Damp proof course joints and slip joints were

assumed to be present at the bottom and top of walls (between brickwork and concrete slabs), respectively. Buildings of 2, 3, 4 and 5-stories (3m storey heights) were considered on sites with firm, moderate and soft soil conditions over a range of peak ground acceleration (PGA) coefficients between 0.05 g – 0.12 g corresponding to the standard design magnitude (500 year return period) earthquake covering most of the major cities in Australia.

The first stage of this study was reported (Willis et al, 2006) to the sponsor and summarised in the literature (Lawrence et al, 2007). The main findings, highlighted here in Tables 1 and 2, were that for the majority of combinations the walls/buildings appear to satisfy the strength requirements corresponding to earthquake induced forces. Where the strength limit state corresponding to out-of-plane bending strength design criteria was exceeded (denoted with “1”s in Tables 1 – 2) it was primarily for walls $\geq 8\text{m}$ in length for sites with $\text{PGA} \geq 0.08\text{g}$. It was also noted that the seismic demand for walls in out-of-plane bending increased with building height, supporting the concept of a height limit in AS 1170.4 for unreinforced masonry construction. For shorter lengths, higher acceleration coefficients or very soft soils were needed to exceed the design criteria. Perhaps the more surprising result was that in-plane shear failure due to sliding at the base of a wall (denoted by “3”s and “4”s in Tables 1 and 2) was detected more frequently than out-of-plane bending failure although again only in a minority of cases and only where the PGA design value was 0.10g or greater. Even so, there is strong evidence that ‘sliding’ does not necessarily compromise the life-safety for a building.

Table 1: Parametric study - office building (from Willis et al, 2006)

Length of Wall, L		4 m				6 m				8 m				10 m			
Site Sub-Soil Class	Hazard Factor, Z	No. of Levels				No. of Levels				No. of Levels				No. of Levels			
		2	3	4	5	2	3	4	5	2	3	4	5	2	3	4	5
A	0.05																
	0.08																
	0.10																
	0.12																
B	0.05																
	0.08																
	0.10														1		
	0.12	4				4				4				1,4	1	1	1
C	0.05																
	0.08														1	1	
	0.10	4	4			4	4			4	1,4	1		1,4	1,4	1	1
	0.12	4	4	4		4	4	4		1,4	1,4	1,4	1	1,4	1,4	1,4	1
D	0.05																
	0.08													1		1	1
	0.10	4	4	4	4	4	4	4	4	4	1,4	1,4	1,4	1,4	1,4	1,4	1,4
	0.12	4	4	2,4	2,4	4	4	1,2,4	1,2,4	1,4	1,4	1,2,4	1,2,4	1,4	1,4	1,2,4	1,2,4

Note: 1 indicates out-of-plane bending failure; 2 indicates shear failure in out-of-plane direction; 3 and 4 indicate in-plane shear failure in walls in short and long directions of building, respectively.

Table 2: Parametric study – home unit (from Willis et al, 2006)

Length of Wall, L		4 m				6 m				8 m				10 m			
Site Sub-Soil Class	Hazard Factor, Z	No. of Levels				No. of Levels				No. of Levels				No. of Levels			
		2	3	4	5	2	3	4	5	2	3	4	5	2	3	4	5
A	0.05																
	0.08																
	0.10																
	0.12																
B	0.05																
	0.08																
	0.10														1		
	0.12													1	1	1	1
C	0.05																
	0.08														1	1	
	0.10										1	1		1	1	1	1
	0.12	3,4	3,4			3,4	3,4			1,3,4	1,3,4	1	1	1,3,4	1,3,4	1	1
D	0.05																
	0.08														1	1	1
	0.10										1	1	1	1	1	1	1
	0.12	3,4	3,4	2,3,4	2,3,4	3,4	3,4	All	All	1,3,4	1,3,4	All	All	1,3,4	1,3,4	All	All

Hence, the CBPI funded a follow-up study to assess the seismic capacity of the same cohort of unreinforced masonry buildings using a displacement-based approach (eg, Griffith et al, 2003; Griffith et al, 2006). In this study, the in-plane sliding displacements and the out-of-plane wall deformations corresponding to the in-plane and out-of-plane ‘failures’ identified in Tables 1 and 2 will be estimated and compared to their corresponding displacement capacities. The motivation behind this approach is the recognition that a masonry wall’s load carrying capacity is not necessarily lost once a masonry wall’s strength limit state is reached. Indeed, collapse prevention for the life-safety design objective can be met as long as a building’s walls do not deflect beyond their capacity which could be substantially greater than the deformations at which their strength limit state is reached. The method being used here is similar to the displacement-based methodology described in Doherty et al (2002) and that which has recently been adopted by the Italian Seismic Code (OPCM 2005) for assessing the seismic capacity of masonry walls in existing buildings.

SEISMIC STRENGTHENING RESEARCH

In closely related work to the industry funded research reported above, collaborative research between the Universities of Adelaide and Newcastle has been investigating the use of FRP strips to strengthen unreinforced masonry walls in bending and in-plane shear. The initial work has focussed on establishing the FRP-brick masonry bond characteristics so that adequate mechanics-based models can be developed to enable effective design techniques for externally-bonded and near-surface-mounted applications of FRP strip reinforcement. Preliminary results have been encouraging, resulting in accurate models for predicting the load at which intermediate-crack debonding occurs in flexural walls (Petersen et al 2007; Willis et al 2007). Further work is now underway between researchers at these two universities and colleagues at the University of Auckland in New Zealand to account for the effect of cyclic loading on the FRP-masonry bond behaviour. The current work will also culminate in dynamic tests of actual buildings slated for demolition. These buildings will first be strengthened with various FRP reinforcing strategies in order to validate the design methodologies being developed by the research team.

CONCLUDING REMARKS

Recent studies have indicated that unreinforced masonry construction which is well designed and constructed (to the minimum modern building code requirements) should survive the so-called design magnitude earthquake (i.e, the 500-year return period earthquake) without collapse. As long as soft ground storeys and other geometric irregularities are avoided, and the walls are well connected to the floor slabs, then most buildings should perform satisfactorily in the design magnitude earthquake. The damage to older, badly deteriorated or poorly constructed unreinforced masonry buildings would, not surprisingly, be much more severe. Current research is working to develop some practical strengthening techniques as well as improving our ability to accurately assess the seismic resistance against collapse for unreinforced masonry buildings but much work remains to be done.

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