Seventh Canadian Masonry Symposium
McMaster University
Hamilton, Ontario
June 4-7, 1995

SEISMIC DESIGN OF UNREINFORCED MASONRY – A REVIEW OF THE AUSTRALIAN REQUIREMENTS

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ABSTRACT

Australia is not in a recognised earthquake zone, but does have a history of small to medium sized intraplate earthquakes. Unreinforced masonry is a commonly used form of construction, so that there is potential for damage to masonry structures from these events. A new earthquake loading standard has recently been issued, and it contains mandatory requirements for the seismic design of structures in all areas. For ductile structures the provisions are usually quite nominal, but there are significant implications for unreinforced masonry in some areas. This paper reviews the requirements of the new Australian earthquake loading code with regard to unreinforced masonry. The requirements range from deemed–to–comply detailing in areas of low seismic risk, to full calculation in high risk areas. Significant height limitations have also been imposed in some cases. Despite these restrictions, well constructed and detailed unreinforced masonry is still an economical and attractive solution in many instances.

INTRODUCTION

Australia is not in a recognised earthquake zone as it is located towards the centre of a continental plate. Intraplate earthquakes that do occur are the result of relatively random movements at local faults in the earth's crust. Australia does have a history of small to medium sized earthquakes, but until recently, none of these events had caused major damage or loss of life as they occurred away from major population centres. The 1989 Newcastle earthquake changed this perception. This moderate earthquake, (estimated to be magnitude 5.6 on the Richter Scale), caused thirteen deaths, numerous injuries, and damage estimated at between 1.5 and 2 billion dollars. Much of this damage was to structural and non–structural unreinforced masonry, thus highlighting the need for more attention to the seismic design of all structures, and particularly those containing masonry.

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A new Australian earthquake loading standard has recently been issued and is about to be included in the building regulations. This standard has mandatory requirements for the seismic design of all structures. For ductile structures the provisions are quite nominal, but ensure that seismic effects have at least been considered. However there are more significant implications for unreinforced masonry as it behaves in a brittle manner. Since this is a widely used material as both a structural medium and as infill or cladding, some design and detailing modifications are necessary in some cases to satisfy the requirements of the new standard. This paper reviews the requirements of the Australian earthquake loading code and discusses the implications of the new provisions for unreinforced masonry. The requirements range from deemed–to–comply detailing requirements in areas considered to be of low seismic risk, to full calculation and design in high risk areas. Significant height limitations have also been imposed in these cases. Despite these restrictions, unreinforced masonry construction can still be used in most areas provided appropriate design and detailing is carried out, and that standards of workmanship are adequate.

AUSTRALIAN SEISMICITY

As mentioned above, Australia is located on a tectonic plate and is therefore subjected to intraplate rather than interplate earthquakes. As a result, earthquakes in Australia are generally smaller in magnitude and less frequent than those which occur in interplate regions. They are also more random in nature. Earthquakes of magnitude 5 are experienced on average every two years, and larger earthquakes have been recorded (Hutchinson *et al.*, 1994). An earthquake of magnitude 6.8 was experienced at Meckering in Western Australia in 1968, and magnitude 6.2, 6.5 and 6.7 earthquakes were recorded at Tennant Creek in the Northern Territory in 1988 (Johnston and Kanter, 1990). Fortunately, damage from these and other similar events was limited due to the remoteness of their locations. However, the Newcastle earthquake of 1989 demonstrated that even moderate earthquakes of smaller magnitude have the potential to cause major damage and loss of life. In this case other factors such as soft soils, poor design and detailing of structures, building deterioration, and lack of consideration of earthquake effects in design also played a major role.

AUSTRALIAN EARTHQUAKE LOADING STANDARD

Because of the relative lack of seismic activity, the common perception in Australia has been that the risk from earthquakes was low and seismic forces need not be considered in design. An earthquake loading standard was produced in 1979 (Standards Australia, 1979) but in most cases was not incorporated in the building regulations and rarely used for design in most areas. As the result of a general review (accelerated by the Newcastle earthquake), a new standard AS1170.4 was issued in 1993 (Standards Australia, 1993). This standard is shortly to be included in the Building Code of Australia, and will then become part of the mandatory building regulations. It therefore has the potential to have a major impact on building design and practice, particularly for masonry construction. The code provisions are generally consistent with the recommendations made by the Applied Technology Council (Applied Technology Council, 1988) and significantly different to the earlier Australian code.

Philosophy of the AS1170.4 Provisions

To be consistent with the various material codes, loads are expressed at the ultimate limit state corresponding to an earthquake event with a return period in the order of 500 years

(Hutchinson, *et al.*, 1994). As for other earthquake codes, the main aim of the provisions are to minimise the risk of loss of life, to improve the general structural performance and the capability of essential structures to survive, and to minimise the risk of damage to hazardous facilities. A static or dynamic analysis can be performed, depending on the design category, the structural configuration, and the building ductility. The requirements apply to all buildings and their components, including domestic structures.

Design Requirements
The design loads and detailing requirements depend on a number of factors:

1. Geographical Location. The design acceleration coefficient (a) varies with location. The Standard contains contours of acceleration coefficients for the whole of Australia which represent values judged to have a 10% chance of exceedance in 50 years (Standards Australia, 1993). The values range from less than 0.05 to a maximum of 0.22 (see Figure 1). One of the important changes from the previous code is that all parts of Australia are deemed to have some risk, so that the possibility of earthquake loading must always be considered, even though in many cases seismic effects will not govern over wind load.

2. Site Factor (S). It has been clearly established that site conditions can have a

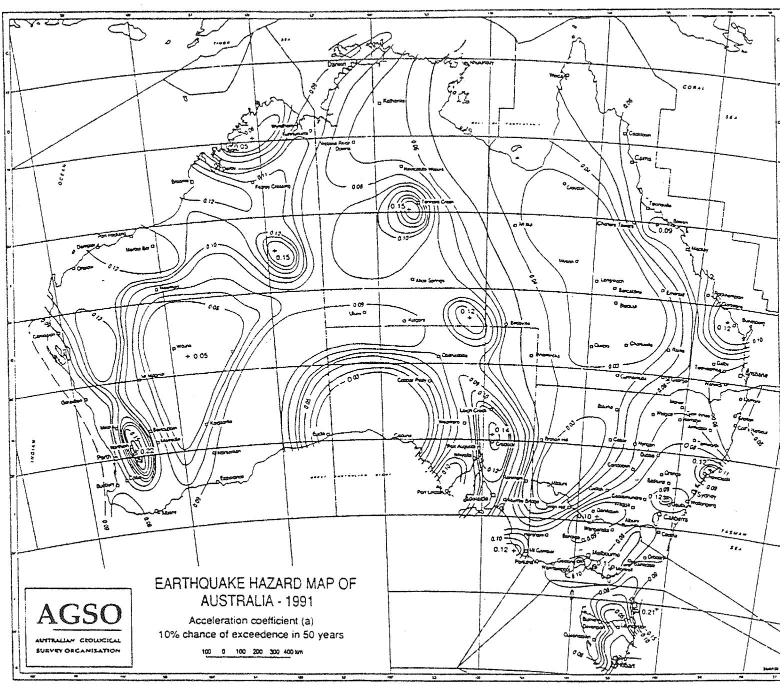

Figure 1. Acceleration Coefficient Map of Australia
(Standards Australia, 1993)

significant local influence on the magnitude of the ground motion, so that a site factor (S) is used to modify the value of the acceleration coefficient. The site factor ranges from 0.67 for good quality rock, to 2.0 for soft clays, loose sands or silts and uncontrolled fill if their depth exceeds 12 metres.

3. Importance Factor (I). This factor increases the value of the base shear force to be transmitted in recognition of the need for special consideration to be given to structures vital to public needs immediately following an earthquake. The value for I in this case is taken as at least 1.25. Normal structures have an importance factor of 1.

4. Structure Classification. Structures are classified into four groups: (i) domestic structures; (ii) Type III structures comprising buildings essential to post–earthquake recovery or associated with hazardous facilities; (iii) Type II structures comprising buildings that are designed to contain a large number of people, or people of restricted or impaired mobility; and (iv) Type I structures comprising buildings not in the above categories.

5. Earthquake Design Category. The earthquake design category controls the type of analysis (dynamic or equivalent static) as well as the design and detailing requirements. It is concerned mainly with life safety and the degree of exposure of the public to earthquake risk. The requirements are based on the provisions of the previous code and on technical work from the United States which was modified to suit Australian conditions (Standards Australia, 1993). A summary of the requirements is given in Table 1.

6. Other Considerations. As is usual for seismic design, the Standard requires account to be taken of other factors which will influence the building performance. A suitable structural system must be selected (bearing wall, moment resisting frame, etc.) and that system must be designed and detailed to ensure that it will behave in the way intended. Factors such as configuration in plan and elevation, torsional and potential soft storey effects, development of effective diaphragm action, deflection and drift limits, etc., must be considered. The provisions apply to both structural and non–structural components.

The discussion which follows will concentrate on the requirements for unreinforced masonry structures.

DESIGN AND DETAILING REQUIREMENTS FOR MASONRY STRUCTURES

A large proportion of masonry constructed in Australia is unreinforced. Reinforced masonry is mainly used for structures in cyclonic regions (in the northern parts of Australia), and for special applications such as retaining walls, large factory walls, etc. The bulk of unreinforced masonry is used in domestic construction (both brick veneer and cavity wall construction) and in light commercial buildings, particularly loadbearing apartment buildings. These latter buildings are typically three or four storeys tall, but some structures in excess of ten storeys have been constructed in the past. As mentioned previously, in most cases earthquake effects have not been included in design, as this has not been required by the building regulations in most states. The advent of the new Standard therefore has significant implications in some cases. These requirements are discussed in the ensuing sections.

Table 1. Earthquake Design Categories

	Structure Classification				
	General Structures			Domestic	
	Type I	Type II	Type III		
aS ≥ 0.2	E	D	C	H1 (B equivalent)	Increased ground movement
0.1 ≤ aS < 0.2	D	C	B	H2 (A equivalent)	
aS < 0.1	C	B	A	H3 (A equivalent)	
Increased need for survival of the structure					

Table 2. Summary of Design Requirements for Domestic Structures

Earthquake Design Category	Ductile	Non–Ductile
H1	No design or detailing	No design or detailing
H2	No design or detailing	Detailing required*
H3	Detailing required*	Static analysis and detailing*

* Detailing requirements
- All parts of the structures to be tied together in the horizontal and vertical planes
- Beams and truss connections (5% of gravity load reaction – H2)
- Beams and truss connections (7.5% of gravity load reaction – H3)
- Wall anchorage to transmit 10(aS) kN/m of wall

Domestic Construction
As indicated in Table 1, there are three earthquake design categories for domestic construction (H1 to H3) with the requirements becoming progressively more severe from H1 to H3. These provisions are summarised in Table 2. With reference to Table 2, masonry veneer housing would be classed as "ductile", as it is connected to a ductile timber or steel structural frame. In most cases, therefore, housing in the major population centres in Australia falls within categories H1 or H2. As can be seen from Table 2, the requirements in these categories are fairly nominal, even for non–ductile full masonry structures.

Detailing Requirements
The principal requirement for structural detailing is that all parts of the structure are tied together in both the vertical and horizontal planes to create a three dimensional structural system capable of transmitting the seismic forces to the foundations. The Standard also sets down minimum forces for wall anchorage which must be transmitted to the supporting system. In many cases these details will already be in place for the transmission of wind loads, particularly with regard to the attachment of external walls and the establishment of effective diaphragms at roof level. However, there are significant differences between wind and earthquake loads: earthquake forces are generated on all components (both external and internal); and earthquake forces are predominantly horizontal, whereas wind forces often involve uplift (this results in major differences in the anchorage of roof systems which, for wind, usually involves some form of tie down with axial rather than shear capacity).

For categories H1 and H2 a "deemed–to–comply" housing standard is being prepared.

Table 3. Summary of Design Requirements for Masonry Structures

	Category				
	A	B	C	D	E
Analysis (S = static) (D = dynamic)	Nil	S or D	S or D	S or D (Regular) D (Irregular)	Not Permitted
Height Limit (Storeys)	4	4	3	2	Not Permitted
Detailing	Note (1)	Note (1)	Notes (1) and (2)	Notes (1) and (2)	Not Permitted

(1) Detailing requirements
- Load paths, ties, and continuity
- Connections designed for 0.05 × gravity load
- Wall anchorage – 5(aS) kN/m – Category A
- Wall anchorage – 10(aS) kN/m – Category B

(2) Additional detailing requirements
- More severe requirements for ties and continuity
- Specific diaphragm design requirements
- Ductility requirements on bearing wall connections
- Openings in shear walls and diaphragms to be considered
- Footing tie requirements

This contains acceptable details for both wind and earthquake forces in the appropriate categories. For masonry housing in category H3, static analysis and appropriate detailing is required. This will involve extra design effort and the involvement of a structural engineer, but the number of structures which will fall into this category is relatively limited.

Other Masonry Structures
Unreinforced masonry structures are classed as "brittle", with the design and detailing requirements becoming more stringent as the earthquake design category changes from A to E. A summary of the requirements for unreinforced masonry structures is given in Table 3. A principle similar to that for masonry housing has been adopted, with only detailing required for the less severe categories, with an increasing requirement for static or dynamic analysis and full design for the more severe cases.

The other significant restriction which has impacted on current practice is the height limit imposed on unreinforced masonry structures. All loadbearing masonry structures in excess of four storeys require the use of reinforced masonry for the structural system. In the most severe cases, unreinforced masonry is limited to two storeys. Even though this may seem a dramatic restriction on existing practice, the reality is that it will have little effect on the typical use of unreinforced masonry. Most commercial or residential loadbearing structures in Australia are typically four storeys or less, and the major population centres such as Sydney or Melbourne are not located in severe earthquake zones (usually structure classification A, B, or C).

Probably the most significant impact of the new standard are the requirements associated with tying and detailing. For all structures, the designer must now ensure that load paths are clearly established and that non–loadbearing walls and all free standing elements (such as parapets) are supported. Minimum design loads for supports and connections are also specified in the Standard. Regardless of the level of seismic load adopted, implementation of these two requirements will go a long way towards ensuring adequate seismic performance. The requirement for the design of connections and supports has raised some interesting problems, as the effectiveness of commonly used details must be demonstrated. These aspects are discussed in more detail later.

Static Analysis
In the Standard, dynamic analysis is only required for masonry structures which are irregular and in category D. In all other cases a static analysis is permitted. As is typical for most static analyses, a base shear is determined and static horizontal forces applied down the structure. These horizontal forces are a function of the total base shear and the location in the building. This total base shear V is then distributed down the building as a series of horizontal loads in a manner accounting for the gravity load at any level and the mode of vibration of the structure. The earthquake base shear (V) is determined from

$$V = I\left(\frac{CS}{R_f}\right)G_g \qquad [1]$$

where

I = importance factor
C = earthquake design coefficient (function of acceleration level and structure period)
S = site factor
R_f = structural response factor (1.5 for unreinforced masonry shear walls)
G_g = gravity load

It is significant to note that in many cases the seismic forces so determined will be less than the design wind forces. In these cases, therefore, the consideration of earthquake effects does not have any major cost implications. However, the need to consider the earthquake response still has the advantage of ensuring that the structure is effectively tied together and that all free standing elements are adequately supported. In this process the designer may still have to design the connections or at least establish that the normal practices are adequate. As with all earthquake analysis, torsional and soft storey effects, stability and excessive storey drift may also have to be considered. Vertical ground movements must also be taken into account for earthquake design categories D and E.

DESIGN OF CONNECTIONS

As mentioned above, with the release of the new Standard, designers now have to give attention to the effective attachment of building components and the support of non–loadbearing elements against seismic effects. In the past, the main concern in the design of these connections was related to the effects of wind loads and various forms of differential movements from temperature and other effects. There are fundamental differences in the design of connections for earthquake and wind, the most important being the direction of the force to be transmitted (wind forces are horizontal but will often cause uplift, whereas earthquake forces are predominantly horizontal). Some of these aspects are discussed below.

Roof Connections
In the past the prime consideration for the connection of walls to a roof in loadbearing

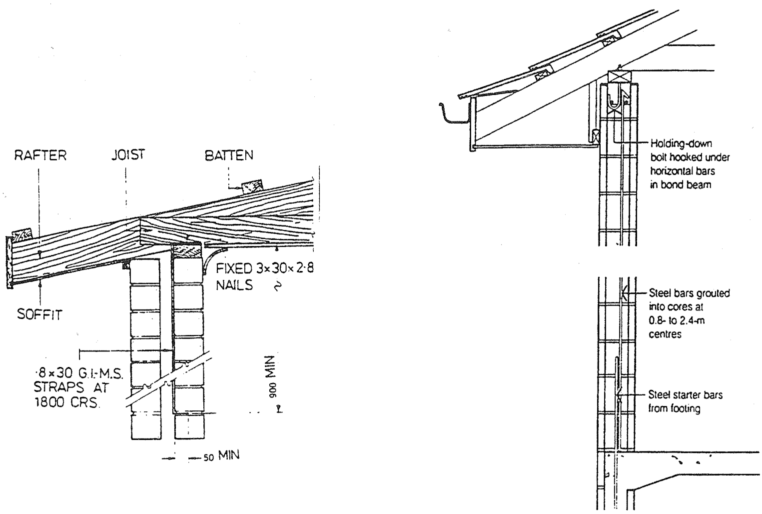

Figure 2. Typical Roof Connection Details

masonry and domestic structures was the transmission of wind forces, with the emphasis being on wind uplift and roof tie down. These effects become more severe with flat roofs and the stronger winds which are encountered in the northern regions of Australia. In moderate wind areas, the connection is typically some form of light steel strapping attached to the masonry to mobilise sufficient wall mass to resist the wind uplift. In cyclonic areas, full tie down is achieved by the use of cyclone rods or bolts extending through to the footings. Typical details are shown in Figure 2.

The earthquake code requires all roofs to be positively attached, with a system capable of transmitting a horizontal force of 5% to 7.5% of gravity load (depending on the category). It is obvious that the strapping detail in Figure 2 will be inadequate for this purpose. Current practice will have to change in many cases, with attachments being designed to provide both resistance to uplift and effective lateral support of the walls.

Floor–Wall Connections
In loadbearing structures consisting of reinforced or prestressed concrete slabs supported by a masonry walling system, the connections between the floors and walls must be capable of providing lateral support to the wall as well as allowing progressive movements to occur between the two elements from the effects of temperature, concrete shrinkage and masonry growth or shrinkage (depending on whether the masonry is clay or concrete). A common connection detail is shown in Figure 3. The normal practice is to incorporate some form of slip joint at the concrete–masonry interface to allow these differential effects to be accommodated. Often one or two layers of a membrane type damp–proof course are used for this purpose. To satisfy the requirements of the new Standard, this connection must also be capable of transmitting a horizontal force of 10(as) kN/m of wall. For unreinforced masonry this requirement creates potential serviceability problems, since if some positive form of attachment is adopted, the long term movements mentioned above will be restrained, thus inducing cracking in the masonry. If a positive form of connection

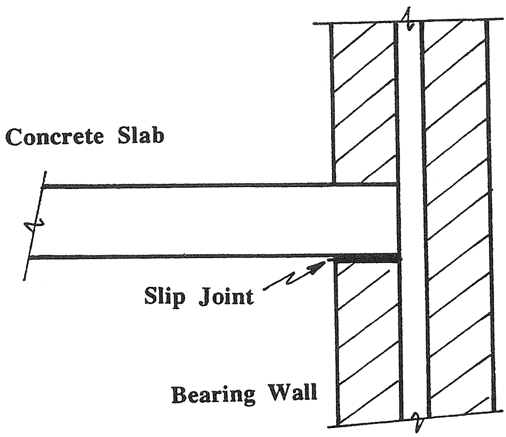

Figure 3. Typical Floor–Wall Connection

is not adopted, then reliance must be placed on the transfer of the seismic force by friction. The frictional capacity of damp–proof courses is discussed below.

Membrane Type Damp–Proof Courses
Membrane type damp–proof courses are widely used in Australia as a barrier at the base of walls to prevent the passage of moisture from the ground to the structure. They typically consist of a flexible membrane manufactured from embossed polythene, or light gauge aluminium covered with polythene or bitumen. The membrane is incorporated in a mortar joint, either sandwiched in the mortar or, more commonly, laid directly on the masonry units with the mortar being placed on top. These same membranes are also used for flashings and in slip joints (see above). The use of these damp–proof courses in masonry walls has significant structural implications, as both in–plane and out–of–plane forces must be transmitted across the joint containing the membrane. The shear capacity of the joint (V_d) in the Australian Masonry Code is given by

$$V_d = C_m f'_{ms} A_b + K_v f_d A_b \qquad [2]$$

where

C_m = capacity reduction factor
f'_{ms} = characteristic shear bond strength of the joint
A_b = bedded area of the joint
K_v = friction factor
f_d = design compressive stress on the plane under consideration (based on the non–removable dead load, taken as 0.8G)

The values of the shear factor K_v and the shear bond strength f'_{ms} for planes containing a damp–proof course have recently been determined in a comprehensive series of in–plane and out–of–plane shear tests (Page, 1995). As would be expected, the shear behaviour depends upon whether the membrane is sandwiched in the joint or placed directly on the masonry units. In both cases, the shear bond strength was low and variable, with the value being higher in the sandwiched case. The friction factors were quite high (in the order of 0.5 in many cases) indicating that the planes do have the potential to transmit reasonable shear forces across the plane by friction. A summary of the results is shown in Table 4 and Figure 4.

For design purposes, it is recommended that the shear bond strength be neglected and the friction factor be taken as 0.30 for most membrane types (Page, 1995). These values are also being recommended for use in calculating the shear capacity of joints containing membranes for design for earthquakes. There is a possible reduction in vertical compression on the shear plane (with an accompanying reduction in shear capacity) caused by the vertical acceleration response of the structure resulting from the vertical component of the ground acceleration. However, this reduction is catered for in the design recommendation: the shear capacity is based on 80% of the dead load acting on the plane; the earthquake load is calculated using the gravity load G_g which consists of the full dead load plus a proportion of the total live load (with the proportion depending on usage but typically 30%–40%). It is assumed that this inherent conservatism will cater for any reduction in frictional capacity produced by the vertical response of the structure (Page, 1995).

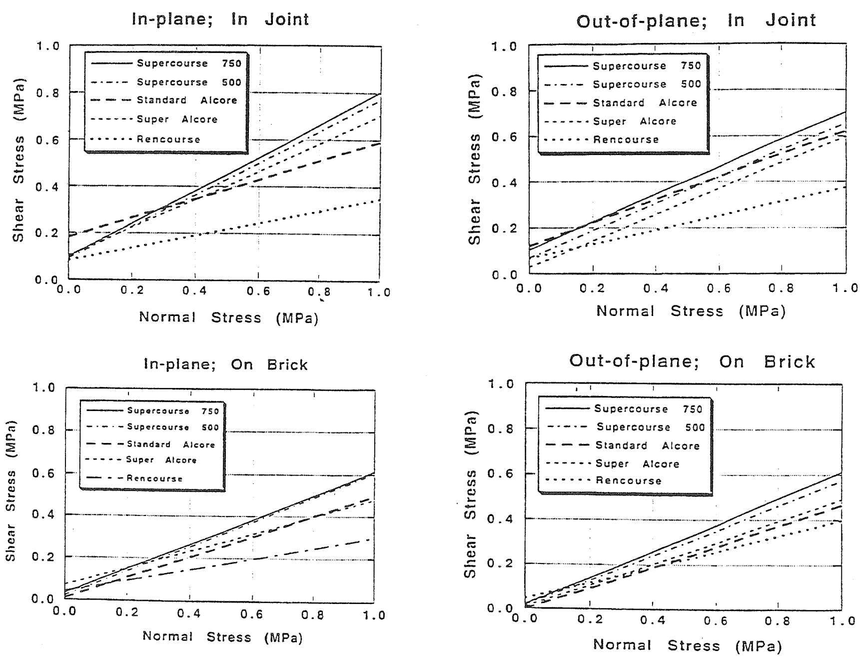

Figure 4. Frictional Behaviour of Membrane Type Damp–Proof Courses

Table 4. Damp–Proof Course Properties

		Friction Factor K_v				Shear Strength f'_{ms} (MPa)			
		In–Plane		Out–of–Plane		In–Plane		Out–of–Plane	
Damp–Proof Course Type	Commercial Name	In Joint	On Brick	In Joint	On Brick	In Joint	On Brick	In Joint	On Brick
Bitumen Coated Aluminium	Standard Alcore	0.41	0.49	0.50	0.47	0.18	0.01	0.12	0.00
Bitumen Coated Aluminium	Super Alcore	0.60	0.41	0.57	0.48	0.10	0.07	0.03	0.01
Polyethylene/ Bitumen Coated Aluminium	Rencourse	0.26*	0.26*	0.31	0.35	0.08	0.04	0.07	0.05
Embossed Polythene	Supercourse 500	0.68	0.59	0.59	0.56	0.09	0.02	0.07	0.02
Embossed Polythene	Supercourse 750	0.71	0.58	0.60	0.59	0.10	0.03	0.11	0.02

* Less than the AS3700 default value for mortar joints of 0.30

MASONRY QUALITY

All of the provisions for the seismic design of unreinforced masonry inherently assume that the design, detailing and construction of the masonry are of a reasonable standard. For unreinforced masonry to perform satisfactorily it is essential that the masonry itself be of reasonable quality with adequate levels of bond strength. (AS3700 assumes that a value of 0.20 MPa for the characteristic flexural tensile strength can normally be achieved without confirmatory testing). It is also essential that the masonry be adequately tied and supported where necessary, and that the masonry be of adequate durability.

Unfortunately in the past this has not always been the case, particularly when the masonry has been non–structural. This was graphically illustrated in the aftermath of the Newcastle earthquake, where a large proportion of the damage to the masonry resulted from poor detailing and construction practice and building deterioration (Page, 1992). One of the reasons for this lack of quality has been the lack of involvement of the structural engineer in the design and supervision of masonry, particularly if it was non–structural. In cases such as this, the supervising architect is concerned mainly with the aesthetics of the finished product, and relies on the tradesman to construct the masonry to an adequate standard. Unfortunately in Australia the standard of bricklaying is highly variable, from excellent to completely unacceptable. There is an urgent need to improve this situation. Apart from the obvious solutions such as better education of the tradesmen, the most important factor is the increased involvement of the structural engineer in the design and construction process, even if the masonry is non–structural. This will involve changes to the current fee structure for consultants, as they are normally not paid to be involved in this process. With the increased emphasis on seismic design, all masonry elements must be considered as "structural" as even non–loadbearing walls are subjected to seismic forces. These aspects are still being addressed.

SUMMARY AND CONCLUSIONS

In countries such as Australia which are subjected to small to medium sized intraplate earthquakes, it is possible to use unreinforced masonry successfully provided it is designed and detailed correctly, and constructed to a reasonable standard. However, it is important that seismic effects be considered at the design stage along with other extreme loads such as wind. Even though in many cases wind loads will govern the lateral load design, there may be some different requirements, particularly with regard to detailing and lateral support of individual masonry elements.

The new Australian earthquake loading code requires that earthquake effects be considered in all parts of Australia for all structures. The Standard also contains restrictions on the use of unreinforced masonry in the areas deemed to be of high risk. In other locations, the requirements range from deemed–to–comply details in low risk areas, to full calculation and design in higher risk areas. Despite these restrictions, unreinforced masonry can still be used in most parts of Australia provided appropriate design and detailing is carried out, and standards of workmanship are adequate. The implementation of the new earthquake code via the Australia building regulations should therefore result in improved levels of performance and safety in future unreinforced masonry structures.

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