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Research Paper 15

Unreinforced masonry structures – An Australian overview

Summary

Unreinforced masonry is widely used in Australia as an architectural and structural material. Because of its high mass, lack of ductility and low tensile strength it is unsuitable for use in areas of high seismicity. However in countries of lower seismicity such as Australia it can be used provided it is designed, detailed and constructed correctly. This paper provides an overview of the use of unreinforced masonry in Australia and discusses the impact of the new seismic loading provisions on existing practice. It is shown that unreinforced masonry can still be used in most instances provided the correct design and detailing techniques are used and the requirements of the appropriate masonry standards implemented.

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Research Paper 15

Unreinforced masonry structures – An Australian overview

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1.0 Introduction

Masonry is one of the oldest building materials known to man. Traditional masonry structures relied on their massiveness of construction for stability, and were proportioned to avoid the creation of tensile stresses. In the period following the second world war traditional loadbearing construction gave way to structures using the shear wall concept, where stability against lateral loads was achieved by aligning walls parallel to the direction of the loads so that the forces could be transmitted by in-plane shear.

This, combined with the use of concrete floor systems acting as effective diaphragms, produced robust 'box' structures with thin walls and effective lateral load carrying capacity. Loadbearing structures of this type offer an economical alternative to framed construction for low and medium rise buildings particularly for structures with repetitive floor layouts.

Unreinforced masonry is a commonly used building material in Australia for loadbearing structures, housing, and for veneer and infill in framed construction. Masonry is widely used because it provides a combined structural and architectural element which is attractive and durable, with good thermal and sound insulation and excellent fire resistance. Loadbearing structures tend to be low rise (although much taller structures have also been constructed) with the commonest example of this type of structure being the three or four storey 'walk-up' apartment building.

Masonry is also widely used in housing, with veneer construction being common in the eastern states, and cavity construction in the west. Single skin, grouted or partially grouted hollow masonry is also used in northern Australia. Various forms of unreinforced masonry are also used in commercial construction both as loadbearing and non-loadbearing elements, often in conjunction with a structural frame.

One disadvantage of unreinforced masonry construction is that it has poor seismic performance, since it is a heavy, brittle material with low tensile strength and exhibits little ductility when subjected to seismic effects. It is therefore unsuitable for areas of high seismicity (although historically unreinforced masonry has been used in the past, often with disastrous results). However in regions of lower seismic activity (such as Australia) unreinforced masonry can be used in most instances provided the structure is designed and detailed for the appropriate earthquake forces and built to the required standard.

This paper gives an overview of the properties and behaviour of unreinforced masonry structures and structural elements, with emphasis on seismic performance. An overview is also given of the current Australian seismic design requirements for unreinforced masonry structures, particularly with regard to the provisions of the new Australian Earthquake Loading Code AS 1170.4[1].

2.0 Masonry properties

Masonry is a composite material consisting of masonry units, (bricks or blocks), set in a mortar matrix. Because of its composite nature and the different properties of the units and mortar, masonry exhibits distinct directional properties and contains potential planes of weakness created by the low tensile (bond) strength at each mortar-unit interface. Despite its centuries of use, the fundamental behaviour of masonry has only been studied and reasonably understood over the last twenty years or so[2, 3, 4].

The need for a better fundamental understanding of masonry behaviour has been prompted by the use of thinner more highly stressed walls, and the increased sophistication of the methods of analysis (such as finite element techniques) for which more sophisticated methods of analysis are required. The methods depend for their accuracy on realistic constitutive models for the materials.

Masonry Units

There is a wide range of masonry units which can be solid or hollow (in the form of bricks or blocks) and made from fired clay, concrete, calcium silicate, or natural stone. Masonry units must be sound and have sufficient strength for the relevant application. Masonry units will normally exhibit elastic-brittle properties with typical compressive strengths of 20 MPa to 100 MPa for fired clay units, 10 MPa to 30 MPa for concrete and calcium silicate units, and as low as 3 MPa to 5 MPa for autoclaved aerated concrete units. Solid masonry units are usually laid with a full mortar bed; hollow units are laid in face-shell bedding with the mortar only applied to the face-shells of each unit. Masonry units must also be durable and not exhibit efflorescence, pitting due to lime particles, or be susceptible to salt attack. In highly corrosive areas, exposure grade units should be used to avoid durability problems.

Mortar

Mortar is the most important ingredient as its characteristics have a strong influence on the strength and durability of the masonry assemblage. It is also the ingredient most susceptible to site problems related to mixing and batching. Mortar must be workable, have sufficient strength, and be adequately bonded to the masonry units. The most effective mortar consists of cement, lime, and a well graded sand, with the proportion of cement increasing with an increasing requirement for durability.

In recent years there has been a trend away from the use of lime, with plasticising additives being used in its place to enhance the workability of the mix. This is undesirable, as lime improves the properties of the mortar in both its plastic and hardened state. Plasticisers are also commonly overdosed and this can dramatically reduce the masonry bond strength as the entrained air reduces the degree of contact at the interface between the mortar and brick.

The best way to achieve workability is by the use of a well graded sand. If this is not available, workability can be improved by the use of lime putty (obtained by pre-soaking hydrated lime), or by the use of a methyl cellulose water thickening additive which retains water in the mix and does not entrain air. Recent Newcastle experience has also shown that mixing of the sand, lime and water for several minutes before adding the cement will significantly improve the mortar workability.

Masonry

The requirements for good quality masonry are set down in the SAA Masonry Code AS 3700[5] and complementary documents[6,7]. The most important properties affecting structural performance are the compressive and tensile strengths. The compressive strength is directly related to the strength of the units, with the strength being substantially less than the unit strength because of the influence of the mortar. The tensile strength is governed by the bond between the mortar and the units as this is typically less than the tensile strength of either of the constituent materials.

Masonry bond strength can vary from zero to more than 1 MPa depending on the correct match of mortar and unit properties, particularly the water retention of the mortar and the suction of the masonry units. The fundamental mechanism of bond between mortar and masonry unit is not fully understood although it appears to be primarily mechanical rather than chemical[8,9].

A collaborative in-depth study of this phenomenon is in progress at the University of Newcastle and CSIRO. At a practical level the value of the bond strength is important as it directly controls the tensile and shear strength of the masonry and hence its capacity to resist transient loads from wind or earthquake. Bond strength is particularly susceptible to workmanship effects and mortar 'abuse' -factors which can only be controlled by effective site supervision.

The SAA Masonry Code AS 3700[5] allows the designer to assume a characteristic flexural tensile strength for masonry of 0.20 MPa without confirmatory testing. This value can be achieved relatively easily with the correct choice of ingredients and laying techniques, but lower values will result if good practice is not followed. The most effective way to assess the bond strength is to carry out a small series of bond wrench tests on masonry constructed on site by the bricklayer from the ingredients to be used. Prisms are constructed on site, cured for seven days, and then each joint tested with a bond wrench in accordance with AS 3700. In-situ bond wrench tests are also an effective means of establishing the strength of existing masonry.

Ties and tying

Since masonry is a brittle material with limited tensile strength, it must also be supported by suitable tying systems to keep the flexural stresses within reasonable limits. Wall ties are used to connect non-loadbearing veneer walls to a structural back-up, and to allow cavity walls to each share in the transmission of the applied loads. Ties come in a range of sizes and shapes depending on the application, and are usually made from steel with some form of protective coating, or of stainless steel where high durability is required. A range of plastic ties have also been developed recently[10].

Ties must have appropriate strength and stiffness and be installed correctly. The current Australian Standard for Wall Ties[11] classifies cavity ties into four duty ratings for strength and stiffness: light; medium; heavy; and extra heavy. These characteristics are assessed by tension and compression tests on small masonry-tie assemblages with the tests reflecting both the behaviour of the tie itself and its attachment to the masonry and/or the veneer. The stiffness values obtained therefore include not only the deflection of the tie but also take-up or other distortions of the attachments. The new joint Australian-New Zealand standard due for release shortly[12] also contains additional ductility requirements based on cyclic tests for ties in more severe seismic areas such as New Zealand.

The other factor which directly affects masonry performance is tie durability, particularly in coastal areas. Durability requirements for wall ties are specified in AS 3700, which requires the use of stainless steel ties (or equivalent) in highly corrosive areas (such as coastal and industrial zones). In the light of widespread evidence of corrosion of ties and fitments these durability requirements are likely to be made more stringent in the updated masonry code which is due for release in 1996.

3.0 Structural design of unreinforced masonry

Introduction

Until the advent of the new Earthquake Loading Code in 1993 unreinforced masonry (URM) structures were usually designed for dead, live and wind loads, with seismic loading often not being considered. A previous earthquake loading code did exist, but since it was not incorporated into the building regulations and in most cases the level of seismic risk was perceived to be low or non-existent, it was usually ignored. Lateral loading considerations therefore related to wind load, which in many cases produce equivalent static loads at least as severe as the current AS 1170.4 requirements.

Consideration of these forces would thus cater for some earthquake effects, but much less emphasis would have been placed on general building layout and detailing (particularly soft storey effects), and lateral loads on internal walls and other internal elements would not have been considered. The 1989 Newcastle earthquake highlighted the need for seismic design, and with the recent incorporation of AS 1170.4 into the Building Code of Australia, consideration of seismic effects is now mandatory.

Because of the low level of Australian seismicity it is feasible to use property designed and constructed unreinforced masonry in most areas. However there is an urgent need for research into the behaviour of URM structures particularly with regard to Australian practice (in relation to the behaviour of connections, slip joints, damp-proof courses, veneers, etc.). Despite extensive international research into the seismic behaviour of masonry structures the bulk of this work has been in relation to reinforced masonry. Most of the unreinforced masonry research which has been performed has concentrated on the behaviour of existing rather than new structures.

Since the Newcastle earthquake and the release of AS 1170.4 there has been an increase in research interest in the seismic performance of Australian URM structures. This research is extremely valuable as it is investigating specific problems related to Australian building practice and Australian levels of seismicity[13-19].

Seismic design

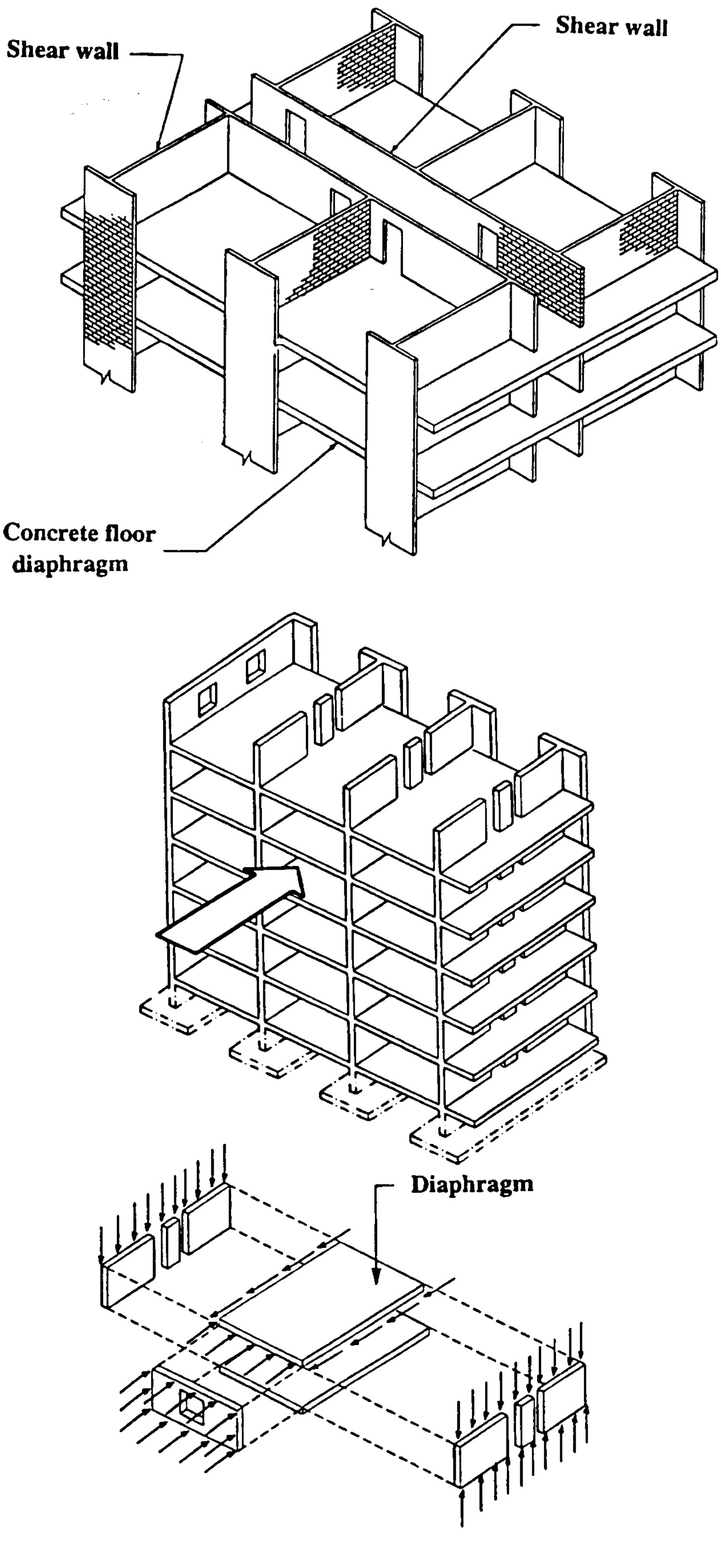
Unreinforced masonry structures have extremely limited ductility so they must be designed to remain essentially elastic. In design, the structure is therefore required to carry a higher level of applied load than a ductile structure. (This is the reason for the low structural response factor Rf in the equivalent static analysis provisions of AS 1170.4 – R_f for an unreinforced shear wall system is 1.5; the corresponding value for a reinforced masonry system in 5.0). As for all seismic design, clear load paths must be established for the structure, and irregularities in plan and elevation must be considered. The most effective seismic resistance will be provided by the 'box' structures described in the next section.

The establishment of loads paths includes the effective transmission of seismic forces across the various connections and any other discontinuities in the structure (i.e. , the influence of flashings, membrane type damp-proof courses and slip joints must be considered).

The other significant feature of unreinforced masonry is its inherently low tensile strength. The tensile bond strength of the masonry should be maximised and tensile stresses avoided wherever possible. Free standing elements must not remain unsupported and veneer elements and other attachments tied to the back-up framing system. These aspects are discussed in more detail in the following sections.

Loadbearing structures

Loadbearing structures can be single or multi-storey with the commonest Australian applications being up to five storeys in height, although much taller loadbearing structures have been built (particularly in Western Australia). In all cases the design principles are the same, with vertical loads being transmitted to the foundations by bearing wall action and lateral loads being carried by the composite 'box' structure.



The seismic performance of a loadbearing structure will predominantly depend upon its lateral load resistance. The basic mechanism of lateral load transmission for a loadbearing structure is shown in Figure 1 . The lateral load will either be applied to the structure by wind or induced in the structure by earthquake ground movements.

In either case walls aligned normal to the loading direction will be subjected to face loads with these elements spanning vertically between the horizontal diaphragms (floors). In some cases they may also span horizontally between loadbearing walls. The concrete floors then act as rigid diaphragms and transfer the load to the shear walls which in turn transmit the forces to the ground by in-plane shear.

The resulting structures are usually quite robust, with relatively short span concrete slab systems supported by numerous walls running in both principal directions. The effective performance of this system obviously depends on the ability of the individual masonry elements to sustain their share of the load, and the capability of the connections between the elements to transmit the appropriate forces.

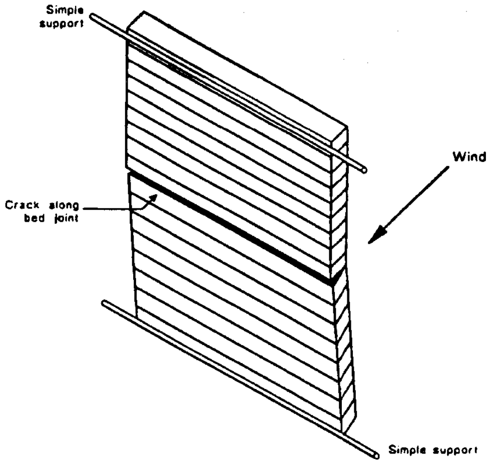
The performance of each of these masonry components is discussed in the ensuing sections.

Figure 1. Load Transmission By Shear Wall Action

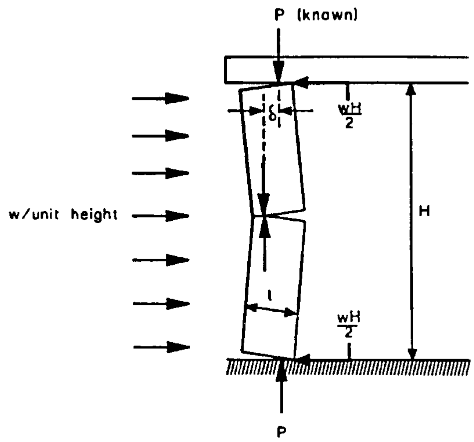
Masonry subjected to face loading

The capacity of unreinforced masonry subjected to face loading depends upon the panel dimensions and support conditions, the level of compressive stress in the wall, and the tensile strength of the masonry. For veneer or lightly loaded panels where the level of compressive stress is low, flexural tensile strength is particularly important.

The capacity of vertically spanning panels which are not built in to rigid supports will be limited and depend upon the flexural capacity of the horizontal bed joints. The flexural capacity in turn will be a function of the level of precompression and the flexural strength of the masonry (see Figure 2a). If the top and bottom of the wall is restrained then significant in-plane arching can develop, and even though flexural cracking occurs at mid-height, collapse will not take place until a mechanism forms. The capacity in this case will be much higher (see Figure 2b). Unfortunately in non-loadbearing panels this in-plane arching will not develop as expansion gaps are normally left around the edges of the panel to allow for long term masonry moisture movements and the deflections of the adjacent structure.



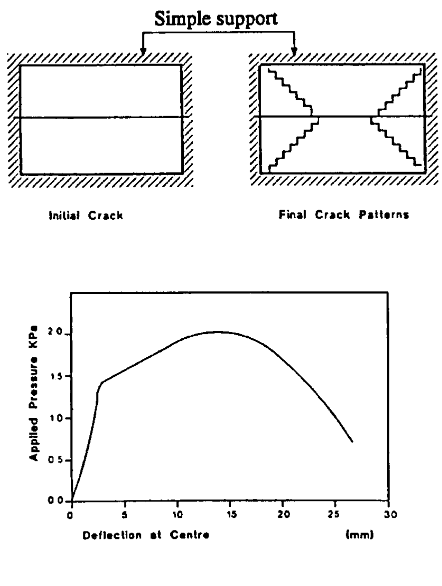
a. No Top and Bottom Restraint



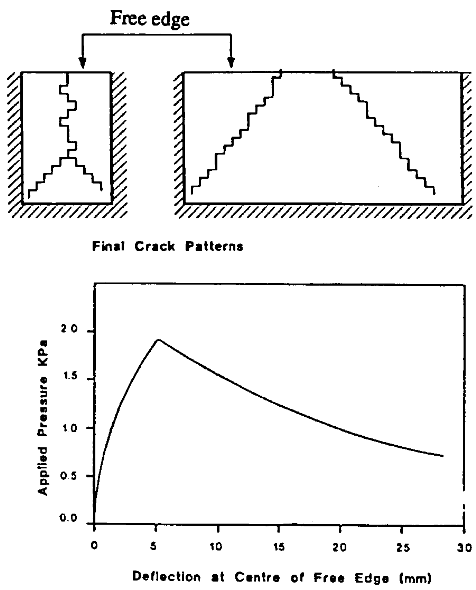
b. Top and Bottom Restraint

Figure 2. Lateral Load Behaviour of Vertically Spanning Walls

Masonry panels which are also supported on their vertical edges have a higher lateral load capacity, as two-way plate action is induced. In all cases failure will not occur after the formation of the first crack but only after a collapse mechanism has developed (this will depend upon the geometry and boundary conditions). Typical lateral load deflection curves and collapse mechanisms are shown in Figure 3. If in-plane arching is able to develop the capacity will be further enhanced.



Four Sided Support



Three Sided Support

Figure 3. Lateral Load Behaviour of Two Way Spanning Walls

Various methods have been suggested for the analysis and design of masonry subjected to lateral loading. These have been recently reviewed by Drysdale *et al*[3]. All arc semi-empirical and vary from country to country. At present there is a no international consensus on the most appropriate technique.

A fundamental solution to the problem has been hampered by the lack of representative constitutive relations for the material. Work is continuing in this area. The Australian Masonry Code currently uses the Strip Method of design which involves summing the capacity of horizontal and vertically spanning strips of masonry, with each strip being considered independently. This works well for solid walls but has some limitations when openings are present. These issues will be resolved in the next edition of the Standard.

All of the above discussion relates to the performance of masonry subjected to equivalent static loads. When the face loading is produced by seismic effects, the equivalent static load must account for the level of ground acceleration, the dynamic response of the structure itself, and the location of the wall in the structure. These aspects have been investigated by Priestley *et al*[20, 21] who show that the input accelerations to walls at different levels are not only of different magnitude but also may be out of phase or have significantly different frequency compositions. Priestley has also investigated the mechanism of collapse of walls spanning vertically between rigid supports subjected to cyclic loading, with the wall rocking about the central cracked joint in a mechanism similar to that shown in Figure 2, and derived the equivalent linear elastic response acceleration for this case.

Parapets and free-standing elements

Parapets and other free standing elements arc commonly used in unreinforced masonry structures. These elements have little resistance to lateral load due to the low flexural strength of the masonry, and hence rely on gravity for stability. The presence of some form of flashing at the base of these elements exacerbates the situation. In addition, these elements are usually located at or near the top of the structure where the effects of ground motion are magnified by the dynamic response of the building. The provisions of AS 1170.4 allow for this effect by the application of a height amplification factor which has a maximum value of 2. This is a simplification of what is quite a complex phenomenon[17].

It is obviously desirable to avoid the use of such elements, or if they must be used, for them to be supported or locally reinforced to provide flexural strength. Grouted and reinforced cavity construction or hollow clay or concrete masonry can be used for this purpose. Alternatively unreinforced parapets can be designed to span horizontally between returns or piers which can be designed to provide overall stability.

Masonry subjected to in-plane shear

The general principles of shear wall analysis are well known. The force in a particular shear wall will depend upon its relative stiffness and in some cases the flexibility of the floor diaphragms connecting the shear walls. Shear walls can fail locally in three ways under varying conditions of biaxial stress: compression failure at the toe; tensile failure at the heel; or shear failure (see Figure 4).

The stress distribution within a shear wall is complex and will depend upon the geometry of the wall, the nature of the load application, the presence of openings, etc. Racking tests on individual wall panels show that failure can occur in one or combinations of the above modes. Local failure will occur when the local biaxial stresses reach a critical value.

The strength of masonry subjected to biaxial stress depends not only on the magnitude and sense of the principal stresses σ_1 and σ_2 but also on their inclination (θ) to the bed and header joints which act as planes of weakness. This is particularly critical if tensile principal stresses are present. In general a three dimensional failure surface in terms of the variables (σ_1, σ_2 and θ) is required to define failure in local regions of a shear wall[4]. Consideration of this local state of stress is necessary if local and progressive failure is to be predicted using finite element or similar techniques. From a practical point of view local failure may have implications for serviceability, but final failure is the main area of interest. Consequently design rules have been formulated from racking tests on masonry panels and observed performance at failure, resulting in a simplified relationship expressed in the form of a Coulomb criterion in terms of the average shear and compressive stresses in the wall (thus ignoring the variations in local stress distribution in various parts of the wall).

The nominal shear strength is given by the following expression which is a lower bound linearisation of the failure surface for lower values of σ_n. Note that the panel is no longer behaving elastically, and its load-deflection behaviour will influence the building response.

$$\tau = \tau_0 + \mu\sigma_n \qquad 1$$

where

τ = shear strength at precompression on

τ_0 = shear strength at zero precompression

μ= an apparent friction coefficient

The parameters in this expression vary considerably, not only because of different material characteristics, but also because of factors affecting the actual stress distributions in the wall[22] . The expression becomes invalid for high levels of σ_n, but these are normally outside the practical range.

Figure 4 shows the potential regions of local failure. Toe failure will occur by crushing under biaxial compressive stress. Failure usually occurs by splitting and spalling normal to the plane of the wall. Failure at the heel occurs when vertical loads are low in relation to the racking load resulting in the development of tensile stresses normal to the bed joint with a consequent horizontal crack. Failure in the centre of the panel is commonly described as 'shear failure' and is typified by diagonal cracking. Failure actually occurs in the bed and header joints under a combination of principal tensile and compressive stresses with subsequent sliding along the joints. The magnitude and inclination of the principal tensile stress is influenced primarily by the ratio of vertical load to horizontal racking load with the 'shear strength' of the wall increasing significantly with the increasing amount of vertical load.

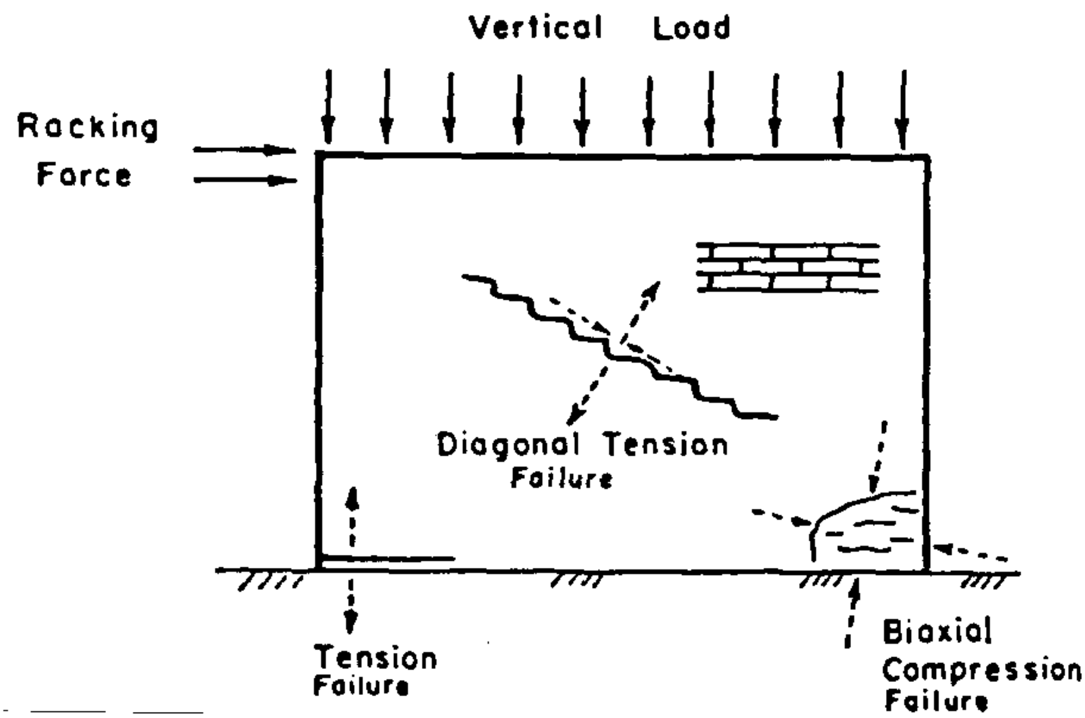


Figure 4. Modes of Failure for a Shear Wall

Unless major openings or discontinuities are present none of the above failures will normally cause complete collapse of the wall, although obviously its capacity may be impaired. Walls subjected to seismic loading will progressively degrade with the repeated load reversal as all or some of the above failures occur in the locations appropriate to the direction of loading. In most cases under cyclic loading a wall will rock on its base as uplift occurs at the appropriate end of the wall[13, 14, 16, 23]. This may correspond with gradual shedding of bricks from the tension end and/or progressive local crushing in the compression region. Because of this process, and possible progressive diagonal failure, unreinforced masonry shear walls do have some capacity for energy absorption. For walls with major openings significant distress and failure may occur in the masonry piers between the openings[20].

4.0 Connections

As mentioned previously the AS 1170.4 provisions require consideration of 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 has been 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 rather than wind. Wind loads mostly relate to the external components of the building, and for roofs the forces will often be upward. Earthquake forces will be predominantly horizontal and occur in both internal and external elements of the structure. These aspects have been discussed in more detail elsewhere[24] but will be briefly reviewed here.

Roof connections

The earthquake standard requires all roofs to be positively attached with a system capable of transmitting a horizontal force of 5 percent to 7.5 percent of gravity load (depending on the category). Many of the current wind hold down details (such as strapping connections) are inadequate for this purpose.

Floor-wall connections

In loadbearing structures consisting of 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 5. 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, or a more positive slip joint comprising two layers of galvanised steel sheet with a layer of graphite grease between them may be used. To satisfy the requirements of the new Standard this connection must be capable of transmitting a horizontal force of 10(aS) kN per metre length of wall (where a is the acceleration coefficient and S the site factor). 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 is not adopted, then reliance must be placed on the transfer of the seismic force by friction. The frictional capacity of damp-proof courses and slip joints is discussed below.

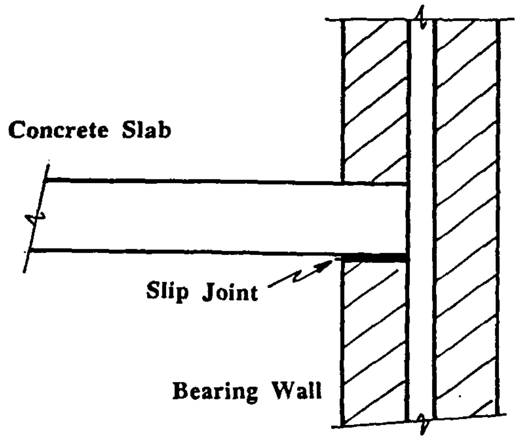


Figure 5. Typical Slab-Wall Connection

Shear capacity of membranes and joints

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 joints 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 (Vd) in AS 3700 is given by:

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

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 static shear tests[25]. Some supplementary tests on slip joints have also recently been performed. 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 given in Table 1 for damp-proof courses and Table 2 for slip joints.

Table 1. 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 AS 3700 default value for mortar joints of 0.30

Table 2. Slip joints between reinforced concrete slabs and brick masonry walls

Joint type	Friction factor (out-of-plane loading)
One (1) layer of Super Alcore (bitumen-coated aluminium)	0.57
Two (2) layers of Super Alcore	0.48
Two (2) layers of galvanised steel with molybdenum disulphide grease	0.06
ABA-FEL slip joint system (similar to above)	0.07

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[25]. In the absence of any dynamic test results these values are also being recommended for earthquake design for calculating the shear capacity of joints containing membranes. There will be 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 to the vertical component of the ground acceleration. However, this reduction is catered for in the design recommendation: the shear capacity is based on 80 percent 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 to 40 percent). It is assumed that this inherent conservatism will cater for any reduction in frictional capacity produced by the vertical response of the structure[25].

There is an urgent need to confirm the dynamic behaviour of these connections as well as verify the specified horizontal force values for lateral supports specified in AS 1170.4. Recent work by Klopp and Griffith has cast some doubt on the specified values[26].

5.0 Masonry veneer

Unreinforced masonry is widely used as a veneer in residential and light commercial construction. Veneers are non-structural elements and rely on the supporting backup frame or wall and the accompanying tying system for stability. Although they are non-structural, the seismic performance of veneers is important because of their widespread use and the consequent high cost of repair if their performance is inadequate.

The behaviour of a veneer subjected to face loading is quite complex as it depends upon the relative flexibility of the veneer and the back-up system, and the stiffness and location of the wall ties. These factors affect the degree of load sharing between the veneer and back-up and the amount of load re-distribution which can occur. There is also a substantial difference in behaviour when the veneer is cracked rather than uncracked, as in its uncracked state the veneer is usually much stiffer than its back-up. A more detailed discussion of this behaviour with regard to wind loading can be found elsewhere[3, 7]. For design purposes it is necessary to know the individual wall tie forces and the corresponding loads on the back-up frame or wall. Typical tie force distributions are shown in Figure 6 for the elastic analysis of a veneer system consisting of medium duty ties connecting an external 110 mm brickwork skin and a back-up system consisting of either a typical stud wall or a highly rigid frame.

It can be seen that the force in each tie is directly influenced by the stiffness of the back-up frame with the forces being tensile in some locations when the frame is flexible. Before the wall cracks the top ties adjacent to the frame support attract a much greater proportion of the load than would be expected from their tributary area. This explains the logic of deemed-to-comply rules which require the number of ties to be doubled in these locations. If the veneer cracks longitudinally at mid-height along a bed joint there is a dramatic redistribution of load in the ties (see Figure 6), with the ties near the mid-height of the wall now being heavily loaded, particularly when the back-up is flexible. It is obvious that the ties play a crucial role in this interaction and their strength and stiffness are both important. The integrity of the tying system itself (with regard to both tie anchorage and the long term durability) are also of the utmost importance.

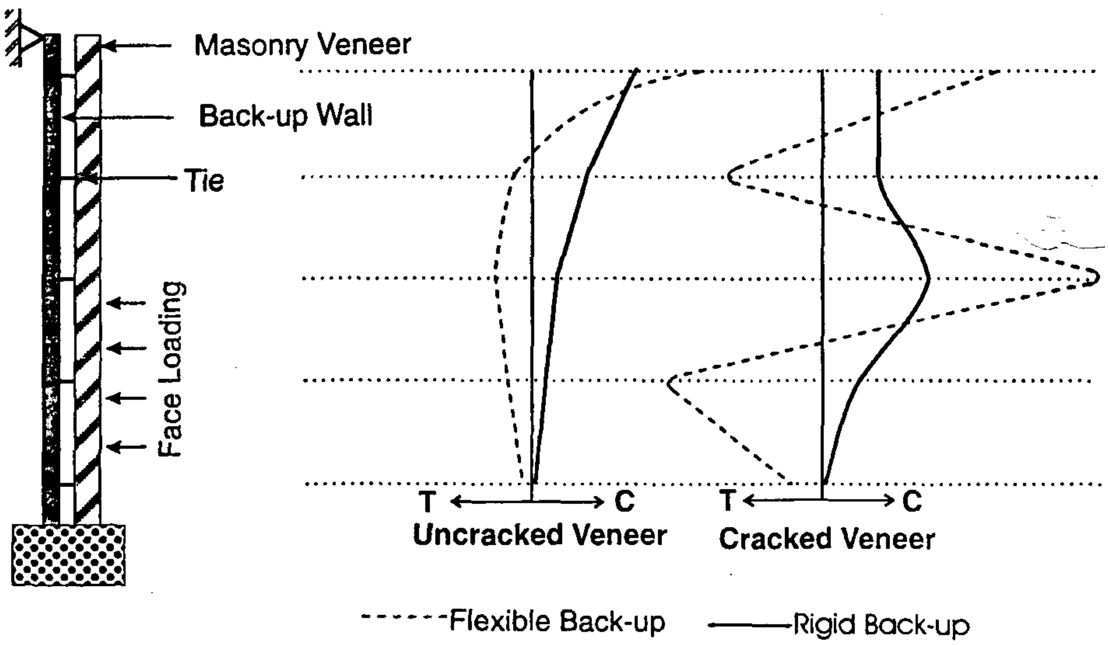


Figure 6. Variation in tie forces for masonry veneers

Behaviour of veneer under seismic loads

The same interaction between the veneer and back-up would be expected for dynamic loading although the interaction will be influenced by the dynamic response of each of the components. Under earthquake loading it is also possible that the veneer would be pre-cracked in shear from in-plane loading effects as the stiffer veneer will initially attract substantial loads even though it is nominally non-structural.

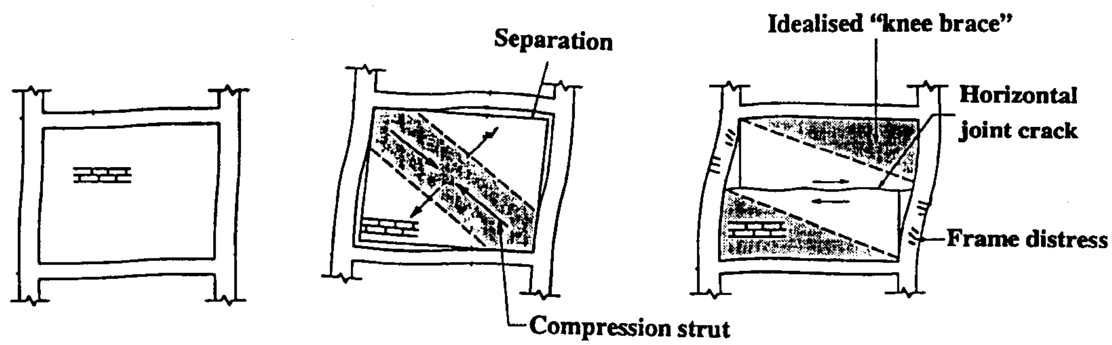
New Zealand studies on the dynamic performance of veneers and veneer tying systems have been carried out by Priestley *et al*[27], Shelton and King[28], Lapish[29], and Allen[30]. This work has shown that unreinforced veneers constructed to accepted specifications would be capable of performing adequately under typical Australian response accelerations. Because of the cyclic nature of the loading the anchorage of the ties is critical, with nailing of the tie to the face of the stud being suspect[28]. The performance of typical Australian wall tie connections to cyclic loading have not been fully assessed and it is possible that some anchorage modifications may be required (for example there is some evidence from the recent Ellalong earthquake that the wire ties which clip on to the flange of steel stud framing may need modification). The revised Australian/New Zealand Wall Tie Standard[12] will include procedures for a cyclic tie test which will allow the more realistic assessment of their cyclic behaviour.

6.0 Masonry infill

Many steel and reinforced concrete frames are built with unreinforced masonry infill panels which have the potential to add considerably to the strength and rigidity of the composite structure. If the masonry is acting compositely in the frame and its contribution has not been considered in the lateral load analysis, a completely erroneous prediction of the response of the structure will be obtained.

A detailed discussion and overview of this phenomenon has been recently presented by Drysdale *et al*[3] . The interaction between infill and frame depends on the area of contact at the interfaces of the two components, and the extent of composite action will depend on the level of lateral load, the degree of bond or anchorage at the interfaces, and geometric and stiffness characteristics of the frame and infill masonry. The possible interactions are shown in Figure 7 with the mechanism at ultimate being either the formation of a diagonal strut in the masonry or a 'knee braced' system due to shear failure along a horizontal bed joint.

The above mechanisms depend on effective contact between the frame and infill. In many overseas countries it has been common practice to build the masonry hard up to the frame. This was particularly the case for existing buildings where little attention was given to this interaction in the original design, with the masonry being assumed to act purely as an architectural infill[31]. This can have disastrous consequences if the contribution of the infill is ignored in the seismic analysis. The mechanism of composite action is less likely to be mobilised in Australia as it is common practice to leave gaps at the vertical edges and top of infill panels to allow for long term moisture expansion of the masonry. The infill panels are then attached to the frames by flexible ties. In this case composite action will not occur until large frame deflections have occurred. The possibility of the mobilisation of the infill should be considered at the design stage, with the best course of action probably being to ensure that the masonry is isolated from the frame by appropriate detailing.



(a) Full Composite action (b) Compression strut (c) Knee brace

Figure 7. Behaviour of masonry infill (after Dysdale *et al*[3])

7.0 Seismic behaviour of unreinforced masonry – lessons from Newcastle earthquake

Nature of the damage

The 1989 Newcastle earthquake was relatively small (estimated as magnitude 5.6) but caused thirteen deaths, numerous injuries, and a disproportionate amount of damage. The bulk of this damage was to unreinforced masonry with estimates of the total cost of damage now being well in excess of $1 billion, making it the most expensive natural disaster in Australia's history. The effects of the earthquake were exacerbated by the presence of soft soils and the presence of old structures in the worst affected areas. Details of the earthquake have been described elsewhere[19,32-34] but it is useful to briefly examine the nature and causes of the damage as they relate directly to the seismic design of unreinforced masonry.

A summary of the damage to masonry is given in Table 3 (over page). It is apparent that much of the damage could have been avoided if even rudimentary seismic design requirements had been in place. It is significant to note that up until the publication of AS 1170.4 in 1993, Newcastle was deemed to be located in a zone of zero seismic risk with no seismic design provisions in force. All lateral loading design was related to wind load only with many aspects crucial to seismic performance not being considered. As indicated in Table 3 the new AS 1170.4 provisions now force engineers to consider seismic effects at the design stage, thus potentially avoiding the same problems in new structures (although the problems remain for many existing structures throughout Australia).

Masonry quality

Apart from the general design aspects described above the other major contributing factor to the scale of the damage was the general quality of masonry construction[19]. As demolition and repairs proceeded, continuing evidence of inadequate standards of masonry design, detailing and construction emerged, often in modern 'engineered' structures. The problems appear to have resulted from the lack of involvement of the structural engineer and/or the architect in aspects of the building design related to masonry. This was particularly the case for framed structures with masonry infill where the engineer was involved only with the reinforced concrete or steel frame and not at all involved with the detailing or supervision of the non-structural masonry cladding and infill. This aspect must be addressed if future problems are to be avoided. Problems particularly related to mortar abuse (with resulting low bond strength), poor general workmanship, and poor tying and detailing (with the complete omission of the tying system in some extreme cases!).

Table 3. Summary of damage to masonry in the Newcastle earthquake

Nature of damage	Cause of damage	Comments
General	Lack of consideration of earthquake loading	No requirements for seismic design
	Soft soil effects	Not considered in design
	Building layout in plan and elevation	Design judgement - seismic implications not considered
	No clearly defined load paths	Design judgement - seismic implications not considered
	Masonry deterioration, particularly lime mortar joints	Lack of inspection and maintenance
	Most of the above now covered by AS 1170.4	
Failure of masonry under face loading*	Inadequate bond strength	Incorrect mortar and poor workmanship
	Wall tie corrosion	Inadequate durability and lack of inspection and maintenance
	Inadequate tying to back-up	Ties of inadequate strength and/or stiffness. Ties incorrectly installed (or omitted)
	All of the above covered by AS 3700 requirements	
Vertical cracks at comers under face loading	Stiff returns at vertical edges causing comer failure	Detailing problem (not considered in design)
Collapse of free standing masonry elements(parapets, chimneys) *	Unstable under earthquake forces	Not considered in design
	Now covered by AS 1170.4	
Collapse of gable ends*	Inadequate tying to roof structure	Not considered in design and/or poor detailing
Damage to masonry cladding and/or infill in framed construction	Excessive deflection of frame and/or bracing and inadequate tying and attachment of masonry	Not considered in design and/or poor detailing
	Now covered by AS 1170.4	
Sliding on membrane type damp-proof courses	Inadequate frictional capacity	Not considered in design
	Now covered by AS 1170.4	
In-plane diagonal cracking	Masonry shear failure	Not considered in design
	Now covered by AS 1170.4 and AS 3700	
Displacement or rotation of internal non-loadbearing masonry walls	Lack of support or tying	Not considered in design or detailing
	Now covered by AS 1170.4	
Collapse of suspended awnings supported by masonry	Failure of suspended tie anchorage	Design and detailing errors
Cracking in older masonry structures	Excessive deflection of timber floor and roof diaphragms	Not considered in design
	Building alterations	Not considered in design

* Some of these elements would also have failed under a design wind.

(Note that most of the above problems could have been avoided or minimised by even nominal seismic design requirements and adherence to current masonry codes).

8.0 Code requirements for unreinforced masonry

As mentioned previously the consideration of earthquake loads is now mandatory for all parts of Australia. The provisions have particular significance for unreinforced masonry particularly with regard to height limitations for loadbearing structures and the increased loading levels specified. The revised edition of the SAA Masonry Code also considers seismic effects and will include an appendix specifically related to seismic design. A brief summary of the implications of the new seismic provisions for unreinforced masonry structures is given below.

General masonry structures

All requirements depend on the 'Structure Classification' which are summarised in Table 4. 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 5. Only detailing is 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 only two storeys. The reasons for these height restrictions are unclear. They do impact on current practice, although probably to only a limited extent. The majority of unreinforced masonry is used in residential construction, and low rise commercial and industrial structures. Most of these structures 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).

Apart from height limits 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 arc also specified in the Standard (see Table 5 footnotes). Regardless of the level of seismic load adopted, implementation of the tying and detailing requirements alone will go a long way towards ensuring adequate seismic performance.

Table 4. Earthquake Design Categories

	Structure classification				↑ Increased ground movement
	General Structures			Domestic	
	Type I	Type II	Type III		
$aS \geq 0.2$	E	D	C	H3 (B equivalent)	
$0.1 \leq aS \leq 0.2$	D	C	B	H2 (A equivalent)	
$aS < 0.1$	C	B	A	H1 (A equivalent)	
← Increased need for survival of the structure					

Table 5. 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 x 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

Domestic construction

As indicated in Table 4, 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 6. Masonry veneer housing is 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 6, the requirements in these categories are fairly nominal, even for non-ductile full masonry structures. For Categories H1 and H2 a 'deemed-to-comply' housing standard is being prepared.

Table 6. Summary of Design Requirements for Domestic Structures

Earthquake Design Category	Ductile	Non-Ductile
HI	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 percent of gravity load reaction - H2)
- Beams and truss connections (7.5 percent of gravity load reaction - H3)
- Wall anchorage to transmit 10(aS) kN/m of wall

Existing structures

There is a large stock of existing unreinforced masonry structures in Australia. Many of these are of the traditional form of construction with massive external masonry bearing walls and internal framing systems often of timber. There is an increasing trend to recycle structures of this type with the accompanying need to improve their seismic resistance. In addition there is a need to improve the seismic resistance of other masonry structures as part of hazard reduction programs. It is significant to note that there are a large number of existing masonry structures in many Australian cities very similar to those that performed badly in Newcastle - many are reasonably old, with free standing elements and parapets, and often in poor condition. There is an urgent need for a hazard mitigation program for structures of this type, not necessarily to upgrade their performance to that of new buildings, but certainly to reduce the risk of death and injury to the general public in the event of an earthquake. Newcastle City Council is one of the few local government instrumentalities with this type of program in place.

(There is some resistance to the implementation of such programs as they may involve considerable cost for the building owner with the accompanying political implications).

One important aspect that must be addressed is the inspection and upgrading of all existing Type 1 masonry structures, as these must be capable of fulfilling their post-disaster function in the event of an earthquake. There are numerous examples of older masonry buildings of this type throughout Australia, and it is essential that their potential performance be assessed and upgraded if necessary. (This was one of the recommendations of the Institution of Engineers Australia report on the Newcastle earthquake[32], but to date little, if any, action has been taken).

A Standard containing provisions for the upgrading of existing buildings is in the final stages of preparation. It recognises that in many cases it may not be economically feasible to upgrade a building to provide it with the capacity to withstand the full earthquake loads specified by AS 1170.4. Rather, the philosophy that has been adopted is to require the structure to withstand a seismic load of a given proportion of that required by AS 1170.4. This ensures that the designer must consider earthquake effects using the same philosophical approach as AS 1170.4 and assess the state of the existing structure, establish clear load paths for vertical and lateral loads, support free standing elements, stiffen and mobilise floor diaphragms, and generally assess the seismic performance of the structure. Deemed-to-comply details for many applications are to be included as an Appendix to the Standard as well as lower bound default values for material properties (together with in-situ test methods for determining material properties if desired).

9.0 Summary and conclusions

Unreinforced masonry is a commonly used building material in Australia as it is economical, attractive and durable, with good thermal and sound insulation and excellent fire resistance. Masonry is thus widely used for loadbearing elements as well as for infill and cladding in domestic and framed construction. Unreinforced masonry does not have good seismic performance as it is a heavy, brittle material with low tensile strength and exhibits little ductility when subjected to seismic effects. It is therefore unsuited for areas of high seismicity. However in regions of lower seismic activity such as Australia unreinforced masonry can be used in most instances provided it is designed and detailed correctly and built to the required standard.

This paper has given an overview of the use of unreinforced masonry in Australia and in particular the impact of the new seismic loading provisions of AS 1170.4. Despite the restrictions imposed by the provisions, correctly designed and constructed unreinforced masonry can still be used in most applications. There is, however, a need for research into the performance of unreinforced masonry systems under dynamic loading, particularly with regard to wall-floor connections, membranes and flashings, and tying of veneer walls. It is also important that the structural engineer be involved in both the design and supervision of all aspects of the masonry construction, even if the masonry is considered to be non-structural. This is not the current practice, with the masonry usually being considered as an architectural rather than a structural material, and the responsibility for detailing and supervision resting with the architect and/or builder.

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