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Feasibility of rammed earth constructions for seismic loads in Australia

Xiang Dong[1], Michael Griffith[2], Veronica Soebarto[3]

Abstract Out-of-plane bending tests were conducted to determine whether rammed earth walls, designed to satisfy the thermal performance requirements specified by the Building Code of Australia (BCA), will satisfy the seismic loading requirements. A 2.4m tall by 1.2m wide full-scale insulated rammed earth wall comprised of two 175mm thick leaves separated by a 50mm thick layer of insulation, was tested and the results were compared to that of a solid 300mm thick rammed earth wall. Both walls remained stable after cracking up to displacement of 50mm (over 20% of wall thickness). The acceleration necessary to generate the initial forces to cause cracking was over 0.77g, well in excess of the maximum design accelerations for face-loaded masonry walls in Australia. Furthermore, it was found that the flexural strength of the insulated cavity rammed earth wall was simply the sum of the flexural strengths of the two leaves, and that both walls after reaching their peak strength and cracking at mid-height responded as two rigid rocking blocks.

Keywords: Rammed earth; Insulation; Cavity; Flexural strength; Seismic loads

1. Introduction

Rammed earth (RE) is perceived to be an environmental friendly and sustainable construction material as it has an extremely low embodied energy, especially when the raw material is locally available (Morel, et al., 2001; Reddy and Kumar, 2010; Treloar, et al.,

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2001). In addition, the large thermal mass characteristic of its typically thick walls enables RE construction to perform in thermally desirable ways since the large thermal mass reduces fluctuations in the interior temperature by providing a long time lag, known as the “thermal flywheel effect” (Baggs and Mortensen, 2006). However, in regions where the summer and/or winter climates are extreme, RE construction is likely to perform poorly as its low thermal resistance (R-value) does not effectively prevent heat transfer. For this reason, typical RE construction does not comply with the Deemed-to-Satisfy provisions of the Building Code of Australia in the National Construction Code (NCC) (Australian Building Codes Board, 2013).

The Deemed-to-Satisfy provisions require that for Class 1 buildings (detached residential) the minimum required R-value for external walls is $2.8m^2K/W$ for all climate zones in Australia except the Alpine zone, where the minimum requirement is even higher at $3.8m^2K/W$. In previous studies (Hall and Allinson, 2009; Taylor and Luther, 2004; Walker and Standards Australia, 2002; Yan, et al., 2005), it has been shown that a typical 300mm thick RE wall has an effective R-value of only 0.24 - $0.70m^2K/W$, which is much lower than the minimum Deemed-to-Satisfy requirement of $2.8m^2K/W$. Fortunately, the NCC allows for an alternative way to meet the Deemed-to-Satisfy provisions. This alternative pathway stipulates that for external walls with a surface density greater than $220kg/m^2$ wall insulation, an addition of thermal insulation with an R-value of 0.5 to $1.0m^2K/W$ will satisfy the requirement. Since a 300mm thick RE wall has a surface density of between 540 and $660kg/m^2$ (Hall and Djerbib, 2004), insulation (with an R-value of 0.5 to $1.0m^2K/W$) can be added to RE walls to satisfy this alternative requirement.

For aesthetic reasons, it is usually undesirable to install insulation on either surface of the RE walls; thus the solution would be to install the insulation layer in the middle of two earthen leaves, forming an insulated cavity rammed earth (ICRE) wall system (Hall and Swaney, 2005). This wall system (Figure 1), which can be configured to meet the R-value requirements of the NCC, is well-accepted and gaining popularity (Hall and Swaney, 2005); however, the question of how such walls will perform when subjected to out-of-plane loading is yet to be determined.

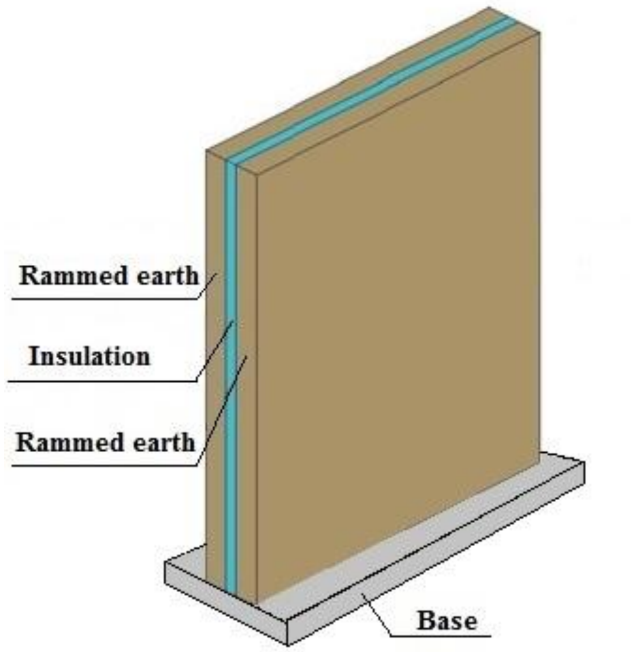


Figure 1 Insulated cavity rammed earth (ICRE) wall

2. Seismic resistance of RE walls

When subjected to out-of-plane 'face' loading, such as that due to wind or earthquake, the resistance of an unreinforced RE wall is highly dependent on its flexural tensile strength f_t (Walker and Standards Australia, 2002). Traditional unreinforced RE walls usually perform poorly during seismic events because of their low flexural strength (Yamin, et al., 2004; Zhou, et al., 2010) which, according to Yamin et al. (2004), can be as low as 0.013MPa. Fortunately, modern RE material is usually stabilised with cement for greater strength. According to published experimental studies (Bahar, et al., 2004; Jayasinghe and Mallawaarachchi, 2009; Reddy, et al., 2007; Reddy and Gupta, 2005), the flexural strength of cement stabilised RE material (also known as soil-cement block or cement stabilised soil in these studies) with cement content of 8-10% was between 0.46MPa-1.05MPa. Such improved flexural strength enables RE wall houses to have better seismic performance. This was shown most recently by surveys of fourteen RE houses in Christchurch, New Zealand after earthquakes in September 2010 and February 2011 (five RE houses in the first survey after the September 2010 earthquake and nine RE houses in the second survey after the February 2011 earthquake) (Morris, et al., 2011; Morris, et al., 2010). During the first earthquake (with a magnitude of 7.1), most of the stabilised RE wall houses performed well, with only slight or moderate damages observed (minor damage occurred to non-structural elements or non-threatening damage to structural elements). These RE houses had a wall thickness of between 200mm and 500mm (corresponding to a wall slenderness ratio of 4.7-12.0) and experienced earthquake shaking with peak ground acceleration (PGA) of 0.15-0.80g. Only one RE wall house, with 500mm thick RE walls, was severely damaged with complete wall collapse, probably because the RE walls were subjected to strong shocking (with a PGA of 0.80g). After the later (February 2011) earthquake (with a magnitude of 6.3), nine unreinforced cement-stabilised RE wall houses investigated in this survey (with wall thicknesses of 150mm to 250mm) suffered only minor cracking.

Although during the survey, the cement content and the flexural strength of the RE materials were not available, the seismic performance of these houses during these two earthquakes implied that unreinforced cement stabilised RE walls have adequate flexural strength to resist moderate seismic loads, as long as they are constructed following the instructions given in three New Zealand standards relating to earth buildings, such as using concrete/reinforced concrete floors and timber/reinforced concrete bond beams (Standards New Zealand, 1998a; Standards New Zealand, 1998b; Standards New Zealand, 1998c).

Installing a layer of insulation in the middle of a RE walls, however, will clearly reduce its flexural strength which is proportional to the square of its wall thickness (Walker and Standards Australia, 2002). It has been shown that the total flexural strength of cavity walls can be predicted as the sum of the strength capacity of the two leaves as long as the wall ties have adequate strength and stiffness to transfer the lateral loads between the two leaves(Brown and Elling, 1979; Page, et al., 1996; West, et al., 1982); however, these studies focused on brick/block cavity walls and brick veneer walls. No such study has been conducted on the flexural strength of cavity RE walls, although the structural properties of RE are similar to those of brick/block masonry (Jaquin, et al., 2009; Jayasinghe and Mallawaarachchi, 2009). Hence, tests were conducted to investigate the flexural behaviour of unreinforced ICRE walls in order to determine whether this wall system has the potential to resist the typical seismic load for Australia.

In this pilot study, a full-scale unreinforced ICRE wall was simply-supported at its top and bottom edges and subjected to out-of-plane loads to induce vertical bending to evaluate its flexural behaviour. For comparison, a solid full-scale RE wall was also tested under similar support and loading conditions. In addition, small beam specimens were cut from the failed walls and tested under four-point bending in order to determine whether the material strength derived from the small beam specimens was consistent with the flexural tensile strength implied by the tests of the full-scale walls. Finally, a parametric study was conducted to identify the range of typical design parameters for RE walls that are likely to comply with the seismic design requirements in Australian Standard AS1170.4 (Structural Design Actions Part 4: Earthquake actions in Australia).

3 Materials and casting of specimens

In order to replicate as realistically as possible actual RE construction in Australia, a qualified builder with much experience in the construction of RE wall homes built the walls used for the tests reported here. The earthen material used in this study was collected at the Fitzgerald Quarry, in Yankalilla, South Australia, which is typically used by local RE builders. The grading curve for the raw soil is shown in Figure 1 (Gepp, 2009). Approximately 10% (by volume) of cement was used as a stabiliser. The material for the wall was mixed using a front end loader as shown in Figure 2. Water was added manually by an experienced mason using a hose. A simple, but efficient, "drop test" (Easton, 2007) was used to determine when enough water had been added to achieve the right consistency (by dropping a ball of the material from chest height onto a firm surface to determine from its behaviour whether it was too dry, too wet or ready for use).

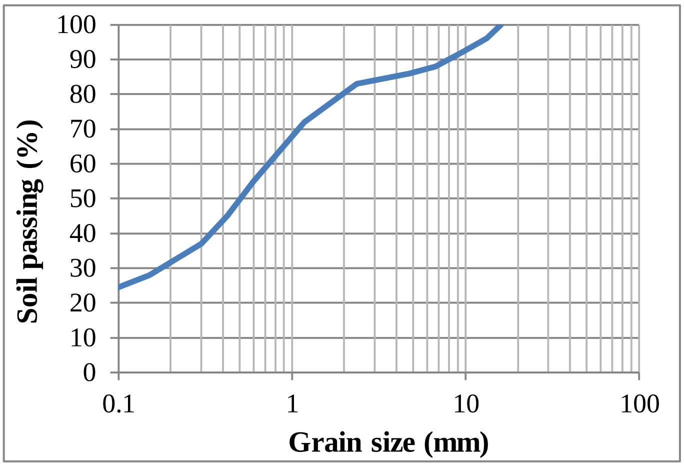


Figure 1 Grading curve for the raw soil

Figure 2 Mixing process of cement-earth mixture

3.1 Casting of full-scale solid RE wall

Once mixed, the cement stabilised earthen material was placed and compacted in a 1200mm x 600mm x 300mm formwork as shown in Figure 3. The RE material was placed into the formwork in four 200mm “pours”. After each “pour”, a hydraulic hammer was used to compact the layer to a depth of 150mm, creating construction joints every 150mm up the wall. This process was repeated three times until the construction of one 600mm high lift or earthen panel was completed. Three more 600mm panels were built using the same process until the wall’s 2.4m target height was reached, as shown in Figure 4. The formwork for the full-scale solid wall was removed 24 hours after casting, and then the wall was allowed to dry in the laboratory for approximately seven weeks before testing.

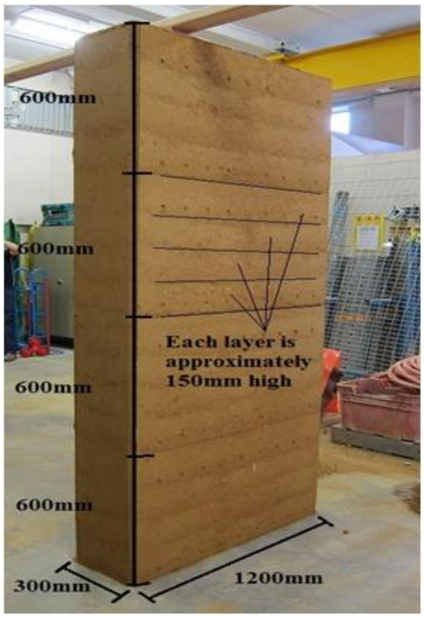


Figure 3 Construction process of RE wall **Figure 4** Full-scale RE wall

After the full-scale solid wall was tested for flexural strength, four small beams were cut from the edges of the top half of the wall as shown in Figure 5 in order to investigate the material properties of the test wall at a manageable scale. The dimensions of the "cut" beam specimens (CBS) taken from the full-scale solid wall are shown in Table 1. After each test, parts of the failed specimen were collected and dried in an oven at 105^{o}C for at least 24 hours to determine their moisture content (MC) at the time of testing.

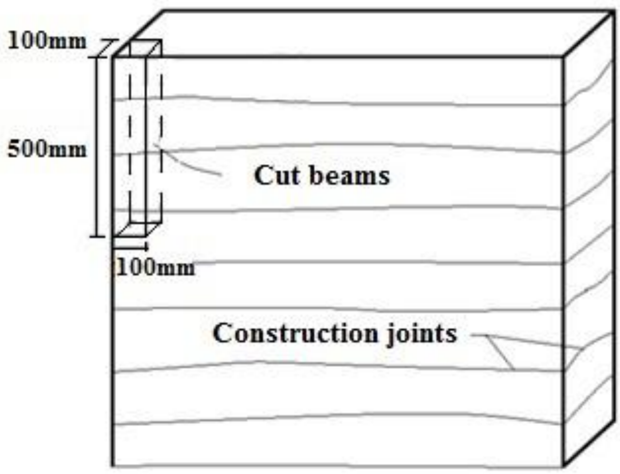
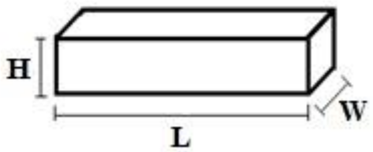


Figure 5 Orientation of cut beam specimens from full-scale walls

Table 1 Dimensions of beam specimens cut from solid wall

Specimens	Dimensions (mm): *L×W×H	
CBS1		499×99×101
CBS2		501×97×100
CBS3		498×102×101
CBS4		499×100×101

*L, W and H stand for length, width and height, respectively.

3.2 Casting of full-scale ICRE wall

After the solid RE wall was built, another batch of material was prepared for the ICRE wall, which consisted of two 175mm thick RE wall leaves – sandwiched around a 50mm thick sheet of polystyrene insulation. The process of constructing the ICRE walls was similar to that of constructing the solid RE wall. The first lift of formwork was setup to create a panel 1200mm long, 600mm tall and 400mm wide, after which three steel wall ties (8mm in diameter, 200mm long with 30mm long returns) were put at the bottom and a piece of polystyrene insulation board (1200mm x 600mm x 50mm) was installed in the middle of the formwork as shown in Figure 6.

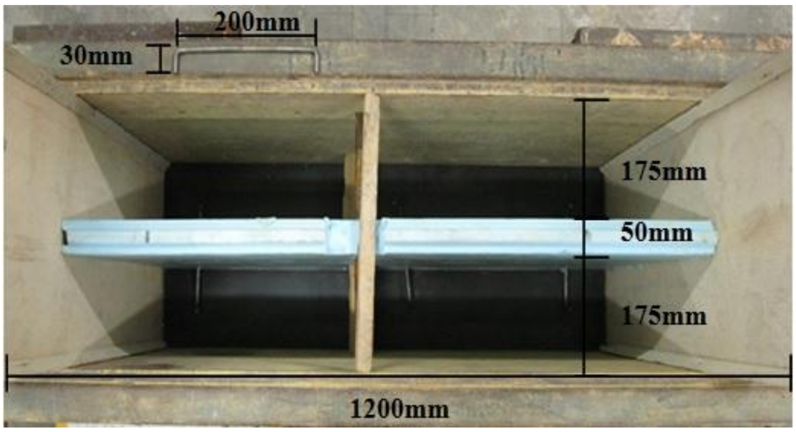


Figure 6 Form work for constructing ICRE wall

As was done for the solid earth wall, the earthen mixture was poured into the formwork 200mm at a time, with each 200mm layer compacted to approximately 150mm before the next layer was added. Three 600mm high panels were constructed in this way until the test wall stood 1800mm. The final 600mm high panel was constructed without insulation and acted as a sort of bond beam, as recommended by local builders who felt that this added to the structural integrity of the wall system. The dimension of the ICRE wall is shown in Figure 7. The cavity wall was covered by wet cloth to cure for 28 days as suggested by the builder, after which it was allowed to dry for another four weeks before testing. After the wall was tested under one-way vertical bending, four small beams were cut vertically from the failed wall (Figure 5) in order to determine the flexural tensile strength of the RE wall material. The dimensions of these beam specimens are shown in Table 2.

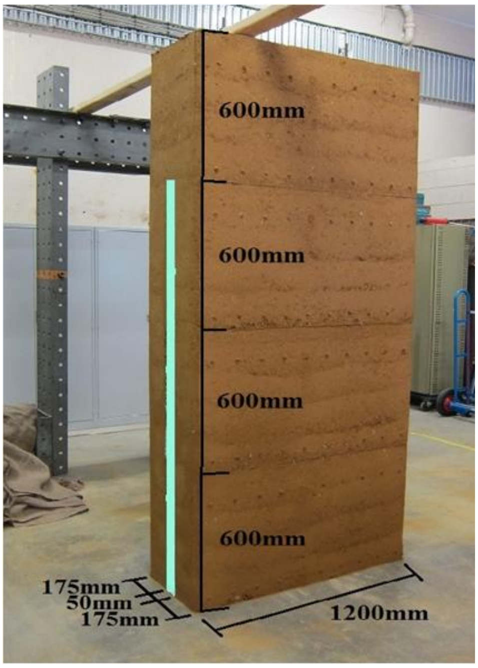
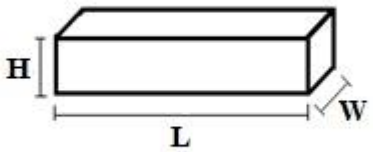


Figure 7 Dimension of ICRE wall

Table 2 Dimensions of cut specimens from insulated cavity wall

Specimens	Dimensions (mm): *L×W×H
CBC1	497×98×101
CBC2	495×101×100
CBC3	488×104×100
CBC4	500×103×101



*L, W and H stand for length, width and height, respectively.

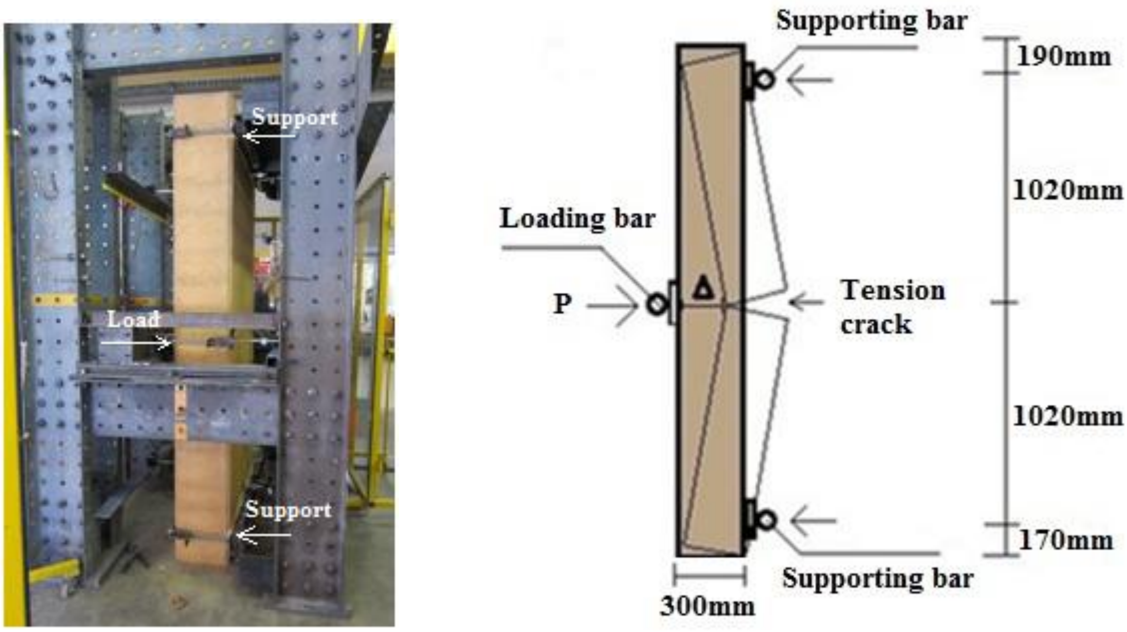
4. Experimental study

The testing program was conducted in two phases. *Phase 1* involved out-of-plane bending tests of the full-scale solid wall to determine its flexural strength, and four-point bending tests of small “cut” beam specimens to determine the material flexural tensile strength. In *Phase 2* the full-scale ICRE wall was tested to determine its flexural strength after which small beams were cut from the cavity wall and tested to evaluate the flexural tensile strength of the cavity wall material.

4.1 Phase 1-Solid RE wall tests

4.1.1 Flexural strength test of full-scale solid wall

The purpose behind testing of the full-scale wall was to study the flexural behaviour of solid RE walls under out-of-plane vertical bending. The test setup is shown in Figure 8(a). The wall was laterally restrained at top and bottom with simple supports. The out-of-plane lateral load was applied at the wall’s mid-height. The loading line and supporting lines were made of cylindrical steel bars. Steel plates (50mm wide and 10mm thick) were pasted onto the wall specimen to minimise stress concentrations at supports and loading points. The load was applied using a hand operated hydraulic jack fixed to a reaction frame.



(a) Test setup (b) Failure mechanism

Figure 8 Test setup and failure mechanism

The lateral load was measured using a 66kN load cell and applied quasi-statically and increased monotonically until failure. As expected, the wall cracked suddenly at its mid-height through tensile flexural failure as shown schematically in Figure 8(b). The lateral load versus mid-wall displacement relationship for the solid wall is shown in Figure 9 where it can be seen that the maximum lateral load was 22.17kN. At failure, the crack extended rapidly through almost the entire wall thickness, with a correspondingly quick drop in the lateral load after which the wall behaved as two rigid blocks rotating about their contact points. Under continued loading the wall's strength continued to reduce slightly after cracking (mid-wall displacement $\Delta = 1.17mm$) with an increasing displacement. The test was stopped when the mid-wall displacement reached $\Delta = 13.60mm$ even though the wall suffered no further damage. From this test it appears that the wall would have continued to remain stable and resist load until the mid-height displacement was equal to its thickness of 300 mm as has been reported in many previous tests on unreinforced brick walls subject to vertical one-way bending (eg, Doherty et al, 2002).

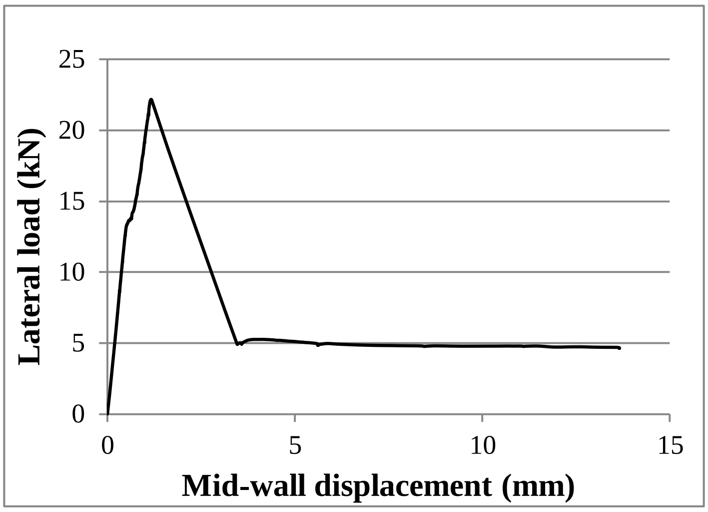


Figure 9 Lateral load and mid-wall displacement relationship (solid wall)

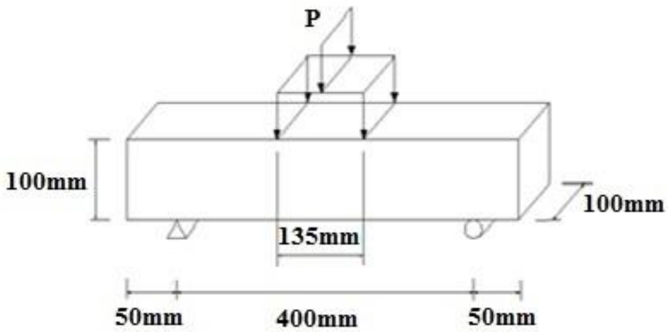
Given the apparent linear elastic behaviour of the RE wall up to failure, the flexural strength of the full-scale wall can be calculated by (Standards Australia, 2011):

$$M_c = (f_t + f_d)Z \qquad \text{Eq. (1)}$$

where f_d = the compressive stress at the mid-height cross-section (in this study, f_d = the stress caused by self-weight of the wall above the mid-height cross-section), Z = the section modulus of the cross-section under consideration ($Z = bt^2/6$, b and t stand for the wall length and thickness), M_c = the moment acting on the wall ($M_c = PH/4$, P and H stand for the applied lateral load corresponding to failure and the height between the two lateral supporting bars). For the solid wall, the wall's width and thickness were measured as 1.20m and 0.30m, respectively. The height between the two lateral supports was 2.04m, and the bulk density of the wall at the time of testing was approximately 2000kg/m^3. Hence, the ultimate lateral load of 22.17kN resisted by the wall implied that the flexural tensile strength of the RE material was 0.60MPa. However, even with such a low tensile strength, the acceleration necessary to exceed that material strength in a 300mm thick solid RE wall is 1.38g which is substantially more than the maximum force specified for design by Section 8 of the AS1170.4 (Standards Australia, 2007) (which corresponds to 0.5g). Furthermore, if such a wall had pre-existing cracks (eg, due to differential settlement) the wall still has its full uncracked displacement capacity and its 'cracked strength' can easily be calculated using $f_t = 0$ in Eq. 1.

4.1.2 Flexural tensile strength tests of "cut" beam specimens

Four-point bending tests were conducted as shown in Figure 10(a) to determine the flexural tensile strength of the beam specimens taken from the solid wall. To make sure that the beam specimens were not damaged by stress concentrations at the supports and loading points, hard wooden boards were used between the test specimens and supporting/loading bars. The load was applied at a constant rate of 0.8kN/min until specimen failure through two loading bars, which provided a 135mm long span of constant bending moment. All of the specimens failed by flexure within the constant moment region as shown in Figure 10(b). The test results are shown in Table 3, where it can be seen that the average flexural tensile strength of the specimens cut from the wall was 0.85MPa with average moisture content of 3.50% and average dry density of 1900kg/m^3.



(a) Bending test setup

(b) Failure pattern of beam specimens

Figure 10 Arrangement of bending test and typical failure pattern of beam specimens

Table 3 Flexural tensile strength test results

Specimen	DD (kg/m^3)	MC (%)	f_t (MPa)
CBS1*	1878.78	3.44	0.40
CBS2	1888.69	3.57	0.66
CBS3	1901.85	3.55	0.99
CBS4	1899.20	3.38	0.90
Mean	1896.58	3.50	0.85
St.Dev.	6.96	0.10	0.17
COV	0	0.03	0.20

Note: the result of specimen CB1 is rejected by rejection criteria.

DD, MC and f_t stand for dry density, moisture content and flexural tensile strength.

4.2 Phase 2-Cavity wall test

4.2.1 Flexural strength test of full-scale ICRE wall

The ICRE wall was tested 10 days after the solid wall was tested using the same test setup (see Figure 8a). During this test, the lateral load was applied to the left hand wall leaf (Figure 11). The right hand wall leaf (Figure 11) cracked first at a peak lateral load of 14.4kN (corresponding to a seismic acceleration of 0.77g) and a mid-wall displacement of 0.8mm. The cracking occurred approximately 130mm above its mid-height (Figure 11), near the first construction joint above the mid-height. The lateral load dropped to 9.8kN once the right hand wall leaf fully cracked, after which the other wall leaf (left hand) cracked at its mid-height. The crack extended rapidly with a drop of lateral load and an increase of mid-wall displacement and stopped when the lateral load decreased to 8.1kN and the mid-wall displacement increased to 2.6mm. Then the wall behaved as two rigid blocks rotating about their contact points and the load further decreased gradually with some fluctuation until the test was stopped with a mid-wall displacement of 51.0mm. The lateral load and mid-wall displacement relationship of the ICRE wall is shown in Figure 12 where it can be seen that the ICRE wall maintained its stability without a significant loss of strength for quite large displacements over 1/3rd of the single leaf thickness. As for the RE wall, the post-cracking strength of the ICRE wall can easily be calculated using Eq 1 with $f_t = 0$.

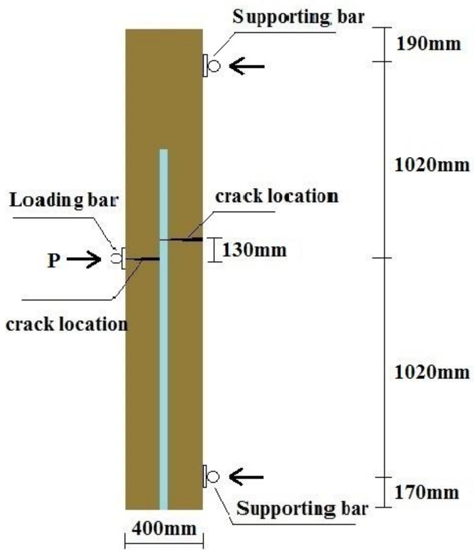


Figure 11 Failure mechanism of the ICRE wall

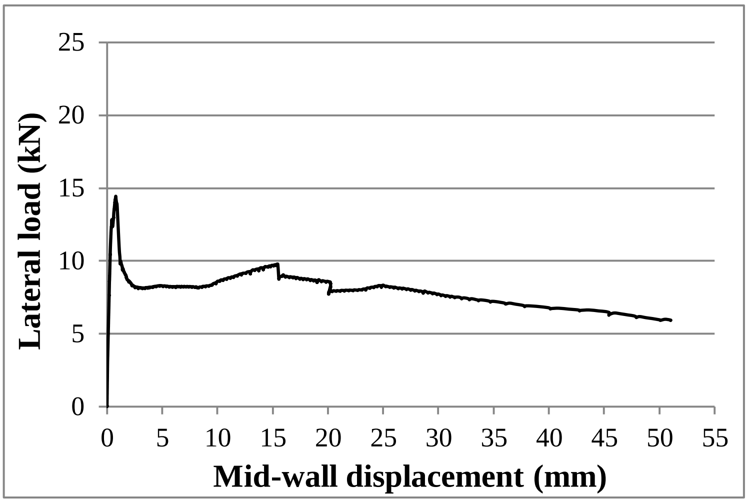


Figure 12 Lateral load and mid-wall displacement relationship (ICRE wall)

Under out-of-plane lateral loads, cavity walls normally display complicated behaviour which is controlled by the stiffness and boundary conditions of each wall leaf, the wall tie stiffness and the wall tie layout (Memari, et al., 2002; Page, et al., 1996; Page, et al., 2007). For the ICRE wall tested in this study, the right hand wall leaf (Figure 11) was supported at top and bottom. The left hand wall leaf could also be considered to be supported at the top and bottom because the top of the wall system was made by solid RE, and at the bottom the cavity was filled with solid insulation board, meaning that the lateral displacement at the top and bottom of the left hand wall leaf was restrained. Furthermore, the lateral load can be effectively transferred between two wall leaves as the cavity was completely filled with insulation board and connected by steel wall ties (8mm in diameter). During the test, the relative displacement between two wall leaves was monitored by two LVDTs on both sides of the wall as shown in Figure 13. The relative displacement between the two RE wall leaves was observed to be less than 0.1mm as shown in Figure 14.

Figure 13 LVDT for recording the relative displacement of two RE wall leaves

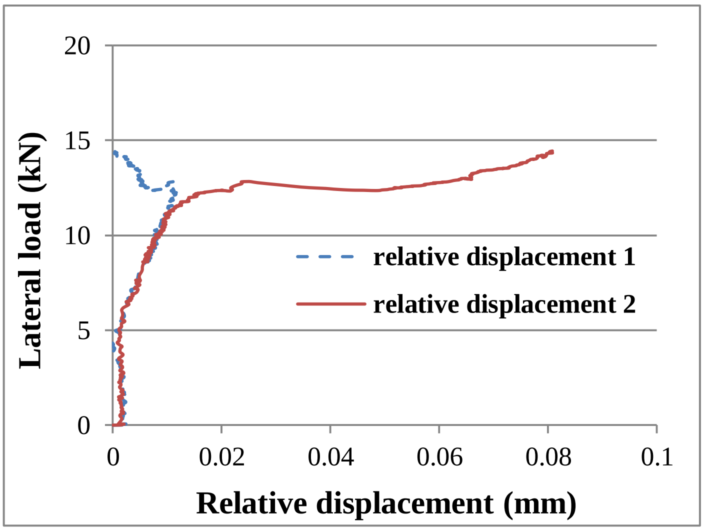


Figure 14 Load versus relative displacement relationship

Given that the absolute value of relative displacement between the two RE wall leaves was very small compared with the wall displacement, it seems reasonable to assume that the two wall leaves had the same lateral displacement under vertical bending and shared the applied load more or less equally. This was further supported by the observation that before the peak load was attained, the left hand wall had not lifted off the floor. This suggested that composite action between the two leaves did not occur (otherwise the whole left hand wall leaf would be lifted up and the right side of the right hand leaf would be the only point connecting to the floor). Therefore, it was assumed that the lateral load to which the wall was subjected was resisted evenly by the two wall leaves (as their flexural rigidities were the same). The bulk density of the RE wall leaves at the time of testing was approximately 2000kg/m^3. Hence the flexural strength of the ICRE wall material was calculated to be 0.58MPa (using the equations presented in subsection 4.1.2).

4.2.2 Tests for small beams cut from the ICRE wall

The flexural tensile strength of the four beams cut from the ICRE wall was evaluated using the same method as that used for testing beams cut from solid wall. The test results are shown in Table 4 where it can be seen that the cut beam specimens CB1-CB4 had a mean flexural tensile strength of 1.00MPa (compared to 0.58MPa for the full-scale wall), with an average dry density of 1870 kg/m^3 and an average moisture content of 4.05%.

Table 4 Flexural tensile strength test results

Specimen	DD (kg/m^3)	MC (%)	f_t (MPa)
CB1	1863.04	4.25	0.74
CB2	1857.80	4.23	1.22
CB3	1908.21	3.60	1.15
CB4	1854.79	4.12	0.88

Mean	1870.96	4.05	1.00
St.Dev.	25.06	0.31	0.23
CoV	0.01	0.08	0.23

It is clear that the cut beams from both walls over predicted the full-scale wall strength. For the solid wall, the full-scale wall strength (0.60MPa) was 70% of the average strength implied by cut beam tests (0.85MPa). For the ICRE wall, the full-scale wall strength (0.58MPa) was only 58% of the mean strength of the cut beams (1.00MPa). It should be noted that the wall thickness was not consistent at 175mm for each wall leaf of the ICRE wall. At the point where the left hand and right hand walls cracked, the thicknesses were 166mm and 170mm, respectively, as shown in Figure 15. If this is considered, the full-scale wall material strength can be adjusted to 0.63MPa, which is 63% of the average "cut" beam's strength and more consistent with the results for the solid RE wall. The reason for the difference in flexural tensile strength obtained from the full-scale wall tests and the smaller cut beam tests is not fully understood. It may be that the construction joints in the wall (spaced approximately 150mm) were not located within the central 135mm span of the small cut beam specimens. This is an issue which requires further investigation.

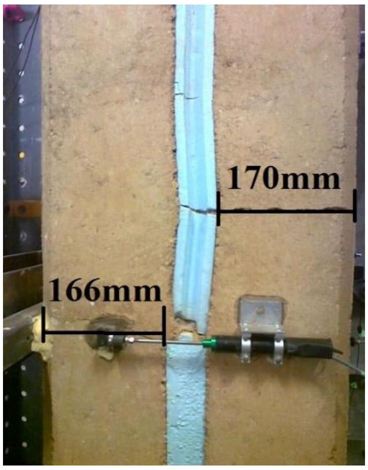


Figure 16 Thickness of cracked cross-section

5. Recommendations for structural designs of RE walls

To date, there is no specific standard for structural design of RE houses in Australia. The only guidance for earthen buildings, the Australian Earth Building Handbook (Walker and Standards Australia, 2002), provides state-of-the-art guidelines for structural designs primarily for one or two-storey earthen houses. Also, AS3700 (Standards Australia, 2011) can be used as a reference document for structural designs of RE as RE is normally considered as a masonry material with similar structural properties to those of block/brick masonry (Jaquin, et al., 2009; Jayasinghe and Mallawaarachchi, 2009). The handbook states that if designs of flexural capacity under out-of-plane loading are required, the loading should be resisted only by the vertical bending capacity of RE walls. The design bending moment should not be greater than the design bending capacity of the wall. Australian Earth Building Handbook (Walker and Standards Australia, 2002) and Australian standard AS3700 (Standards Australia, 2011) provide method to predict the design moment capacity of walls under vertical bending considering a capacity reduction factor for structural design of masonry members:

$$M_{cv} = (\O f_t' + f_d) \times Z \qquad \text{Eq. (2)}$$

where $\O = 0.6$ is the capacity reduction factor for unreinforced masonry members subjected to actions other than compression (Standards Australia, 2011) and f_t' is the characteristic flexural tensile strength of RE.

Assuming that the compressive stress subjected to the top of RE walls caused by ceilings/roofs is negligible, the compressive stress at the mid-height cross-section of a wall is contributed solely by wall self-weight above the cross-section, meaning that $f_d = \gamma H/2$, where: γ is the specific weight of rammed earth. According to the tests conducted in this research, a value for γ can be assumed to be $19kN/m^3$.

Under seismic acceleration, the demand moment for RE walls can be expressed as (Standards Australia, 2007):

$$M_{dv} = wH^2/8 = a_d\gamma LtH^2/8 \qquad \text{Eq. (3)}$$

where: a_d= the corresponding acceleration at the centre of mass,

$a_d = [k_pZC_h(0)]a_x[I_ca_c/R_c]$, k_p= the probability factor (1), Z = hazard factor, $C_h(0)$ = spectral shape factor for a building with a period of zero, a_x = the height amplification factor, $a_x = (1 + k_ch_x)$, k_c= 0.17 for structure height less than 12m, h_x= height at which the component is attached above the structure base of the structure. The product of $k_p Z C_h(0)$ is in effect the peak ground acceleration for the soil site class. For ground storey walls that are supported at their top and bottom edges, h_x is taken as the wall's mid-height. $I_{c,}a_c, R_c$ = the component importance factor (1), component amplification factor (1) and component ductility factor (1), respectively.

By combining equations 2 and 3, it can be derived that:

$$M_{dv}/M_{cv} = 57ZC_h(0)(1 + 0.085H)H^2/4(0.6f_t + 9.5H)t \qquad \text{Eq. (4)}$$

A wall will fail when the demand moment exceeds the moment capacity, meaning that the ratio of moment demand to moment capacity should not exceed 1. According to equation 4, this ratio is determined by wall height, wall thickness and flexural strength of the wall, as well as the hazard factor and spectral shape factor. According to the experimental studies performed in this study, the characteristic flexural strength f_t' of different specimens is summarised in Table 5.

Table 5 Characteristic flexural strength of RE

Specimen type	f_t (MPa)	St.Dev. (MPa)	$f_t^{'}$ (MPa)
CBS	0.85	0.17	0.57
CBC	1.00	0.23	0.62

A parametric study has been conducted considering key factors that will affect the ratio of moment demand to moment capacity. The characteristic flexural strength of "cut" beams from solid RE and ICRE wall was 0.57MPa and 0.62MPa, respectively. Hence the ratio of characteristic flexural tensile strength to the mean flexural tensile strength for the tested specimens was 0.671 and 0.620 for solid wall material and cavity wall material, respectively, giving a mean value of 0.645. The mean flexural strength of the two full-scale walls was 0.59MPa; hence the characteristic flexural strength of full-scale RE walls can be calculated to be $0.59 \times 0.645 = 0.38$MPa.

The earthquake hazard factor Z-value in Australia is between 0.03-0.22, while for most of the areas, the Z-value is between 0.05-0.15 (Standards Australia, 2007). In this parametric study, three Z-values were considered, namely 0.05, 0.10 and 0.15. The site sub-soil can be classified into four classes namely strong rock (A_e), rock (B_e), deep or soft soil/very soft soil (D_e/E_e), and shallow soil (C_e), corresponding to a spectral shape factor $C_h(0)$ of 0.8, 1.0, 1.1 and 1.3, respectively (Standards Australia, 2007).

The relationship between the ratio of demand moment to moment capacity and the wall thickness for a typical 3m tall RE wall (the maximum wall height required in Australian Earth Building Handbook is 3m) considering the largest spectral shape factor $C_h(0) = 1.3$ and different hazard factors in Australia are calculated as shown in Figure 17:

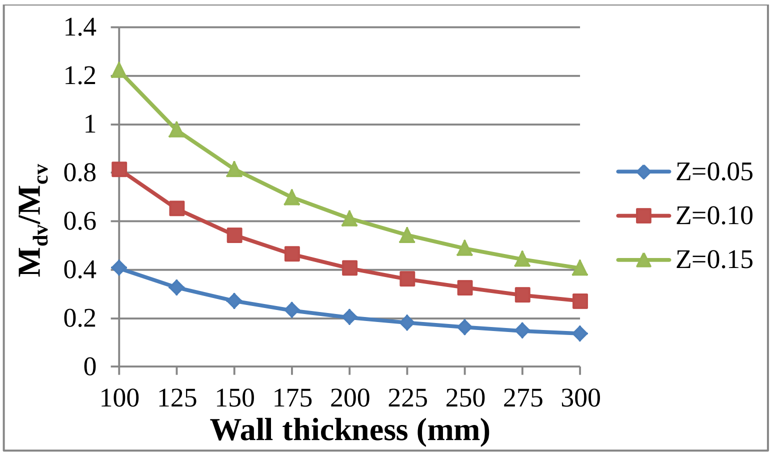


Figure 17 Relationship between M_{dv}/M_{cv} and wall thickness

It can be seen from Figure 17 that 3m tall RE walls with a wall thickness as thin as 125mm can be safely applied in Australia (except for some areas in Meckering, Western Australia where the hazard factor is greater than 0.15). In areas with a harzard factor no greater than 0.10, the thickness of RE walls can be further reduced to 100mm. It should be noted that in order to obtain a surface density greater than 220kg/m^2, the minimum RE wall thickness should be 116mm (assuming that the RE wall has a density of 1900kg/m^3). However, as the minimum required wall thickness in the Australian Earth Building Handbook is 200mm (Walker and Standards Australia, 2002), which is considerably conservative, it can be concluded (refer Figure 17) that any RE wall of 200mm thickness or greater will have the flexural capacity to withstand the seismic loads specified in AS1170.4 for any city in Australia.

For cavity walls, there is no requirement for the minimum wall leaf thickness in the Australian Earth Building Handbook (Walker and Standards Australia, 2002), while AS3700 (Standards Australia, 2011) requires a minimum thickness of 100mm for each wall leaf. In addition, it is requied by AS3700 that for strength capacity design of cavity walls under out-of-plane bending, each wall leaf should be assessed seperately. Therefore, for ICRE walls (no taller than 3m), each RE wall leaf can be built as thin as 100mm in regions where the harzard factor is no greater than 0.10. For regions with a harzard factor of 0.15, the thickness of 3m tall RE walls should be at least 125mm, or 100mm thick RE walls can be used if the wall is no taller than 2.7m.

6. Conclusions and recommendations

In spite of the small sample size for each test, several conclusions and recommendations can be drawn from the test results:

1. The wall ties and the 50mm thick stiff insulation layer used in the ICRE wall specimen tested in this study can effectively transfer lateral load between two leaves and the two leaves (with the same flexural rigidity) tend to have similar deflection under vertical bending. The flexural strength capacity of such ICRE walls can be calculated as the sum of the flexural strength of the two leaves.
2. The RE and ICRE walls both remained stable for displacements well in excess of their cracking displacements. They rocked as two rigid bodies about their base and mid-height crack locations. This response is similar to that reported in previous research on clay brick masonry where the displacement capacity of rocking rigid bodies has been recommended to be a reliable wall response mechanism.
3. Even though both walls were shown to have sufficient uncracked strengths to resist the 500 YRP earthquake loads specified for any site anywhere in Australia, taller walls or more severe (> 500 YRP) earthquake loads could induce forces that would exceed the flexural strength of these walls. However, these tests suggest that they have the rocking displacement capacity to safely withstand much larger ground shaking.
4. The requirement of minimum wall thickness (200mm) in the Australian Earth Building Handbook is too conservative for single storey houses in Australia. Cavity RE walls with

125mm thick RE wall leaves can resist the most severe seismic loading in Australia as long as they are less than 3m tall.

Acknowledgements

The writers would like to sincerely acknowledge the assistance from David Robert from Stabilized Earth (Adelaide) Pty Ltd. for providing the specimens of material tests and the full-scale rammed earth walls, and the lab staff from School of Civil, Environmental & Mining Engineering at the University of Adelaide for conducting the experimental program.

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Response to Reviewers:

Reviewer #1: This paper is well written and has presented some interesting, and useful, findings which are related to the robustness of constructions built of rammed earth fulfilling insulation requirements.
Whilst the paper is accepted the reviewer would like the authors to consider inserting a couple of paragraphs to address the following issues:

1. The capacity assessment of the RE wall as presented in the paper is based on a stress based approach which relies on the assumption of a characteristic tensile strength of the materials. There may well be situations where the tensile strength of the wall has been compromised by the effects of differential settlement, or the like. In that case, should we still be using Eq.(1) but letting ft = 0 MPa. Alternatively, should rigid body mechanics be used instead to quantify the horizontal strength capacity of the wall ?
This has been addressed by adding text to highlight how to handle pre-cracked RE walls (using ft = 0) as well as noting the significant post-cracked strength of the two walls and of course their substantial displacement capacities (see pages 12, 14, 16 and conclusions on pg 25.).

2. It is noted that recommendations made in the paper was based on seismic actions consistent with a return period of 500 years across the whole of Australia. Apparently, the wall was deemed safe by the authors when the bending moment action associated with this level of ground shaking was found to be exceeded by the calculated bending moment capacity. However, one may criticize this approach for not taking into account the ultimate performance behaviour of the wall should the intensity of shaking happens to be higher (than that predicted by Equation 3 and a Z value for RP of 500 years) in a rarer earthquake event. Reservation with the approach seems to be justified in view of the (apparent) brittle nature with this form of construction. Would these walls experience abrupt wholesale collapse ? or undergo rocking motion? The second author is known to have published papers to address this issue in the context of unreinforced masonry walls. However, none of his paper has been cited in the manuscript.

This has been noted in the discussion of the wall's rocking response where we note that they both displayed significant displacement capacity beyond cracking and that much larger displacement demands than those for the 500 YRP earthquake could be required to cause ultimate collapse. This is also noted in the conclusions. (refer pages 12, 16, 18 and conclusions on pg 25.)

Apart from these two issues the reviewer is happy with the submission.

Reviewer #2: Very good paper, well researched and structured, topical and recommended for publication.

A few minor comments for attention:

1. The approach presented for checking the earthquake performance of ICRE walls appears based on a force based approach, where failure is defined when the wall cracks. This approach is considered conservative and assumes adequate support at the edges. A comment on the post cracking behaviour using displacement based approaches is considered worthy for completeness.

This has been added on pages 12, 14, 16 and in conclusions on pg 25.

2. The AS3700 reference is not using the latest edition - needs correcting

Done – see pages 13, 21, 24 & reference list.

3. The RE density is quoted as 1896.58 kg/m^3 - recommend that this is rounded off to 1897 or preferrable 1900

Agreed – mean values given to 3 sig figures on pages 14 and 19.

INSTRUCTIONS AND REQUIREMENTS FOR SUBMITTING REVISIONS

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