SHAKING TABLE TEST OF FULL-SCALE BRICK VENEER HOUSE

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SUMMARY

This paper discusses the testing of a single room brick veneer house subjected to simulated ground vibrations from blasting on a shaking table. The focus of this work is on the cosmetic in-plane performance of unreinforced masonry veneer. The specimen is subjected to gradually increasing vibration intensities and the resultant crack patterns are observed. The level of vibration necessary to achieve a temporary and permanent change in induced cracks is investigated. Results highlight limitations of existing velocity based limits and indicate displacement based limits to be more appropriate.

INTRODUCTION

Certain residential areas in Australia are situated near mines or quarries that adopt blasting practices to assist in the removal of rock. Due to this activity low level vibrations may be experienced in residential areas at considerable distances from the origin of the blast. Despite the low severity of vibrations transmitted to houses, residents frequently become concerned about the potential of the vibrations to damage their homes. A pilot study which monitored three houses subjected to ground vibrations found the response to ordinary environmental loads was at least equal to the response caused by blasting (Gad et al, 2005). While recent amendments to Australian vibration standards suggest environmental limits are conservative with respect to damage, there remains a need to identify damage thresholds particularly for non-structural components of residential structures.

A comprehensive testing program has been carried out to identify the relationship between level of blast vibration, drift and damage thresholds (Heath et al). The final stage of this study has been the testing of a full-scale single room brick veneer house. Ground vibrations were simulated as uniaxial excitation in two orthogonal directions with gradually increasing severity. Peak table velocity was increased from barely perceptible levels to nearly 80 times the environmental limit commonly adopted by Australian mine operators. The onset and development of damage was carefully monitored from initiation through to complete crack development. This paper reports on the development of damage in the masonry veneer.

LITERATURE REVIEW

Extensive work has been conducted internationally to examine the in-plane performance of masonry structures. However, the majority of research has focused on seismic events, mostly examining the performance under load-bearing conditions, recognising unreinforced masonry veneer to be uncommon for modern construction practice in seismic regions. A variety of different test methods have been discussed by the likes of Macchi (1982), Hendry (1987) and Atkinson (1989) to investigate the characteristics of masonry from a materials level to a systems level. Page (1977) undertook experimental tests on masonry panels while more recently, Vermeltfoort (1993) performed in-plane tests on 12 masonry panels with and without penetrations. Tomazevic (1996a, 1996b) examined the performance of monotonic, cyclic and earthquake simulated lateral loading on masonry panels and modelled the response of the panels to seismic loading. A test rig was developed by Nichols et al (1998) to subject masonry panels to biaxial stress and a harmonic load. This permitted the study of the gradual deterioration and effective stiffness under dynamic in-plane loading (Nichols et al, 2000).

While the above-mentioned authors have investigated the response of single components to dynamic loads, there have also been a number of larger shaking table studies conducted on masonry. Gulkan et al (1990) and Clough et al (1990) studied the performance of four full-scale masonry houses subjected to seismic loading with each wall oriented either parallel or perpendicular to the direction of loading. An additional test on a fifth specimen oriented the structure 30 degrees to the direction of loading to induce components of vibration normal and parallel to the alignment of the walls (Manos et al, 1983). Through comparison with results from prior tests, it was established that biaxial loading did not have a detrimental effect on the in-plane performance of masonry elements. A study by Magenes et al (1994) investigated the performance of masonry walls with different unit and mortar combinations and observed different failure mechanisms arising from varying levels of axial load. Klopp et al (1998) investigated the performance of unreinforced masonry wall panels of varying height to varying levels of precompression and frequency of harmonic loading. Many researchers conducting dynamic tests on masonry report characteristics relating to ultimate strength and the development of hysteresis curves. However, little data is published relating drift to cosmetic damage in non load-bearing unreinforced masonry assemblages with penetrations.

EXPERIMENTAL SETUP

The MTS shaking table used for the experiment is a biaxial table, having two 100kN dynamically rated actuators driven by a hydraulic pump delivering 63l/min. The table has a maximum stroke of 200mm in the horizontal plane in both directions. A PC provides a displacement input to the controller unit and also serves as the data acquisition system. The

table is made of steel being two metres square and 45mm thick. An extension frame is bolted to the table increasing the plan area to 2.57m x 2.77m (refer to Figure 1).

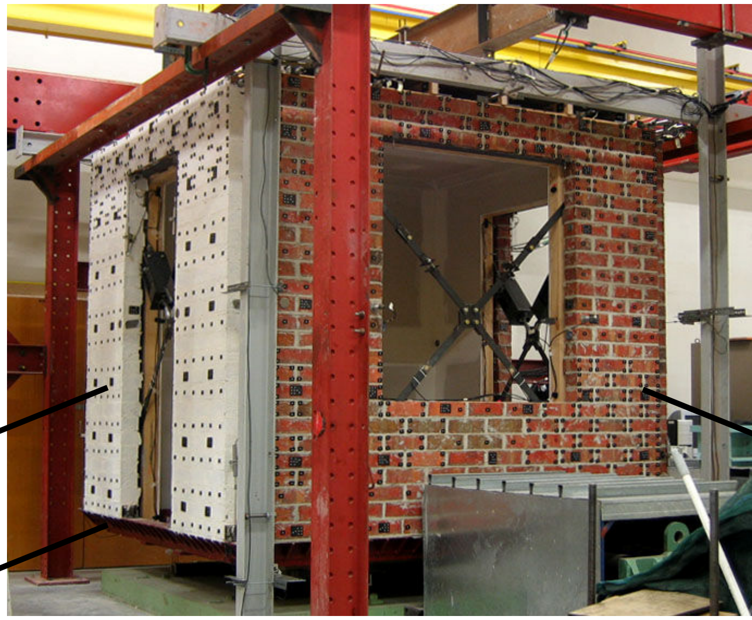


Figure 1. Specimen instrumented and setup for testing in north-south direction.

Specimen Configuration

The timber frame was composed of 90mmx35mm pine studs, noggings and bottom plate with a 90mmx45mm top plate and was purchased from a prefabrication company then bolted to the shaker table. In addition to the standard corner connections stipulated by AS1684-2006, angle brackets bolted orthogonal frames together at eave level. Typical roof trusses were not used for the roof structure and it was assumed the roof system provided a rigid diaphragm to promote identical drift in parallel walls. A 2.3m square reinforced concrete slab 115mm thick (1500kg) was bolted to the north and south frames to simulate a vertical load similar to that experienced by a 8m wide house having a tiled roof. The north and south walls included a 1195mm wide x 1385mm high window while the east and west walls had doorways equal to 900mm wide x 2070mm high making the structure symmetrical (refer to Figure 1). A professional plasterer lined the interior with 10mm plasterboard and a 70mm cornice.

In order to replicate the dynamic properties of a typical house a variable stiffness bracing system was adopted. Each active brace (aligned with direction of excitation) included disc springs which could be stacked such that the overall contribution to structure effective stiffness could vary up to approximately 27kN/mm. This system ensured linear load-deflection behaviour permitting a controlled investigation of the relationship between damage and drift. The bracing was bolted through the bottom plate to the table extension frame and through the top plate and end wall studs.

Since the primary objective of the test was to examine the in-plane performance of the masonry walls, returns were not included between the veneer walls. A professional bricklayer was employed to construct the masonry using a 1:1:6 (C:L:S) by volume mortar mixed to a workable consistency. In total, seven machine mixed batches of mortar were made with the

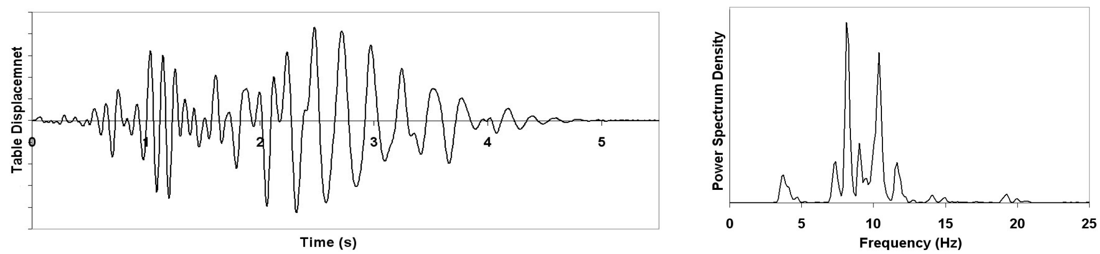
volume of water varying from 2.25 to 1.75 times the volume of cement used in a batch due to variations in the moisture content of the sand. Extruded clay units having dimensions of 230mm (L) x 110mm (W) x 76mm (D) and three 47mm diameter cores were used to construct the walls. General purpose stainless steel veneer ties were spaced at 470mm – 520mm centres in the horizontal and vertical directions in accordance with AS3700-2001. Each tie was nailed to the frame with 40mm nails. A 100mm x 100mm x 6mm equal angle lintel embedded 150mm either side of the openings was used to support three courses of units above the windows and doorways. Each wall was built to a height of 2320mm being one unit short of a height equalling that of the frame to accommodate instrumentation. Stoppers were bolted to the table at the end of each wall and inside each doorway to prevent sliding. For similar reasons a damp proof course was not included.

Instrumentation

The data acquisition system incorporated National Instruments hardware, logging 60 channels including 22 linear voltage displacement transducers (LVDT's), 29 accelerometers, eight strain gauges and a single channel recording table input. Sampling rate was 256 Hz and recorded on a PC running custom data acquisition software written in LabView. Accelerometers were magnetically fixed to small steel discs, ten LVDT's were fixed to a frame surrounding the structure to measure global displacement while the remaining 12 were positioned at the corners of the penetrations to measure relative deformation. Some instrumentation was repositioned when changing direction of shaking (east-west to north-south or vice versa). A secondary data acquisition system containing five triaxial geophones measured the table and eave level response. Permanent deformation induced in the masonry at locations not measured by displacement transducers was measured using a photogrammetry system developed at The University of Melbourne. Nearly 2000 retro-reflective targets were positioned on the exterior of the masonry veneer and photographed from approximately 200 stations once testing at certain vibration levels was complete.

Input Signal

A total of 25 blasts from two mines and 33 blasts from six quarries were analysed as potential traces for the table input. Each trace contained triaxial ground vibration recordings at distances ranging from 100 – 2885m from the blast. Analysis of the horizontal components of these recordings gave a suite of 116 files for comparison. Each signal was normalised to a peak unit velocity then compared by a response spectrum analysis based upon a single-degree-of-freedom system. Consideration of the expected response for a structure having a natural frequency between 5 – 12Hz identified a recording from a mine blast (refer to Figure 2) to be adopted as the signature trace during testing. The signature trace contains two distinct components having different frequency contents reflecting a separation of Rayleigh waves compared to the earlier arrival of the compression and shear waves at the recording location. This trace formed the beginning of a ten second duration recording. During testing the signature trace was scaled by magnitude only to preserve the frequency content (refer to Figure 2). Three tests would be performed at each level of intensity investigated.



(a) (b)

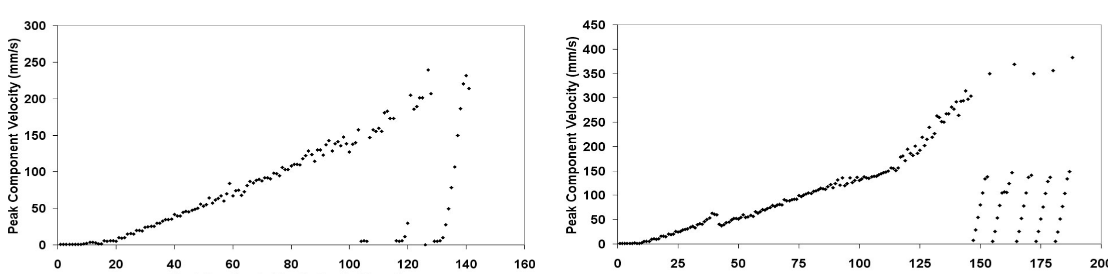
Figure 2. Signature trace used as the primary input to the shaking table; (a) displacement-time history and (b) power spectrum density.

RESULTS

Testing commenced when the masonry was at an age of 188 days and completed at an age of 315 days. During testing cracks did not develop in the plasterboard. However, a number of modifications to the test setup were necessary during testing which are discussed below.

Loading History

The signature trace was run though the table at vibration intensities corresponding to peak component velocity (PCV) multiples of 5mm/s up to an intensity of approximately 150mm/s in both directions followed by 10mm/s increments at levels above 150mm/s. Rotations between the two directions of excitation occurred regularly to reduce the possibility of experiencing extensive damage in one direction only. In total, 12 rotations occurred during testing with a rotation occurring upon achieving an increase in the intensity of approximately 20mm/s. Beyond 130mm/s in the north-south direction a rotation was not performed until testing in the north-south direction was complete. This enabled investigation of the development of cracking from initiation through to complete development in the east and west masonry walls. On two occasions during testing in the north-south direction low level vibrations were run through the table after a new crack formed (refer to Figure 3a). On a third occasion a series of vibrations gradually increasing to 150mm/s were run through the table. These tests enabled investigation of the behaviour of existing cracks to increasing levels of vibration. Upon completion of testing in the north-south direction one final rotation occurred at which point testing in the east-west direction resumed at an intensity of 135mm/s. The intensity of vibration was then increased gradually in the east-west direction until testing was complete. The increase in vibration intensity was only interrupted when new crack(s) developed and a series of 5 – 150mm/s blasts were run through the table (refer to Figure 3b).



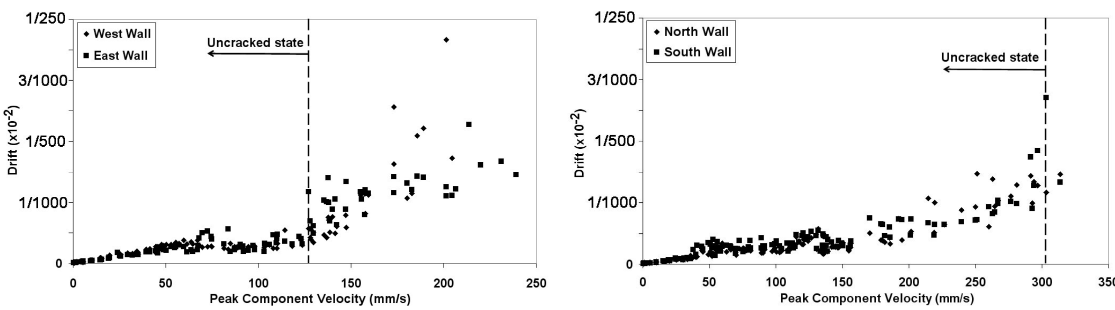
(a) (b)

Figure 3. Peak velocity of table measured in (a) north-south and (b) east-west directions.

In-plane Deformation

A number of modifications were made to the test setup during the test program which influenced the drift-PCV relationship. In the case of the north-south excitation, loosening of ties became prominent in the out-of-plane walls at an intensity equal to approximately 70mm/s (refer to Figure 4a). Some ties pulled away from the frame while others loosened in the mortar. This deterioration was exacerbated as the wall developed a mode of vibration that was independent of the fundamental mode of the frame. A similar experience occurred during east-west excitation at an intensity of approximately 130mm/s. Following this deterioration retro brackets were fixed to the frame and epoxied to the masonry at mid-height, three quarter height and eave level with the restraint being activated when exciting the walls out-of-plane. When a rotation occurred the brackets were disconnected to ensure the racking response was not affected. It may be seen in Figure 4 that the installation of retro brackets on out-of-plane walls reduced peak drift recorded on the masonry walls being excited in-plane. A similar observation was made with the drift of the frame.

During testing in the east-west direction at average table intensities between 40 - 60mm/s the table delivered a higher intensity vibration to the south wall compared to the north wall evident by the apparent amplification of peak drift (refer to Figure 4b). This was attributed to a characteristic of the shaking table rather than specimen behaviour and was not experienced in the north-south direction.



(a) (b)

Figure 4. In-plane drift of masonry walls measured prior to onset of cracking in (a) north-south and (b) east-west directions. Data included in plots is a subset of entire data set to improve clarity for measurements of smaller drifts.

Initial visible cracking in the north-south direction occurred in the east wall at a PCV of 127mm/s corresponding to a drift equal to 1/860. First cracking in the west wall occurred at a PCV of 173mm/s corresponding to a drift of 1/390. The greatest drift measured in the east and west walls prior to onset of cracking 1/750 and 1/870 respectively. For east-west excitation initial cracking occurred in the north and south walls at a PCV of 303mm/s while the greatest prior vibration experienced was 314mm/s generating a drift equal to 1/750 in the south wall and a drift of approximately 1/700 in the north wall. A summary of the initiation of all cracks and corresponding PCV is presented in Table 1.

Crack Patterns

While globally each wall experienced racking, the resultant crack pattern demonstrated that each wall experienced local in-plane flexure. In-plane flexural stresses were particularly

evident in the east and west walls where cracks passed through units (refer to Figure 5). Beyond vibration intensities of 200mm/s the west wall experienced a considerable increase in cracking which subsequently led to dramatic increases in measured drift. The development of cracks WWCA2 and WWCB2 (refer to Figure 5) demonstrate lifting caused by the lintel under excessive racking displacements. The growth of WWCC also occurred due to in-plane flexure under excessive racking with some restraint being provided by the bottom two levels of ties which remained in good condition throughout testing.

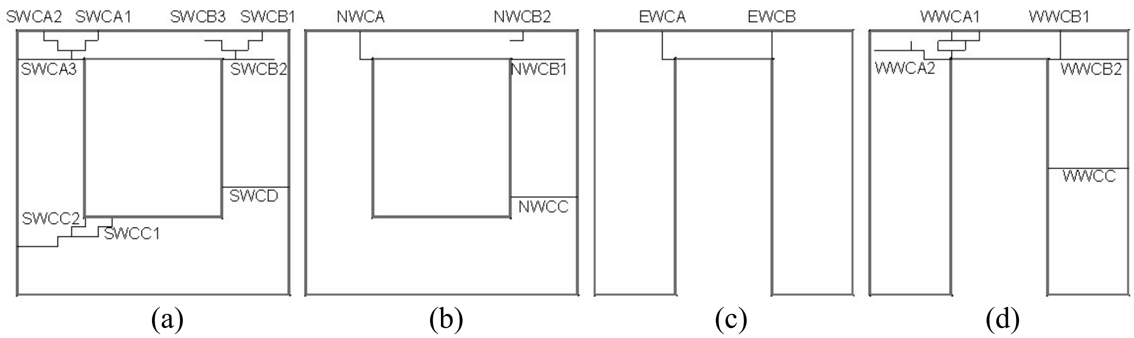


Figure 5. Crack patterns developed at completion of testing in masonry veneer: (a) east wall; (b) west wall; (c) north wall; and (d) south wall. Crack notation includes wall name, crack name and number (e.g. west wall, crack 'A', number 1 becomes WWCA1).

Table 1. Peak table velocity and drift recorded during vibration causing onset of cracks.

Crack	PCV (mm/s)	Peak Drift (mm)	Crack	PCV (mm/s)	Peak Drift (mm)
SWCA1	303	1/370	EWCA	189	1/720
SWCA2	350	1/110	EWCB	127	1/850
SWCA3	350	1/110	WWCA1	173	1/390
SWCB1	303	1/370	WWCA2*		
SWCB2	369	1/80	WWCB1	173	1/390
SWCB3	382	1/170	WWCB2	206	1/150
SWCC1	350	1/110	WWCC	206	1/150
SWCC2	350	1/110			
SWCD	350	1/110			
NWCB1	303	1/1000			
NWCB2	382	1/480			
NWCC	350	1/740			

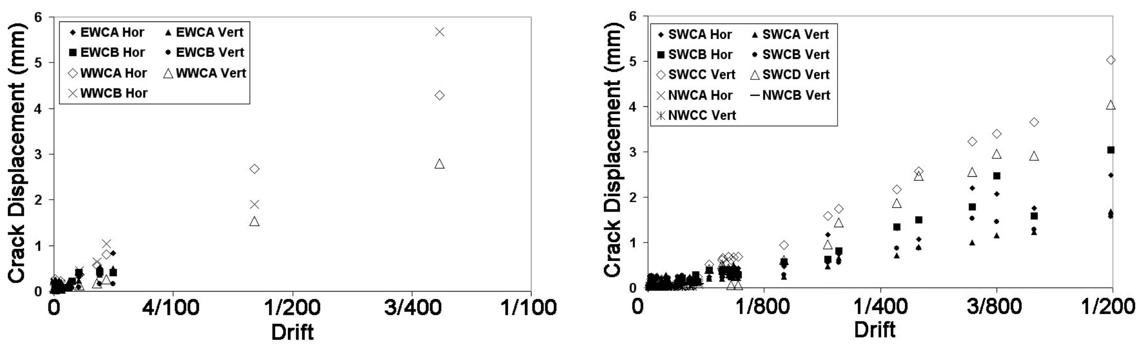
* Data acquisition system did not record file.

The growth of cracks in the south wall also produced stress concentrations at the corner of the penetration brought about by in-plane flexure of the piers on either side of the penetration. The development of crack SWCD three courses above the nearest corner and greatest stress concentration is attributed to variability of workmanship. The north wall experienced comparatively lower drifts than the south wall and consequently experienced less damage. At the conclusion of testing a fully developed mechanism had not developed in the north wall unlike the south wall. Table 1 contains the peak drift recorded during the respective vibration responsible for developing each crack. It is important to note the drift reported here

corresponds to the peak drift recorded during the vibration rather than the drift at crack initiation.

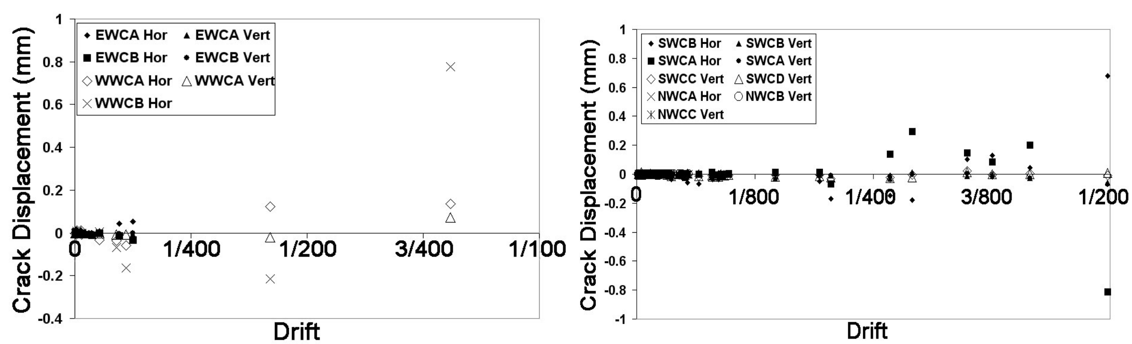
Behaviour of cracks during excitation

After cracks initiated in the masonry from a high intensity vibration a number of smaller blasts were run through the table to investigate temporary and permanent changes to crack width. A number of vibrations less than 25mm/s were run through the table in the north-south direction (refer to Figure 3a) to investigate behaviour at regulatory limits. In addition, a single series of vibrations ranging from 5 – 150mm/s were run through the table close to the completion of testing in the north-south direction. The same series of vibration intensities were run through the table in the east-west direction upon achieving crack development (refer to Figure 3b). Displacement transducers measured the relative horizontal and vertical displacements due to cracking in the top corners of the windows and doors. Additionally, vertical displacement was measured in the bottom corners of the windows. Figure 6 and 7 contain the peak temporary and permanent displacements recorded during this secondary shaking respectively.



(a) (b)

Figure 6. Peak temporary crack widths measured during (a) north-south excitation and (b) east-west excitation.



(a) (b)

Figure 7. Permanent change in crack widths during: (a) north-south excitation; and (b) east-west excitation. Horizontal (Hor) and vertical (Vert) components are measured at the corners of the penetrations.

As expected, the increase in temporary crack width is approximately linear with drift for both directions of excitation. The first detected temporary change in crack width during north-

south shaking was measured to have a peak of 0.46mm and occurred in the west wall when a drift equal to 1/1875 was experienced (PCV=50mm/s). The same vibration also induced the first detected permanent change in crack width, measured as a reduction equal to 0.03mm. A vibration intensity equal to 150mm/s causing a drift of 1/240 in the west wall was the first to cause an increase in crack width (equal to 0.12mm). During east-west shaking the first measured temporary change in crack was 0.02mm and occurred at a drift equal to 1/5200 (PCV = 50mm/s). When a drift equal to 1/675 was reached in the south wall at a vibration intensity of 104mm/s a permanent reduction in width of 0.03mm occurred in SWCB. The first permanent increase in crack width during east-west shaking was equal to 0.13mm and occurred during a vibration causing a peak drift of 1/265.

CONCLUSIONS

A single room house having construction typical of Australian brick veneer houses has been subjected to gradually increasing levels of simulated ground vibration. Based upon a response spectrum analysis of 116 ground vibration signals, a signature trace was adopted for testing purposes that was expected to yield the greatest in-plane response of the test house. At the onset of cracking, similar crack patterns were experienced in the east and west walls. Crack patterns in the north and south walls were similar although cracking in the north wall did not develop to the same degree. The greatest drift measured in the north-south direction prior to the onset of cracking was 1/750 in the east wall and 1/870 in the west wall. The maximum drifts measured for east-west shaking prior to the development of cracking were 1/700 in the north wall and 1/750 in the south wall.

While temporary changes in crack width during excitation were measured at very low drifts, no detection of crack movement was made during vibrations within current environmental limits of 5-10mm/s. The lowest drift responsible for partially closing an induced crack was 1/1875 while the first increase in crack width occurred during a vibration generating a peak drift equal to 1/265. Similar damage states in the two directions were observed for equivalent drifts demonstrating the relevance of displacement based regulatory limits. The 186mm/s discrepancy in vibration level necessary to induce cracking in the two orthogonal directions highlights the limitations of velocity based limits currently employed.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the generous contributions made by the following:
- The project industry partner, Terrock Consulting Engineers
- ARC Linkage Grant No. LP0211407

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