SHEAR CONTROLLED ULTIMATE BEHAVIOR OF NON-DUCTILE REINFORCED CONCRETE COLUMNS

KITTIPOOM RODSIN, NELSON LAM, JOHN WILSON AND HELEN GOLDSWORTHY
UNIVERSITY OF MELBOURNE

AUTHORS :

Kittipoom Rodsin is a post-graduate student at University of Melbourne

Nelson Lam is a Senior Lecturer at University of Melbourne.

John Wilson is an Associate Professor at University of Melbourne.

Helen Goldsworthy is a Senior Lecturer at University of Melbourne.

ABSTRACT:

Reinforced concrete columns supporting buildings with a soft-storey in regions of low and moderate seismicity are typically non-ductile and susceptible to shear failure. Half-scaled reinforced concrete column specimens representative of current construction practices were tested to study their cyclic force-deformation behaviour. Importantly, the gravitational load carrying capacity of the column at ultimate conditions was of particular interests in the investigation. The innovative Vision Methology System (VMS) was used to measure the deformation of the column in 3D. Using the VMS technique, deformation in the column has been resolved into flexural, shear and yield penetration components. Experimental results were presented in this paper and compared with behaviour predicted using various existing models. The discrepancies between the experimental results and the model predictions are highlighted. The development of an accurate deformation models forms part of the displacement-based methodology for the seismic performance assessment of buildings with a soft-storey.

1. INTRODUCTION

Buildings supported by soft-storey columns are well known to be vulnerable to collapse and severe damage under earthquake excitations. The vulnerability of such buildings is aggravated by non-ductile behaviour of the columns and this behaviour is often expected in regions of low to moderate seismicity where detailing is generally poor. These columns, particularly those with a low shear span-depth ratio, are susceptible to brittle failure associated with shear. A recent survey of soft-storey columns around the Melbourne Metropolitan Area (MMA) revealed very low shear span-depth ratios. These columns are susceptible to shear failure during an earthquake. To identify the vulnerability of these soft-storey buildings, the load-deformation behaviour of such columns under cyclic loading was modelled accurately as part of the displacement-based seismic evaluation methodology.

Column deformation is made up of flexural, shear and yield penetration components. Methods that are used to predict deformation due to flexural and yield penetration such as those developed by Watson et al. [1994] and Alsiwat and Saatcioglu [1992], have been shown to be capable of providing realistic estimations. However, the truss analogy method [Park and Paulay, 1975] has been investigated as a part of this project and does not give a good prediction of shear deformation particularly when the behaviour of the column is controlled by shear. The more recently developed model based on Modified Compression Field Theory (MCFT) [Vecchio and Collin, 1986] has been found to be more reliable in providing realistic predictions. Estimates made using this theory have been compared with experimentally determined values in this paper.

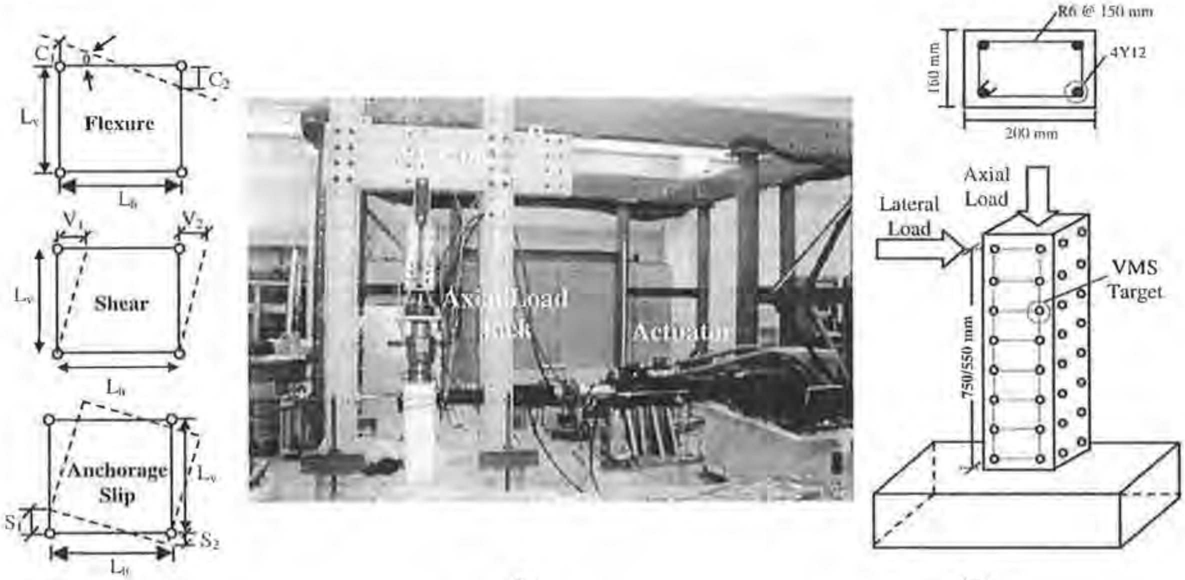
An experimental program was undertaken to investigate the cyclic force-deformation behaviour of half-scaled cantilever column specimens that are representative of real conditions in the MMA. The effects of shear on the deformation behaviour and failure mechanisms were of particular interest. The measured deformation has been resolved into the flexural, shear and yield penetration components using *Digital Close-range Photogrammetry Technique* (which is also known as the *Vision Methology System, VMS*). Movement of column segments, which cannot be measured reliably in such detail by conventional methods, was monitored using this technique. The measured deformation components have been compared with theoretical predictions.

With traditional experimentation, load testing of the column is normally terminated when the lateral load resisting capacity of the column has deteriorated by 20%. This approach seems justified in high seismic regions which are characterized by high energy demand associated with strong ground shaking which would require substantial residual strength capacity of the column to laterally support the building. Under conditions of displacement controlled behaviour which is low in energy demand, the building could survive the earthquake regardless of its residual lateral strength. The ability of a damaged column to carry axial load becomes the criterion to define the limit of ultimate performance. Whilst many column tests have been done in the past, only few tests emphasize the axial load carrying capacity of the column in the damaged stage [Moehle et al., 2002]. The experimentation described in this study uses this criterion to identify the deformation capacity of non-ductile columns.

2. EXPERIMENTAL PROGRAM

Surveys of soft-storey columns in the MMA revealed potential non-ductile behaviour due to lack of confinement. These columns were designed in accordance with the Australian Standard but without seismic provisions. The quantity of the transverse reinforcement was usually controlled by code-specified minimum requirements. An axial load ratio of 0.2 is typical for ground floor columns when calculated according to the suggestion in ATC-40 [1996]. Shear span-depth ratios vary between 2.0 and 4.6. The reinforcement ratio is found to be in order of 1.5%. For these columns, the influence of shear actions on the column behaviour becomes pronounced when the shear span-depth ratio is low (i.e. less than 3.5) in which case brittle failure may result.

To study the influence of shear on the behaviour of non-ductile columns, half-scaled reinforced concrete column specimens with varying shear span-to-depth ratios were tested. The specimens consisted of cantilever columns supported by a 300mm concrete block which was secured to the strong floor of the laboratory. Column sectional dimensions were 200mm in the direction of the applied load and 160mm in the orthogonal direction. An axial load ratio of 0.2 was selected to represent typical ground floor columns and shear span-to-depth ratios of 3.75 and 2.75 were used. The lengths of column specimens were 750mm and 550mm for first specimen (S1) and second specimen (S2) respectively. The reinforcement ratio for both columns was 1.4%. Detailing of column specimen is shown in Figure 1c.



(a) (b) (c)

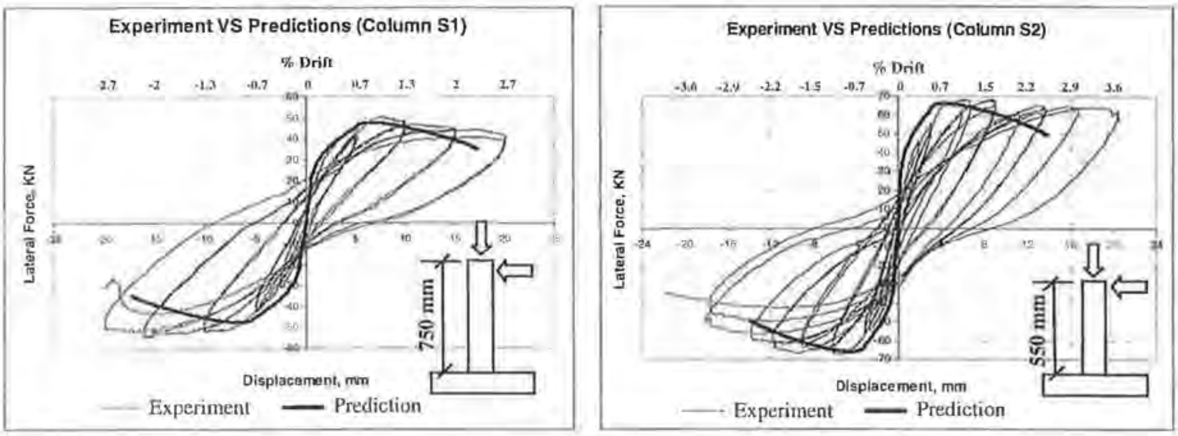
Figure 1 Experimental set up and interpretation of deformation from VMS data

The innovative VMS technique was used for measuring section deformations and surface strains of the columns in 3D (using closely spaced "targets" which were attached to the surface of columns as shown in Figure 1c). Using this technique, sectional deformation of the columns can be resolved into flexural, shear and yield penetration components as illustrated in Figure 1a.

The experimental set up as shown in Figure 1b was used to simulate gravity and lateral earthquake loads. The constant axial load of 280kN (axial load ratio = 0.2) was applied to a specimen using a 500kN-jack. The axial load was monitored and kept constant throughout the test. The lateral load was applied using a 250kN-hydraulic actuator. Specimens were subjected to quasi-static loading history as suggested by Priestley and Park [1987]. The loading history was modified to suit the non-ductile behaviour of the column. A column specimen was initially subjected to one cycle of lateral loading ±0.75 times the ideal yield strength at the critical section (F_y). The yield displacement was then found by extrapolating a straight line from the origin through the force-displacement point at $0.75F_y$ to the theoretical flexural strength F_y. The average of the two values calculated for the two cyclic reactions was adopted. Subsequent loading consisted of displacement-controlled testing to ductility ratios (μ) of ±1, ±2, ±3, ±4, ±5, and ±6. Two cycles of loading were used with each ductility ratio to ensure that the hysteretic behaviour could be maintained. Digital photos of the columns were taken when the peak displacement had been reached in each load cycle. The test was terminated only when the column had lost its axial load carrying capacity.

3. EXPERIMENTAL RESULTS

The force-deformation behaviour of the column specimens (S1 and S2) subjected to cyclic loading is shown in Figure 2. With the flexure-dominated column (specimen S1), the gravitational load carrying was lost following the buckling of the compression reinforcement. With column specimen S2, high shear forces have resulted in the formation of shear cracks distributed along the length of specimen. The specimen was capable of carrying full axial load whilst its lateral strength has deteriorated.



(a) Column S1 (b) Column S2

Figure 2 Hysteretic behaviour of column specimens under cyclic loading

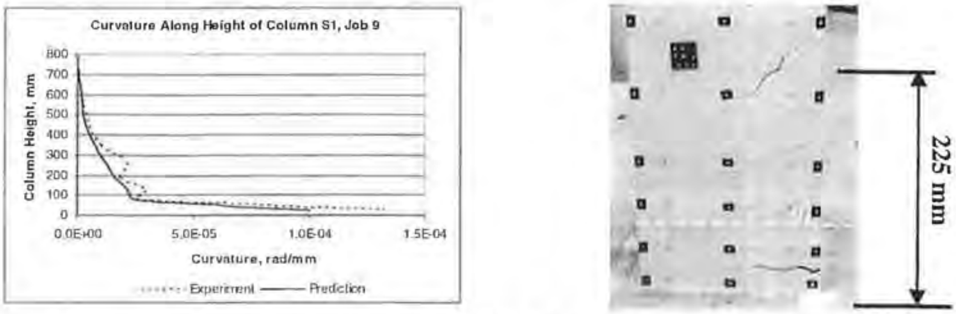
In both column specimens, yield deformation occurred at approximately 1% drift when existing cracks increased in size and new inclined cracks were developed. Shear cracks gradually developed and with the crack widths increased in subsequent cycles of loading. Deterioration of the column lateral strength was observed under cyclic loading. Lateral strength of column specimens dropped by 20% at a drift limit of approximately 2.7%. Bar buckling in column specimen S1 (due to spalling of concrete cover at the critical section) resulted in the sudden loss of the column gravitational load carrying capacity. It is concluded that column S1 failed in flexural compression given that column failure was initiated by the buckling of the compression reinforcement. In contrast, column S2 was able to sustain axial load following a 20% reduction in its lateral load resisting capacity. Such desirable behaviour is associated with the uniform distribution of shear cracks (as opposed to the localized spalling of concrete) in the column as shown in Figure 3b. Due to the influence of shear, the column is able to undergo more displacement before the compression strains reach a value sufficient to cause spalling of the cover concrete. Finally, at a drift of 3.6%, shear failure associated with the opening of diagonal cracks and buckling of longitudinal bars could be observed just before the column lost its axial load carrying capacity. Column S2 eventually failed in compression flexural shear. Figure 4 contains photos of the test columns when their axial load carrying capacities have been lost.

Curvature distribution along the column height is shown in Figure 3 along with the observed crack patterns. In specimens 1 and 2, combined shear and flexure stresses resulted in diagonal tension cracking. Crack angle (θ) tends to be steeper when the column is subjected to higher shear forces. At a certain drift limit, pronounced diagonal cracking in column S2 was observed over a wider zone compared to cracks formed in column S1. The comparison between curvature observed from the VMS technique and curvature calculated from bending moment at a section showed good agreement for column S1 but disagreement for column S2. This is because diagonal tension cracks in plastic hinge zones increase the available plastic rotation by spreading the zone of yielding along member [Park and Paulay, 1975]

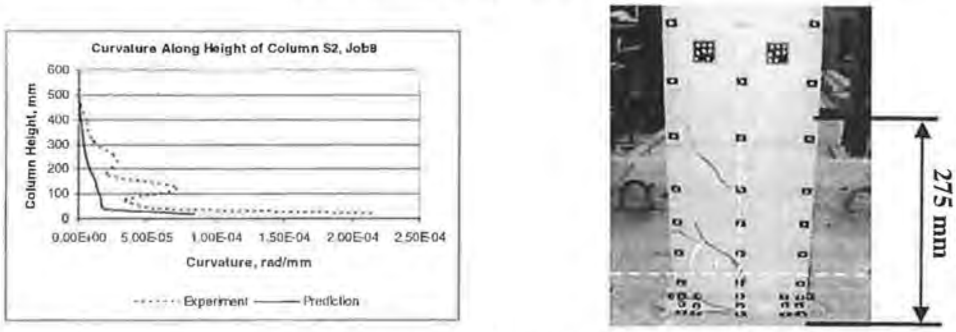
The VMS technique was employed to resolve deformation into the flexural, yield penetration and shear components. The deformation patterns in Figure 1a were used to interpret data obtained using VMS techniques (refer to Table1). From this table, it is clear that flexural deformation and yield penetration make a significant contribution to the total deformation of the column. Shear deformations calculated from the prescribed pattern were found to be approximately 5% and 10% of the total deformation for column S1 and S2 respectively. However, as explained previously, diagonal cracks in column S2 increased the available plastic rotation above the level predicted by flexural theory. There were difficulties in accurately accounting for additional deformation due to the effect of shear cracking.

4. THEORETICAL PREDICTIONS

The envelope curves of hysteretic force-deformation behaviour of column specimens were constructed by plotting lateral shear force against the summation of deformation components comprising flexural deformation, yield penetration and shear deformation. Flexural deformation can be estimated by integrating curvatures that have been calculated in accordance with representative stress-strain relationships of concrete and steel [Watson et al., 1994] assuming plane sections remain plane. The effects of yield penetration in the column longitudinal reinforcement at the anchorage to the foundation were estimated in accordance with the recommendations by Alsiwat and Saatcioglu [1992].

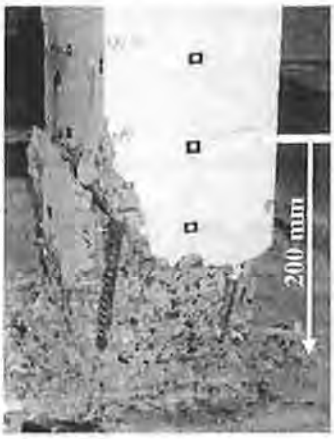


(a) Column S1 at 1.3 % drifts.



(b) Column S2 at 1.2 % drifts.

Figure 3 Curvature distributions along column height and their crack patterns



(a) Column S1

(b) Column S2

Figure 4 Column specimens after loss of axial load carrying capacity

Shear deformation was calculated using the truss analogy method which is consistent with behaviour of a cracked section [Park and Paulay, 1975]. Linear elastic behaviour of the concrete "strut" and the steel (stirrups) "ties" is assumed in the method. This led to an over-prediction of shear deformation in non-ductile columns. An alternative method is to use modified compression field theory (MCFT) [Vecchio and Collin, 1986] in which each concrete crack is treated as an element in its own right and with its own characteristics (i.e. equilibrium and constitutive relationships formulated in terms of the average stresses and strains). In this study, the MCFT was used to predict shear deformation of the column specimens. It was found that shear deformations predicted by the MCFT were in broad agreement with experimental results.

Table 1 Deformation components of column specimens

	Half Cycle	Deformation Components Obtained From VMS								Predictions	
		Total Def.		Flexural Def.		Yield Penetration		Shear Def.		*Total Def.*	*Shear Def.(MCFT)*
		(mm)	%Drift	(mm)	(%)	(mm)	(%)	(mm)	(%)	*(mm)*	*(mm)*
Specimen S1	3	**5.06**	*0.67*	**3.9**	*77.19*	**0.98**	*19.35*	**0.17**	*3.46*	***4.47***	***0.17***
	4	**4.86**	*0.65*	**3.71**	*76.35*	**0.94**	*19.25*	**0.21**	*4.40*	***4.47***	***0.17***
	7	**9.90**	*1.32*	**8.05**	*81.36*	**1.43**	*14.40*	**0.42**	*4.24*	***9.30***	***0.30***
	8	**9.89**	*1.31*	**7.99**	*81.09*	**1.35**	*13.75*	**0.51**	*5.17*	***9.30***	***0.30***
Specimen S2	3	**3.05**	*0.55*	**2.25**	*73.82*	**0.55**	*17.95*	**0.25**	*8.23*	***2.27***	***0.17***
	4	**2.75**	*0.5*	**1.89**	*68.72*	**0.69**	*25.00*	**0.17**	*6.28*	***2.27***	***0.17***
	7	**6.53**	*1.19*	**4.64**	*70.97*	**1.26**	*19.35*	**0.63**	*9.68*	***4.67***	***0.47***
	8	**5.74**	*1.05*	**3.82**	*66.63*	**1.37**	*23.84*	**0.55**	*9.53*	***4.67***	***0.47***

The aforementioned deformation model have been used to construct envelope curves for hysteretic force-deformation behaviour of the columns (refer to Figure 2). The model is shown to provide good predictions for the ascending curve for column S1 but some discrepancies between experimental results and experimental predictions are evident for column S2. This is explained by the additional contributions to rotation of the column by shear cracking. It has also been found that the model over-predicts post-peak deformation.

Whilst column deformation could be increased by shear cracking, the available column ductility could be compromised by shear strength degradation. In this study, shear strength of column specimens have been checked against recommendations by ATC-40 [1996], Priestley et al. [1994] and Moehle et al. [2002]. Both columns are not expected to fail in brittle shear mode but there is a possibility that shear strength degradation may lead to shear failure in column S2.

The model based on shear-friction theory developed by Moehle et al. [2002] has also been used to estimate the ultimate limit of deformation (at which the axial load carrying capacity of the column will subside). This model for predicting limiting drift is only valid for columns that are expected to fail in shear (i.e. column S2). Using this model, the limiting drift was found to be 2% based on a crack angle of 65 degree whereas the limiting drift is approximately 3.6% when the crack angle varies between 35 and 65 degree. It is evident that this model is overly conservative in predicting limiting drift. Furthermore, the prediction is very sensitive to the assumed crack angle (which is very sensitive to changes in height within the column).

5. CLOSING REMARKS

Results from experimental investigation into the cyclic behaviour of half-scaled reinforced concrete columns have been reported. For both columns, deformation at yield and 20% loss of lateral strength was found to be 1% and 2.7% drift respectively. Compression bar buckling has resulted in the axial failure of column specimen S1 at a drift limit of 2.7%. In contrast, column specimen S2 was able to maintain the full axial load up to a drift limit of 3.6% (at which the column failed by flexural shear). The test revealed that additional rotation at shear cracks increased deformation capacity of columns. The ability of columns to sustain axial load at large deformations was improved due to this effect. The deformation model provided satisfactory predictions for the ascending envelope curve but over-predicted post-peak deformation. An analytical model that can accurately and reliably estimate the limiting drift of a column (at the threshold of loss in axial load carrying capacity) has yet to be developed. Further column tests with different detailing and shear span-to-depth ratios will be undertaken to validate the model. Subsequently, the model will be part of the displacement-based methodology that can be used in predicting the performance of soft-storey buildings in regions of low to moderate seismicity.

6. ACKNOWLEGMENTS

The research is funded by the AEES Earthquake Engineering Research Scholarship, 2002. The authors would like to acknowledge AEES for financial support.

7. REFERENCES

Alsiwat, M. and Saatcioglu, M., (1992) Reinforcement Anchorage Slip Under Monotonic Loading, Journal of Structural Engineering, Vol 118(9), pp 2421-2437.

ATC40 (1996) Seismic Evaluation and Retrofitting of Concrete Buildings, Applied Technology Council, USA.

Moehle, J., Elwood K. and Sozen, H., (2002) Gravity Load Collapse of Building Frames During Earthquakes, Special Publication, Uzumeri Symposium, American Concrete Institute, Farmington Hills, Michigan.

Park, R. and Paulay, T., (1975) Reinforced Concrete Structures, Wiley.

Priestley, M. and Park, R., (1987) Strength and Ductility of Concrete Bridge Columns Under Seismic Loading, ACI Structural Journal, Vol 84(2), pp 61-76

Priestley, M., Verma, R. and Xiao, Y., (1994) Seismic Shear Strength of Reinforced Concrete Column, Journal of Structural Engineering ASCE, Vol 120(8), pp 2310-2329.

Vecchio, FJ., and Collin, MP., (1986) The Modified Compression Field Theory For Reinforced Concrete Elements Subjected to Shear, ACI Structural Journal, Vol 83(2), pp 219-231.

Watson, S., Zahn, F. and Park, R., (1994) Confining Reinforcement for Concrete Columns, Journal of Structural Engineering, ASCE, Vol 120(6), pp 1798-1824.

OUT-OF-PLANE FLEXURAL RESPONSE TESTSUSING DRY-STACK MASONRY

J. VACULIK[1], M. C. GRIFFITH[2], B. M. E. HOGARTH[3], J. TODD[3]
UNIVERSITY OF ADELAIDE

AUTHORS:

[1] PhD Candidate, Department of Civil and Environmental Engineering
The University of Adelaide, Australia

[2] Associate Professor, Department of Civil and Environmental Engineering
The University of Adelaide, Australia

[3] BE Graduates, Department of Civil and Environmental Engineering
The University of Adelaide, Australia

ABSTRACT

The complexities of the out-of-plane behaviour of unreinforced masonry (URM) walls have resulted in a limited understanding of the topic despite over 30 years of research. A lack of experimental data, especially for walls with openings adds to the difficulties in understanding the subject. This paper summarises a series of experimental tests recently performed at the University of Adelaide on half-scale, dry-stack (absence of mortar) masonry walls. The first key aim of experimental testing was to identify failure mechanisms in walls with various configurations especially walls with openings, in order to form a basis for the development of a new analytical method. The second aim of this research was to produce load-displacement data for walls with various aspect ratios and levels of vertical precompression.

1. BACKGROUND

While there exists a large amount of research relating to the in-plane lateral loading of unreinforced masonry (URM) structures, current understanding of the out-of-plane flexural behaviour of URM is fairly limited. This is a troubling fact from the point of view of seismic safety, since masonry is fundamentally weak when resisting lateral loads in the out-of-plane direction. Especially complex is the problem of two-way bending, which occurs when walls supported on at least two adjacent sides are subjected to lateral face loads. Such walls undergo a combination of horizontal, vertical and diagonal bending, with failure occurring along the crack lines associated with each type of bending. The applied moments are resisted by a combination of flexure and torsion acting on the bed and perpend joints.

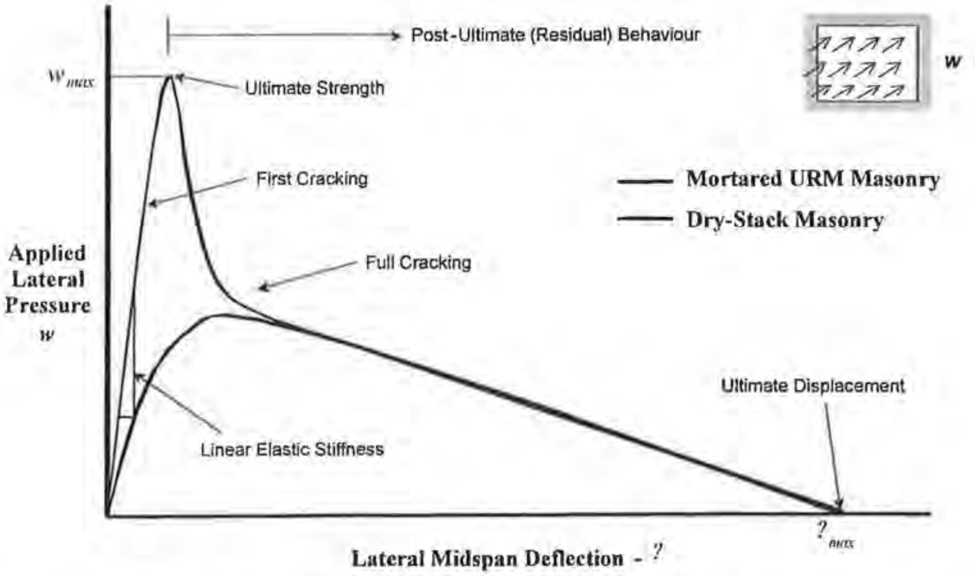


Figure 1 – Typical behaviour of URM and DSM in two-way bending

The complexity of the behaviour of URM walls arises from the structural indeterminacy of the usual wall configurations, coupled with the anisotropic and brittle nature of URM materials (Drysdale 1993). As shown in Figure 1, a typical URM wall undergoing lateral displacement deforms in a linear elastic manner until its flexural tensile strength is reached and cracking of joints begins to occur. The wall loses stiffness with progressive cracking of joints and typically reaches its ultimate load capacity at a relatively small displacement compared to its ultimate displacement capacity. Deformation beyond the point of ultimate load capacity results in continued reduction of stiffness due to continued cracking until full cracking is reached. Following full cracking, residual resistance is provided by a restoring moment due to vertical loads as well as frictional resistance mechanisms on the bed joints. A point of maximum displacement is finally reached, beyond which the wall becomes unstable.

2. EXPERIMENTAL TESTING

Because of the lack of experimental data available for walls with openings, testing in this project was focused on obtaining data for the ultimate strength and cracking patterns for walls with different combinations of door and window openings. A second aim of testing was to obtain load-displacement data for walls with various aspect ratios and levels of vertical precompression. For brevity, throughout this paper the term URM refers to conventional unreinforced masonry constructed with mortar, whereas DSM refers to dry-stack masonry walls constructed without mortar.

2.1. Dry-Stack Masonry

Experimental testing was performed using half-scale dry-stack masonry (DSM) walls. The half-scale bricks were cut down from standard clay pavers into units with the average dimensions of 115_55_33 mm. The average material unit weight was 20.2 kN/m_. Although URM would be preferred for its structural behaviour to provide more representative results with respect to conventional construction, it was infeasible due to time and financial constraints.

The key advantages of DSM over mortared masonry lie in its construction and cost efficiency, both of which were essential for the laboratory work conducted in the project. However, for practical use, we require that the structural behaviour of DSM behaves in a manner to similar mortared masonry. The ultimate strength of URM walls is governed by the tensile capacity of the mortar joints and is therefore much greater than that of DSM. Consequently, the ultimate strength of DSM cannot be compared directly with the ultimate strength of URM. Nonetheless, since the opening up of unit joints in DSM to form distinct cracks is fundamentally similar to the cracking of mortared masonry due to the breaking of mortar joints, it follows that similar analytical models can be used to predict the strength of both masonry types. In addition, once mortared masonry becomes fully cracked it behaves in a similar manner to DSM, since its behaviour is controlled fundamentally by frictional resistance and flexural rocking of rigid bodies. Thus it is expected that the load-deflection behaviour beyond the point of ultimate load is similar for the two methods of construction (see Figure 1) and that consequently the residual behaviour of mortared walls can be investigated using DSM. Furthermore, it is believed that the load-displacement curves obtained using DSM may provide a reasonable basis for the development of a displacement-based method for seismic design of URM.

2.2. Tilting Table Tests

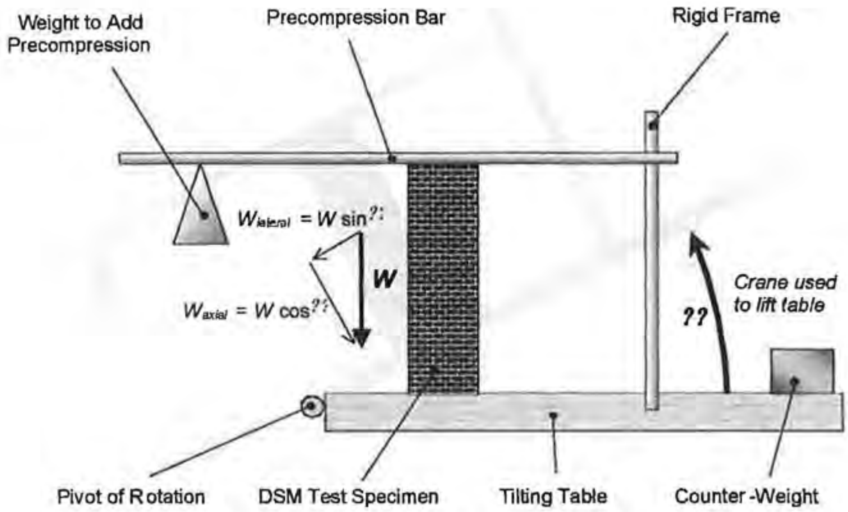
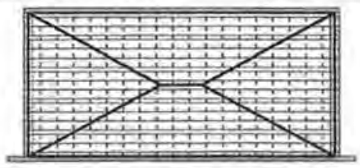
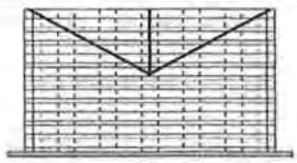
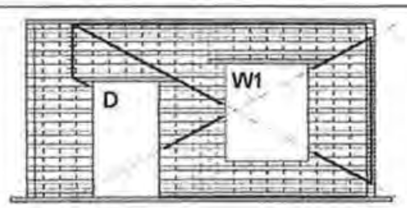
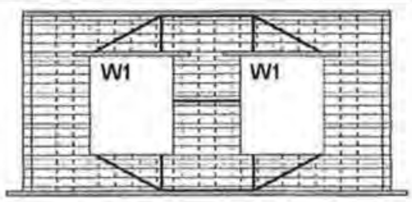
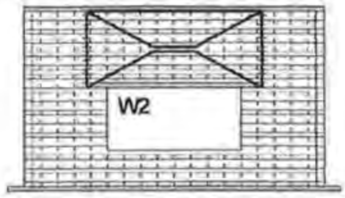
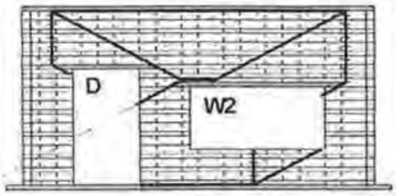


Figure 2 – Tilting table test setup

Table 1 – Summary of experimental results for tilting table tests

WALL ID.	WALL DIMENSIONS LENGTH (mm)	WALL DIMENSIONS HEIGHT (mm)	OBSERVED COLLAPSE MECHANISM	σ_v (kPa)	W_{ult} (%)
1A	2128	960		35.5	42.0
2A	1668	960		0.0	8.2
2B	1668	960		0.0	8.2
3A	2358	1170		7.6	8.3
3B	2358	1170		8.4	23.7
3C	2358	960		8.2	34.9
4A	2473	1170		7.4	8.5
4B	2358	1170		8.4	27.0
4C	2358	960		8.4	27.3

5A 5B 5C	2013 2013 2013	1170 1170 960		8.4 9.3 8.9	18.6 29.8 43.5
6A 6B 6C	2358 2358 2358	1170 1170 960		7.6 8.5 8.0	13.1 16.8 36.2

Notes:
- Ultimate strength (W_{ult}) is expressed as percentage of the axial weight at collapse.
- Observed cracking patterns are shown only for the A specimens (for brevity).
- Axial precompression (σ_v) calculated at the point of failure.
- Dimensions of openings (L_H) in mm: Door (D) 460_780, Window 1 (W1) 575_660, Window 2 (W2) 920_420

A tilting table test setup was used to slowly tilt the DSM walls until collapse as illustrated in Figure 2. A counter-weight was provided to ensure the table remained stable during the test, while a crane was used to slowly lift the table. Precompression was applied on some walls using leverage. All tested walls (except wall 2A) were simply supported at the top and bottom edges, with short end returns at the side edges to provide lateral support. During tilting the self-weight of the wall is gradually converted into a lateral force component. A major advantage of this is that a uniform lateral force is applied to the wall based on the distribution of mass, analogous to earthquake loading. Its shortcoming is that the test is load-controlled and thus provides no insight into the behaviour of the test specimens beyond the maximum strength (see Figure 1).

Because the resistance provided by the flexural and frictional torsion mechanisms is proportional to the normal force acting upon the bed joints, it follows that the overall lateral load resistance of a DSM wall is proportional to the axial force. Therefore, as the wall tilts it gradually loses strength and hence it becomes convenient to express the strength of a DSM wall as the ratio of the applied lateral load to the axial weight of a wall at the point of failure ($W_{lateral}/W_{axial}$), which is equal to the tangent of the angle of tilt (θ) at failure (see Figure 2).

The variability in the ultimate strength (W_{ult}) is highlighted in the experimental results shown in Table 1. Specimens A and B have identical dimensions, while the C specimens have slightly shorter height. In the A specimens, the lateral ends were stabilised using precompression bars on the end return walls (see Figure 2), whereas in the B and C specimens the returns were clamped. Consequently it is believed that the vertical edges in walls A behaved as simply supported, while the vertical edges in walls B and C behaved with a greater degree of rotational fixity. Both types of boundary conditions are commonly found in practice, with the former corresponding to walls stabilised with short returns and the latter representing walls with fixed vertical edges, for instance when the wall is built into an engaged pier or another wall.

Walls 2A and 2B show an excellent degree of repeatability, as the failure mechanism was localised along the unsupported edge and was therefore unaffected by the side boundary conditions. However, significant variability was observed between the A and B specimens in the other tests, especially in walls 3 and 4, where the ultimate strength of the B specimens was approximately triple that of the A specimens. It is uncertain why this difference was so large, given that in each case that A and B specimens failed by very similar mechanisms, however it is believed that the aforementioned difference in the vertical edge boundary conditions may have been at least partially responsible. Furthermore, due to possible arching of the wall with increasing deflection, clamping of the return walls in the B specimens may have caused a significant increase in precompression within the return walls, thus increasing the internal resistance within the wall.

It should be noted that although strict application of the Australian Masonry Code AS 3700 (Standards Australia 2001) predicts zero strength for masonry in the absence of mortar, the DSM specimens tested demonstrated a reasonable load capacity purely due to the effects of self-weight and any additional applied vertical precompression. For instance, wall 1A was shown to have sufficient strength to resist an equivalent acceleration of 0.42 g's.

2.3. Airbag Tests

Airbag tests were used to gain insight into the load-displacement behaviour of DSM masonry, which is expected to resemble to the residual behaviour of URM as discussed in Section 1. Walls were tested under displacement control, enabling the collection of post ultimate strength data, with both unloading and reloading. Walls with three different aspect ratios (height/length) were tested, including H/L = 0.40, 0.45, 0.58. Each wall was tested with three different levels of vertical precompression. The load-displacement behaviour of one of these specimens is shown in Figure 3, whereby the lateral load as a percentage of weight is plotted versus the ratio of displacement to wall height and versus the ratio of displacement to wall thickness.

It may be worth noting that the wall tested with 0.036 MPa precompression (Figure 3) was a repeat of test 1A performed using the tilting table. The tests, however, show poor repeatability with the wall achieving a strength to weight ratio of 42 % on the tilting table and 24 % using the airbag setup. The reason for this difference is not entirely clear, given that the wall configurations were identical, however the airbag used did not cover the entire face of the wall and thus tended to concentrate the load slightly toward the centre of the wall. Although this would reduce the load resistance of the wall, it is unlikely that this would fully account for the difference in the observed strength.

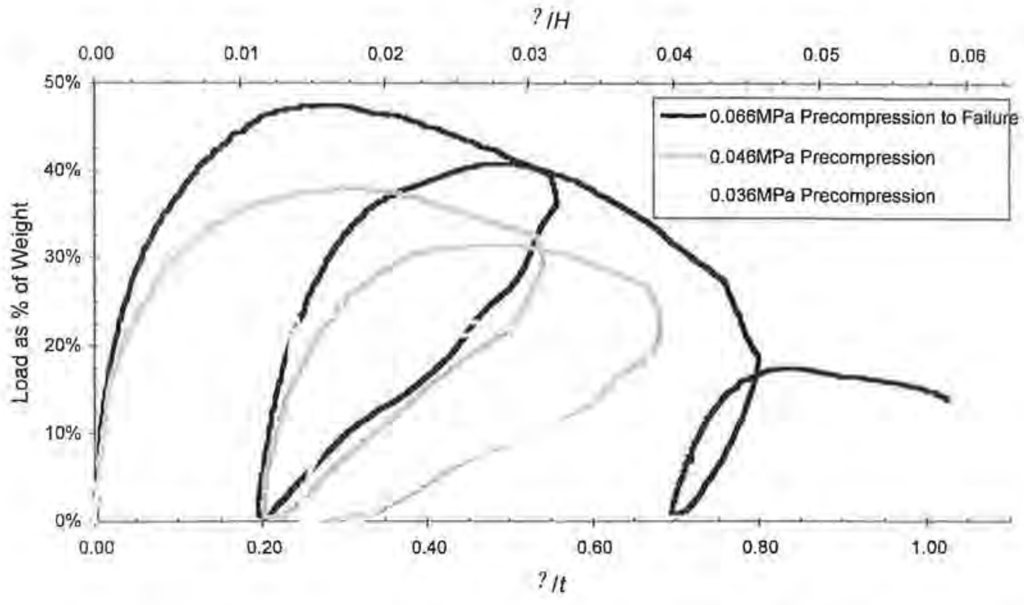


Figure 3 – Load vs Displacement of DSM (*L* = 2180mm, *H* = 960mm, *t* = 55mm)

As the above curves demonstrate, the DSM specimens behave in a reasonably ductile manner beyond their ultimate load. The curves show that after ultimate strength there is no sudden drop of load bearing capacity of the walls, but a gradual decline. All the walls subject to displacement-controlled tests exhibited similar ductile behaviour. In each case, the unloading path did not follow the initial loading curve, implying that the walls behave inelastically. As the walls are reloaded the curve rejoins the initial loading curve at the point of unloading. These observations are a direct result of the frictional resistance mechanisms involved in resisting the lateral loads, which behave in a naturally ductile and inelastic manner.

3. CONCLUDING REMARKS

This paper summarised a series of recent experiments at the University of Adelaide into the behaviour of dry-stack masonry under out-of-plane loading. For a detailed account of results the reader is referred to Vaculik et al (2003). The observed cracking patterns for DSM were generally consistent with those expected in mortared URM. Thus it is proposed that tilting table tests on DSM may be used as an alternative method of identifying failure modes in URM under earthquake induced loading. In contrast to the non-linear elastic behaviour observed by Doherty et at (2002) for URM in one-way vertical bending, the post-peak behaviour of masonry walls in two-way bending has been shown to be non-linear inelastic. It is therefore expected that hysteretic damping may be significant for URM in two-way out-of-plane bending under dynamic loading.

4. REFERENCES

Doherty, K., Griffith, M. C., Lam, N., and Wilson, J. (2002). *Displacement-Based Seismic Analysis for Out-Of-Plane Bending of Unreinforced Masonry Walls.* Earthquake Engineering and Structural Dynamics 31: 883-850

Drysdale, R. G., Hamid, A. A., and Baker, L. R. (1993). *Masonry Structures: Behaviour and Design*. New Jersey, Prentice-Hall.

Standards Australia (2001), *Australian Standard AS 3700-2001* (SAA Masonry Code). Standards Australia, Sydney

Vaculík, J., Hogarth, B., Todd, J. (2003). *Experimental and Analytical Investigation of the Flexural Response of Brick Masonry*. School of Civil and Environmental Engineering. Adelaide, The University of Adelaide: 140.

DEVELOPMENT OF A PHOTOGRAMMETRY SYSTEM TO MONITOR BUILDING MOVEMENT

D. J. HEATH[1], J. CORVETTI[2], E. F. GAD[3] AND J. L. WILSON[4]
THE UNIVERSITY OF MELBOURNE

ABSTRACT:

This paper reports on an ongoing research project investigating the response of residential structures to blast vibrations from mining activities. One aim of this project is to establish levels of ground vibration and resulting strains responsible for crack initiation and propagation in masonry and plasterboard. This paper focuses on the measurement techniques developed as part of this project to monitor and measure cracks that may develop due to blasting or environmental loads such as foundation settlement and thermal changes. The development and application of close-range digital photogrammetry is discussed for static surveys. This system provides high quality measurements in terms of accuracy and resolution.

Photogrammetry has traditionally been used for topographical surveys using aerial photography. With the evolution of digital technology, advancements in photogrammetry have been tremendous. In this project photogrammetry has been used to survey large structures for three-dimensional measurements. Close-range digital photogrammetry measures deformation using digital photographs and a specialised software program to establish the three-dimensional coordinates of targets located on the structure. Comparisons between surveys reveal relative deformations. The capabilities of this system are discussed with respect to measuring fine crack development in masonry and plasterboard where conventional LVDT's and strain gauges are impractical due to difficulty in predicting location of crack formation and direction of crack propagation. This system has been developed for laboratory and in-field measurement of deformation at local and system levels.

[1] PhD Candidate, The University of Melbourne, Victoria, Australia
[2] PhD Candidate, The University of Melbourne, Victoria, Australia
[3] Senior Research Fellow, The University of Melbourne, and Senior Lecturer, Swinburne University of Technology, Victoria, Australia
[4] Associate Professor, The University of Melbourne, Victoria, Australia

1 – INTRODUCTION

Blast vibrations from mining activities propagate considerable distances in the form of ground vibrations and airblast. Ground vibrations are transmitted through compression (P), shear (S) and Rayleigh (R) waves, while air vibrations (airblast) propagate through air similar to a sound wave. In many parts of Australia residential areas experience blast vibrations due to their proximity to mines and quarries where blasting practise is employed. When a residential structure experiences vibrations it responds in racking and flexural modes. Racking is predominantly generated by ground vibrations and has the greatest damage potential. Airblast and ground vibrations cause an out-of-plane response often inducing secondary rattling and although this is startling to residents, it has little potential to cause damage.

Concern has been expressed by some residents that blast vibrations are damaging their homes. A recent study (Moore et al, 2002) has been conducted on three houses subjected to varying levels of blast vibration in the Newcastle region of N.S.W. This report concluded that current environmental vibration limits were conservative with respect to the onset of damage. These results are consistent with limits set by the U.S. Bureau of Mines (Siskind et al, 1980) for blasting practice in the United States. However, two key areas were identified as requiring further research and improvement: the first is the identification of damage thresholds in non-structural residential components including masonry (Heath et al, 2003) and plasterboard (Corvetti et al, 2003), secondly, the quantification of residual stresses in houses due to environmental loads such as variations in temperature, soil moisture, wind and humidity.

An environmental monitoring programme has commenced using terrestrial (close-range) photogrammetry that involves the measurement of deformations in a number of residential structures over a 12 month period. Photogrammetry may be used to reliably and accurately calculate structural deformation to assist in predicting residual stresses. Existing measurement systems offering comparable accuracy do exist such as strain gauges, LVDT's and level loop surveys but have practical and economic limitations in terms of resolution of measured points.

This paper provides an overview of photogrammetry and the development of a mid-range system in Section 2 and presents a number of case study applications in Section 3, with conclusion in Section 4.

2 – PHOTOGRAMMETRY BACKGROUND

Digital photogrammetry is a relatively recent surveying technique although the origins have been traced back to Leonardo da Vinci in 1492 when he demonstrated the principles of optical projection. In 1864 French Colonel Aimé Laussedat demonstrated the principles of photographic surveying through a rooftop survey of Paris (Slama, 1980). The terminology "photogrammetry" was adopted in the early 20th Century during the establishment of various surveying societies.

Photogrammetry is a three-dimensional coordinate measuring technique. Traditionally, this surveying technique has been used for aerial (analogue) photogrammetry where consecutive overlapping photographs are used to establish

topographic information. Close-range digital terrestrial photogrammetry uses the same principles but the medium for measurement is digital photographs.
By having at least two photographs of the same targets, triangulation of the points can be achieved by developing "optical rays" from the camera to each target. Hence, the location of the camera to each point common to both photographs may be identified via mathematical intersection. Triangulation is a two phase process consisting of interior and exterior orientation. Interior orientation establishes internal camera geometry, enabling exterior orientation which establishes the camera location in object space. Digital photogrammetry may also be extended to real-time measurement using two fixed (tripod-mounted) cameras.

2.1 – PURPOSE OF A PHOTOGRAMMETRY SYSTEM

One of the advantages of a photogrammetric survey is the ability to measure a network of targets at a high resolution and high accuracy. This surveying technique offers a non-contact, unobtrusive method of determining object geometry that is particularly important when the functionality of the object must not be disturbed. The retro-reflective targets are durable in an exterior environment, permitting monitoring over time. The three-dimensional measurement capabilities offer detection of local and global deformations.

There are many advantages for selecting photogrammetric surveys over traditional surveying techniques. Strain gauges and LVDT's are conventional instrumentation offering similar accuracy in measurement but have practical and economic limitations. An additional feature of photogrammetry over traditional measurement techniques is the ability to measure three-dimensional deformation between targets. The advantage of photogrammetry over a theodolite is that each photograph is capable of capturing many points. One of the beneficial abilities of terrestrial photogrammetry is its ability to provide a high resolution of points being measured with relative ease. This is particularly useful when an object is deforming from an elastic to an inelastic range such that the location and direction of propagating cracks cannot be accurately predicted.

2.2 – HOW PHOTOGRAMMETRY WORKS

Existing photogrammetry systems are capable of using natural features of an object for measurement although accuracy is limited. To enhance accuracy, retro-reflective targets are fixed to the surface of the object at salient locations. These targets are highly directional, efficiently reflecting light towards its source. The object is then photographed from different locations ensuring overlap between photos. The number of photos should reflect the complexity and size of object geometry. A high-end consumer hand-held digital camera is sufficient and does not require tripod mounting. The digital images are then processed by software using triangulation and a least-squares adjustment to establish three-dimensional coordinates of the targets.

2.3 – COMPONENTS OF A PHOTOGRAMMETRY SYSTEM

Industrial grade photogrammetry systems exist with the industry standard being the INCA camera/V-Stars software combination developed by GSI in Florida, USA. Tests show this system exceeds the 1 part in 120,000 accuracy standard specified by the supplier (Brown, 1998). For an object with largest dimension 2.4m this translates to an accuracy of 20 micron. In this research project, it was not economically feasible to acquire this system so the decision was made to build a mid-range system at a fraction of the cost (approximately 5%) of the industrial grade system. Development has been in collaboration with the Department of Geomatics at The University of Melbourne.

The hardware components of a semi-automated mid-range photogrammetry system are a digital still video CCD camera, scale bar and retro-reflective targets. To ensure internal geometry of the camera remains stable, a fixed focal length lens must be used. At present, a 6.1 megapixel Nikon D100 camera body is coupled with a Nikkor 18mm (wide angle) lens. Internal geometry has been stabilised by fixing the focus within the lens and providing additional restraint to clamp the lens to the camera body. Clamped to the end of the lens is a ring flash to ensure the lens is at the centre of the light source due to the high directionality of reflection from the targets. A photo of the camera with the ring flash is shown in Figure 1.

Figure 1: Nikon D100 with Sunpak ringflash and optical stabilisation.

A scale bar is a device containing two or more calibrated targets providing a reference for scale in object space. Targets that are fixed to the object are essentially a tape that is 100 to 1000 times more reflective than white paper with a black mask containing circular penetrations. The required diameter of the target depends upon the distance between the targets and camera. A 6mm diameter target (refer to Figure 2) is a general purpose size requiring a camera-to-target distance of less than 4m for the mid-range photogrammetry system. This criterion ensures each target is adequately represented by a sufficient number of pixels. The dark tape ensures good contrast between the circular target and material immediately adjacent and a circular target shape. These characteristics are important for analysis in the software.

Figure 2: Retro-reflective targets 12mm, 6mm and 3mm in diameter.

2.4 – AUTOMATED AND SEMI-AUTOMATED MEASUREMENT

Fully automated close-range photogrammetry requires an exterior orientation device (EOD) to provide a reference for each target during automated analysis. This hardware contains at least five calibrated targets within a circular ring. Industrial grade digital photogrammetry systems are capable of achieving fully automated three-dimensional measurement using coded targets. However, the photogrammetry system developed at The University of Melbourne offers full automation only for surveys that are predominantly two-dimensional. Full automation may be achieved only when the EOD is viewed in each photograph. For large objects or objects requiring loop surveys semi-automation must be used as the EOD may not be viewed in all photographs.

Australis is the software used for this photogrammetry system. Camera and scale bar data may be recalled from a database with the camera data being linked to photographs from the survey. Semi-automation requires the examination of photographs individually for the manual labelling of each target. There are two phases in measuring points. The first is triangulation (resection and intersection) that is performed on each image and the second is a least squares bundle adjustment. For semi-automated measurement, triangulation is performed on each image. A bundle adjustment is performed periodically to progressively update a network of three-dimensional data coordinates corresponding to measured points.

Semi-automated measurement requires a window to be placed over a target candidate in a photograph. Australis uses an algorithm to examine the RGB intensity of each pixel within that window. If the RGB distribution contains a pattern of values above a threshold value conforming to a predefined target (circular) shape, a centroid calculation will be performed. This process is used when automation is not possible or when image quality is poor.

Automated measurement is similar to semi-automated measurement without manual processing. Instead, an algorithm scans every pixel of each image examining the RGB value to identify potential targets. When groups of pixels qualify for shape and intensity criteria, a centroid is calculated and a potential target identified. The centroid location is referenced to object space according to the location of the EOD. The final step is the matching of coordinates of each potential target in object space and the automatic assignment of target labels.

Both measurement techniques achieve a three-dimensional network of targets in an arbitrarily assigned coordinate system. If the purpose of surveying is to measure deformation, a reference survey must be made prior to the object undergoing deformation. The targets must remain on the object during deformation and then surveyed once deformation is complete. The same analysis procedure is applied ensuring a consistent labelling scheme. It is then possible to evaluate how the object has deformed by comparing the two datasets. For short-term and laboratory surveys it is possible to have stable targets that do not experience deformation (attached to the laboratory floor for example). By selecting stable (static) targets to define the coordinate system common to both surveys, deformation is measured by subtracting the x-, y- and z-coordinates of the first survey from consecutive surveys. If stable reference targets are not possible, consecutive survey datasets must be transformed onto the control dataset. This is done according to a least squares adjustment with the resulting difference between the two datasets yielding deformation.

2.5 – SYSTEM CAPABILITIES

Two aspects of photogrammetry that can have a dramatic influence on the accuracy of results are associated with the object setup and measuring device. For the object, a close spacing of targets will improve the accuracy of a survey but a complex network of targets may reduce accuracy. An object that contains targets predominantly in a single plane will be significantly less accurate for measurements made out-of-plane. Factors associated with the measuring device known to affect accuracy include the number and size of pixels in the CCD, stability of the internal camera geometry and the number of photos taken of each target during the survey. An increase in the number of pixels in the CCD translates to each pixel representing a smaller space on the object. The internal stability of the camera is critical for accurate triangulation and an increase in the number of photographs promotes a convergence towards the true target centroid. There are other factors affecting accuracy but these are beyond the scope of this paper and are discussed elsewhere (Kraus, 1993 and Mikhail et al, 2001).

3 – VERIFICATION AND PERFORMANCE OF PHOTOGRAMMETRY SYSTEM

The commissioning of the mid-range photogrammetry system has included a number of surveys and calibration exercises. During this process modifications were made to the hardware and survey procedures. Calibration was achieved using a micrometer, calibration wall and the survey of a full-scale house. Some of these are discussed below.

3.1 – CALIBRATION WALL

The calibration wall is considered to be free from deformation including thermal variation and foundation settlement. It is constructed from concrete blockwork in a climate controlled environment. This wall hosts a network of targets calibrated by Vision Metrology Services using an industrial grade photogrammetry system. The wall has retro-reflective targets attached on a grid with spacings of approximately 400mm x 400mm as shown in Figures 3 & 4.

Figure 3: Targets on 'calibration wall' illuminated by flash.

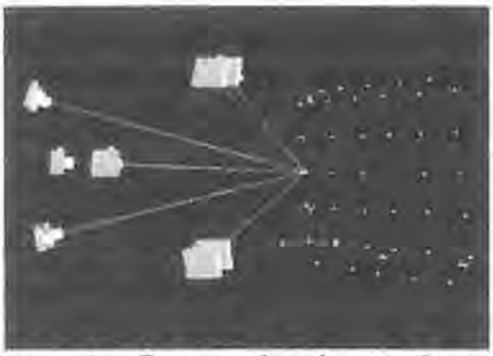


Figure 4: Survey showing optic rays from a target to each camera location.

The calibration wall is 2000mm x 2000mm such that each target has been measured using the industrial photogrammetry system to within 17 micron of its true value. A survey of this wall has also been conducted with the newly developed mid-range photogrammetry system. A comparison of the results revealed the mid-range system has an accuracy of 33 parts per million (ppm). That is, a possible 'sphere of error' with a radius of 60 micron (again) around each target centroid (Refer to Figure 5). Included in this accuracy measurement is a 13 micron error inherent in the targets.

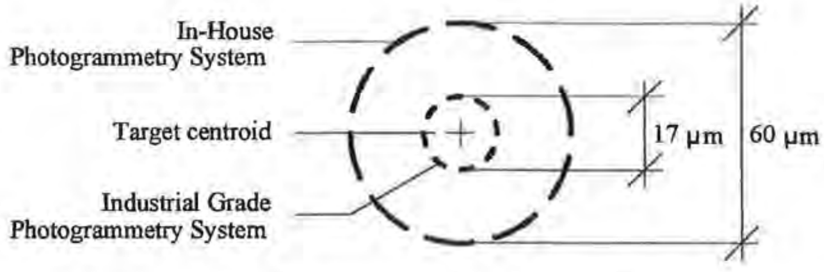


Figure 5: Schematic representation of performance of a mid-range photogrammetry system relative to an industrial grade photogrammetry system.

3.2 – 3-D HOUSE SURVEY

Large-scale applications are currently in progress for the monitoring of movement in residential structures, one of which is located in Bacchus Marsh, Victoria. The purpose is to establish structural movement due to environmental factors, particularly vertical movements due to soil heave. Figure 8 depicts this house that has targets located just above the damp proof course and just below eave level following the same two mortar courses around the entire structure. Target spacings are approximately 1m on the structure that is 28.6m in length and 8m in width. Figure 9 depicts targets and approximately 250 camera locations from during the loop survey, solved for during analysis.

Figure 8: Residential structure undergoing environmental monitoring

Figure 9: 3-D model depicting targets and camera locations

Due to limitations of the mid-range system and complex object geometry, techniques have been developed to achieve accuracy comparable to an industrial grade photogrammetry system. These techniques include roofline targets visible form both sides of the structure, rotating targets beyond the structure perimeter and targets located on corners. Monthly surveys are being conducted to record movement due to seasonal variations with subsequent surveys being used to establish deformation.

This monitoring programme is being conducted in parallel with a study for the Office of Housing (OoH) that manages the public housing of Victoria. The OoH has over 1500 damage inspection reports for different properties scattered throughout Victoria. An interrogation of these reports is being made to identify typical damage attributable to environmental factors in residential structures. Trends obtained from this component of research will assist with the on-going monitoring program to establish the effects of environmental loads and verify damage thresholds. The outcome of this work will provide guidelines for the mining and insurance industries to quantify expected damage in residential structures due to typical environmental conditions as well as the combination of environmental and vibration induced loads.

4.0 – CONCLUSION

There are various regions in Australia where residential areas are in sufficient proximity to mines and quarries to experience vibrations from blasting. Some residents are concerned that current blast practises are responsible for causing damage to their homes. This has necessitated research into establishing damage thresholds for non-structural components in houses and quantification of stresses due to environmental effects. A mid-range accuracy digital photogrammetry system has been developed to provide performance with high accuracy and resolution for laboratory and field measurement. Modifications to hardware and survey procedures have been made to achieve comparable accuracy to an industrial grade photogrammetry system. Calibration surveys have revealed the performance to be approximately one-third the accuracy of an industrial grade photogrammetry system. This system has been deployed in-field to measure the response of residential houses to environmental loads. Results of this programme and a study of residential structures state-wide, will provide information to the mining and insurance industries on expected damage to residential houses due to environmental and vibration induced loads. Possible applications for photogrammetry in earthquake engineering include surveys of critical infrastructure such as hospitals and other major infrastructure such as dams and bridges.

ACKNOWLEDGEMENTS

The authors would like to thank the following:

- The project industry partner, Terrock Consulting Engineers
- ARC Linkage Grant No. LP0211407
- Department of Geomatics, The University of Melbourne, for assistance in developing the photogrammetry system.
- Victorian Office of Housing, Department of Human Services, for provision of property inspection reports
- Vision Metrology Services for facilitating results using and providing system for industrial grade photogrammetry measurement.

BIBLIOGRAPHY

Brown, J.
V-Stars/S Acceptance Test Results, Geodetic Services, http://www.geodetic.com, 1998

Corvetti, J., Gad, E., Deiss, M. and Wilson, J.
Estimation of Non-Structural Damage in Residential Light Framed Walls, Proceedings of the 2003 AEES Conference, Melbourne

Heath, D., Gad, E., Wilson, J. and Sinadinovski, C.
Experimental and Numerical Testing Techniques to Model Unreinforced Masonry, Proceedings of the AEES 2003 Conference

Kraus, K.
Photogrammetry – Volume 1: Fundamentals and Standard Processes, Ferd. Dummlers Verlag, 1993

Mikhail, E. M., Bethel, J. S. and McGlone, J. C.
Introduction to Modern Photogrammetry, John Wiley & Sons, Inc., 2001
Moore, A., Richards, A., Wilson, J., Gad, E., Page, A., Fityus, S. and Simundic, G.
Structure Response to Blast Vibration, ACARP Ref. No. C9040, November, 2002.
Siskind, D. E., Stagg, M. S., Kipp, J. W. and Dowding, C. H.
Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting, USBM Report of Investigations RI 8507, 1980.
Slama, C. C. (Editor-in-chief)
Manual of Photogrammetry, Fourth Edition, American Society of Photogrammetry, 1980

EDUCATING THE PUBLIC ABOUT EARTHQUAKES

ALLAN L. MCDOUGALL
ESM RFD

AUTHOR:

Allan McDougall completed a Bachelor of Science and Diploma of Education at the University of Adelaide. His experience includes teaching chemistry, being an RAN Attack Class Patrol Boat Commanding Officer and working for the State Emergency Service and Disaster Management Services, with training and operational roles in the State Emergency Operations Centre.

ABSTRACT:

Many Australians fail to take earthquakes seriously and this is in part due to the infrequency of major earthquakes that impact population centres throughout this nation. Educational strategies that promote earthquake awareness are, however, needed and such strategies should focus on empowering communities to respond appropriately.

This paper discusses some strategies for educating the community with respect to earthquakes.

EDUCATING THE PUBLIC ABOUT EARTHQUAKES

1. Introduction

What should the public be told about earthquakes? What can the public do about the advice that is given? What responses from the public would be considered appropriate?

2. Communicating Risk

Depending on the circumstances, risk communication may need to "scare people" whilst in other circumstances it needs to "calm people down". In any event it ought to evoke an appropriate response from the public. However, if the public is to respond appropriately then it needs to be empowered to do so. If the public is informed of an impending hazard threat in a way that does not recommend strategies for personal empowerment, then the likely response from the public will be, to a greater or lesser degree, panic. Even when aiming to scare people, the aim must be to productively scare people rather than cause panic. It could be argued that panic has never, nor ever will be, an effective emergency management tool. Communicating with the public also involves advising, alerting and reassuring as appropriate. Successful risk communication provokes people to an extent commensurate with the hazard threat.

Like it or not, some hazard threats, such as impacts from unfamiliar hazards such as near-earth objects and earthquakes may be subject to the giggle factor and dismissed by some members of the community as lying at the fringes of plausibility.

3. Examples of informing and not informing the public

The aftermath of the Burra Earthquake:

In early February 2001, the Seismology Section of Primary Industries and Resources South Australia was contacted by seismologists from the University of Beijing and advised that, on the basis of an examination of the data obtained from the Burra earthquake of 5th March 1997, an earthquake of Richter Magnitude 6, with an epicentre within a 150 km radius of Burra, would very likely impact sometime towards the middle of the year or even into 2002.

The State Government was informed of this prediction (and predicament!). Looking at a worse-case scenario, behind the scenes, the State Disaster Organisation did a lot of preparatory work re-visiting the State Disaster Plan and conducting exercises to ensure that the State Disaster Organisation would be ready to respond should the earthquake strike. In addition, advice was sought from seismic experts around the world to enable some reliable assessment to be made as to the credibility of those making the prediction. It was considered that little could be done to empower the community, and as a result, the community was not advised of the prediction.
In the fullness of time, thankfully, the prediction was revealed to be false.

Earthquake and tidal wave prediction:

January 19th, 1976 was the day on which, according to an Adelaide housepainter-cum-clairvoyant, Adelaide was likely to meet its doom. He predicted that an earthquake and tidal wave would impact metropolitan Adelaide. There was no scientific basis for his prediction, just a "feeling". Despite the snake-oil flavour to all of this, numerous Adelaide people took heed. State Premier Don Dunstan went to the Glenelg Jetty at the time of the prophesied incident to prove that there was nothing to worry about. The Premier spoke out against the "quite nonsensical hysteria arising from the earthquake and tidal wave prediction."
The Premier went on to say, " There is absolutely no basis for it at all. And I would not make a statement about it because I think it's such nonsense, but for the fact that it has already caused a very great deal of community damage, and is likely to cause more from the reports and complaints that have been made to me. There have been families which have put themselves into debt to move out of South Australia at that time, there are other families who have sold their houses when they couldn't afford to do so. That sort of thing has happened amongst some poorer sections of the community. I'm trying to see to it that there is no more damage, and trying to reassure people that there is absolutely nothing in this at all."

As it turned out, the prediction proved to be a fizzer, and it would have been far better for the public to be told nothing at all.

Y2K threat:

In 1999 it was considered possible that oil refineries might be shutdown by Y2K impacts, with consequent fuel shortages across the community. Behind the scenes arrangements were made for the emergency services to have access to fuel reserves, but the possibility of no fuel was not broadcast to the public. It was considered that informing the public would have resulted in widespread hoarding across the community. Furthermore, given that, in the closing weeks of 1999, the price of a new jerry-can was $69, (a prohibitive price for some who fell into the lower socio-economic group), it was considered likely that there would be instances of large quantities of petrol being stored in plastic beverage bottles, or even in bath-tubs. Giving an opportunity for the community to panic about the threat of fuel shortages could have significantly raised the risks associated with storing fuel inappropriately, and therefore telling the public may have proved counterproductive to the overall quest to maintain public safety. As a consequence no mention was made to the public of such a threat.

MIR Space Station re-entry:

The re-entry of the MIR Space Station, involved a device weighing 140 metric tons of which 40 tons were expected to break into pieces and survive re-entry and impact the earth at near sonic speed. These pieces, some of which were expected to be of the order of 700kg in mass, were forecast to slow down and fall to the Earth's surface, impacting in a well-defined remote area of the Pacific Ocean (some 1500-2000 km southeast of Australia). Pacific Rim and Pacific island nations cautioned their populations to be prepared to take cover when the heaviest artificial object ever in orbit fell to Earth.
According to reports, there were four circumnavigations of the globe during which MIR could be safely brought back to Earth. Of significance to South Australia, if the attempt

were to be left to the fourth and last pass, then this would have been a trajectory that passed over this State. Advice, consistent with that being promulgated elsewhere throughout the nation was broadcast to the public. Such advice included recommending that if people had a basement in their home, then that would be a suitable place to take shelter. A more realistic and pragmatic approach to the threat might have reasoned that, in the event of being in the direct path of a 700 kg piece of white hot metal travelling at near sonic speed, it might have been more worthwhile to have stood outside one's home and at least enjoyed the fireworks for the last split-second of one's life. In reality there was little advice that one could give to empower the community to withstand the impact of this hazard.

This hazard threat seemed to come within the "giggle factor" category for most of the population since despite wide media coverage, it seemed to evoke, at best, blazé responses. Many people seemed resigned to the knowledge that the likelihood of being targeted was very small and that, even if they were, there was little that they could do.

4. Earthquake? – Are you kidding?

What causes communities to either ignore or heed advice and warnings? For many regions of Australia the pre-eminent hazard is bushfire and even though such fires can be counted on to impact many communities throughout the nation each summer, many people living in fire-prone areas nevertheless display a cavalier attitude fire safety. Given the infrequent impact of earthquakes on Australia's urbanised areas and the current hit-and-miss nature of earthquake prediction, it comes as no surprise that communities across Australia see little need for, and therefore show little motivation towards, preparing for earthquake impact. Do we resign ourselves to the avalanche of public apathy or can something be done to improve the situation?

5. Factors that shape community responses to warnings

Differing perceptions of hazard risk:

What appears dangerous to one person may be perceived as safe by someone else and vice versa.

Denial of hazard risk:

A coping mechanism when confronted with the risk associated with a hazard that gives rise to the viewpoint, "If I don't confront the reality, I don't have to take action."

Safety in numbers:

"Well, if my house is going fall down, then there'll be a lot of others buried under rubble as well!"

Procrastination/Apathy/Laziness:

"Not to worry... she'll be right!"

Illusion of immortality:

"It won't happen to me! – that "bullet-proof" attitude that makes young people, in particular, so vulnerable to road accidents.

Prejudice towards, or vote of no confidence in, the bearer of the message:

"That's just typical of the Council! Who was the bright spark that got them on the earthquake band-wagon? Why can't they just stick to picking up the garbage each week... they can't even do that properly yet!.."

In reality, too many people:

- are often careless about important things;
- often leave things until the last moment;
- are lethargic, in some areas at least;
- are often initially enthusiastic about an issue, but the enthusiasm wanes after a while. (Only sometimes does the reverse occur, i.e. indifference changes gradually into enthusiasm);
- are not prompted by reason so much as by other factors;
- are not pro-active but instead, re-active, and,
- can't be warned or told anything.

Given these factors, concerted efforts to counter-act human nature are almost invariably time consuming and expensive.

6. Targeting schools - a mitigation strategy that would be effective

It is proposed that, recognising as a given, such failings of human nature as referred to in para 5 above, communities can be coaxed into preparing for earthquakes by subversive, means. One window of opportunity comes via school education and it could be argued that the most appropriate setting is within primary schools. As many parents would attest, when secondary school students arrive home from school and are asked, "What happened at school today?" likely as not, given that many secondary students relegate their parents to the "un-cool" category, the answer will be "nothing much," supported by a shrug of the shoulders. When the same question is posed to primary school students, whose lives still bear some signs of childish innocence, the answer is likely to be quite different. There may be a detailed regurgitation of what was learned at school and an insistence that a poster made on the subject be attached to the kitchen refrigerator. As well, primary school children may challenge their parents to adopt a mitigation measure recommended during the school lesson and harangue their parents until it is implemented.

Getting into the primary school curriculum with empowerment rather than merely ghoulish, frightening material would be effective since disaster-related subjects are attractive to teachers because of their ability to capture the attention of students. On the other hand, teachers are reluctant to teach subjects for which they lack the resources, so in order for an earthquake readiness programme to be successful in schools, it must be supported by a generous supply of teacher and student resources.

At secondary school level, a continuation of the earthquake message could be further consolidated in the form of examples within existing subjects. To an extent this occurs already, although it could be argued that opportunity exists to make the penetration of the earthquake topic more thorough. In the case of mathematics, for example, it is obviously easier to teach a mathematical concept if a practical application can be utilised to illustrate the concept's relevance. If logarithmic models are being discussed, why not have the Richter scale included in that discussion? When resonance is being discussed in the physics lesson, why not include the resonance of buildings during earthquakes? With imagination, many aspects relating to earthquakes could enhance a range of subjects including mathematics, physics, chemistry, geography, history, and the social sciences to an extent far greater than at present. This would require pro-active links between seismologists and textbook writers.

7. Educating the community at large about earthquakes

Given the fickleness of public interest, many agencies know that if their cause is to strike a chord with the community, then their message needs to be inserted through the small window of opportunity immediately following the broadcast of a relevant event. An education programme, spasmodic admittedly, could capitalise on overseas earthquakes by ensuring that a ready-to-go educational package is pro-actively presented to media outlets. Approaching the media rather than vice versa would ensure that the message promulgated to the public, rather than being an impromptu affair, would be as effective as possible. Such information provided to the public needs to:

- be unambiguous;
- avoid as much as possible the use of unfamiliar terms;
- be couched in a way that makes it understandable;
- be authoritative;
- empower those who hear the message;
- be able to stir the recipient to action within an appropriate time frame, and
- be conveyed by a communication method that is relevant.

8. Summary

An essential component of educating, advising, warning and reassuring the public about earthquakes is empowerment. Individuals need to be equipped with the skills to apply strategies that are appropriate to meet the threat, and imaginative methods may need to be explored in order to do this effectively. This paper raises the argument that unless individuals can be empowered to take mitigatory action, then it is unwise to inform them of the hazard threat. In the interests of public safety, however, those with knowledge in the field of earthquakes are nevertheless obliged to do what they can to prepare communities as best as possible for earthquake impact. Perhaps one of the more successful ways of enhancing public safety could arise from a more imaginative interaction between seismologists and the school curriculum.

NARROW-BAND ANALOGUE TO BROAD-BAND DIGITAL ACCELEROGRAMS ON THE WEB – THE AUSTRALIAN EXPERIENCE

VAUGHAN WESSON, GARY GIBSON, KEVIN MCCUE AND CHARLES BUBB

AUTHORS:

Vaughan Wesson: Vaughan is Technical Services Manager at Environmental Systems and Services Pty Ltd in Melbourne. He has over 25 years experience in seismology and the development of seismic instrumentation. He is the chief architect of the Kelunji series of seismic recorders.

Kevin McCue: Kevin is Director of the Australian Seismological Centre, a small Canberra-based engineering seismology consulting firm, and Adjunct Professor at CQU. He was co-founder of AEES with Charles Bubb and David Rossiter and has been on the committee as secretary, national delegate to IAEE and Newsletter editor since its inception.

Gary Gibson established the Seismology Research Centre in 1976 and is an Honorary Research Associate at Monash University. He is Honorary Secretary of the International Seismological Centre. His interests are in observational seismology and its practical applications.

Charles Bubb: Now retired, Charles graduated from UWA and Imperial College and rose to become Director of Engineering at the Commonwealth Department of Housing and Construction. He oversaw many major projects both in Australia and overseas and strongly influenced the shape of both earthquake and wind engineering AS codes and practice. He is the first life member of AEES of which he was co-founder, inaugural president and first national delegate of AEES to IAEE.

ABSTRACT:

This year marks the 25th year since AS2121-1979 was published. The disciplines of Earthquake Engineering and Engineering Seismology in Australia have matured in that time. Our focus is on the development of strong motion data collection and instrumentation in Australia; from a zero data-base and no instruments in 1970, to a significant strong motion data collection and about 100 operational accelerographs in Australia today.

This turnaround was initially driven by engineers on the 1979 earthquake code committee to meet their requirement for spectral data, while precisely timed waveforms was not an issue at that time. Ultimately seismologists took over the development of instrumentation with their need for both near-source and distant, time-stamped observations, thereby providing for both needs. Attenuation studies require ground motion measurements at a wide range of distances from *each* earthquake, so a high density of instruments is needed to ensure that near-field motion is recorded from the *all* moderate to large magnitude events.

Each earthquake demonstrates that there is never enough data - more instruments are still required and the only way to achieve that is with cheaper and more versatile instrumentation, and a committed program for routine maintenance and operation. Today's networks need modern recorders that are cheap, rugged, reliable, flexible, networked and accessible.

The Need For Strong Motion Data

Strong motion is defined (Lee and others, 2003) as ground motion having the potential to cause significant risk to a structure's architectural or structural components, or its contents. Lee and others (2003) define a strong motion instrument as a strong motion accelerograph.

Traditionally, and still in many countries, seismologists used very sensitive seismographs to record distant earthquakes, while engineers used strong motion accelerographs to record motion from nearby events as it affects bedrock, sites, and structures.

Despite improvements in the dynamic range and frequency response of recording systems, no single seismic recording system will cater for all needs. Seismographs will be driven to full-scale at moderate levels of motion, so provide little information about large nearby earthquakes. Ground motion amplitude is used to compute magnitudes, so magnitudes for large events can only be computed at moderate to large distances unless significant effects of local attenuation must be considered.

Accelerographs are more sensitive to high frequency motion than seismographs, and can easily record high amplitude motion. They are excellent for recording strong motion from all large earthquakes, and from nearby small earthquakes.

Earthquake Motion

Earthquake motion is usually recorded at the surface or on a structure. The purpose is to learn more about the earthquake source, the seismic wave travel path, the site response, or the response of the structure.

Figure 1: Source, travel path, site and structure.

Many inter-related parameters are used to describe these aspects of earthquake motion.

Earthquake Source location (longitude, latitude, depth and time), size (fault length, width, area, displacement), orientation (fault type, focal mechanism), dynamics (stress drop).

Travel Path distant related parameters (epicentral distance, hypocentral distance, depth), attenuation related parameters (geometric spreading, absorption, scattering, reflections).

Surface motion free-surface amplification, amplification by soft-surface sediments and by topography, and attenuation of motion by thick layers of soft sediments.

Structure parameters include natural frequency (or period) and damping.

Traditional strong motion applications have emphasised the motion of structures and sites. It is now apparent that strong motion recording has a part to play over all aspects of earthquake motion.

Applications of Strong Motion Data

The simplest application of strong motion data is to measure effects of an earthquake on a structure or site. This can be most significant for the owner of a structure.

If measurements are taken at several points, such as on nearby bedrock, at the foundations of the structure, and at other places on the structure (usually near the top), then frequency dependent site and structure transfer functions can be calculated. These transfer functions illustrate the dynamic properties of the site or structure. Hence motion from small to moderate events can be used to anticipate the motion experienced in a larger event.

Provided a strong motion accelerograph records time precisely (preferably to 0.01 seconds or better), its data can be used in the same way as seismograph data to locate earthquakes. A recorder near to the epicentre will provide very good constraint on the earthquake depth. Provided that waveform analysis software is available, strong motion data can be used for many routine earthquake applications, although not for studies of for small distant events.

Attenuation Using Strong Motion Data

During the early days of seismology, seismologists used sensitive seismographs that would reach full-scale if closer than a hundred kilometres from even moderate earthquakes, while engineers used limited dynamic range accelerographs that could only record larger nearby earthquakes. The dynamic ranges of each was limited, with full scale being only 300 to 500 times the noise level, and there was a large gap between full scale on seismographs and the noise level of strong motion accelerographs.

Each group developed attenuation equations showing the reduction of seismic wave amplitudes with distance, seismologists for magnitude estimation and engineers for earthquake hazard studies. The equations were each self-consistent within the magnitude and distance ranges of the data used to develop them, but it was difficult to relate the two.

Seismologists knew that attenuation varied significantly from place to place, and since moderate and large magnitudes could only be calculated from data recorded at considerable distances, these magnitudes would be significantly affected by variations in local attenuation. Magnitude equations were developed for Western Australia, South Australia, and eastern Australia that each appeared to be self-consistent.

With digital recording, the dynamic range increased, and the gap was eliminated (although seismometers are still much more sensitive than accelerometers). As earthquake monitoring improved in the 1980's and 1990's, it became apparent that the early attenuation equations could not be extrapolated beyond the magnitude and distance ranges for which they were defined, or else anomalous results would be found. In Victoria, the SRC found that magnitude estimates decreased as the distance to an earthquake decreased, while in South Australia the traditional magnitude functions would give magnitudes that decreased with increasing distance.

Allen (2004) developed magnitude scales that do not vary with distance, and which can be compared from place to place. That is, an earthquake of magnitude 3.0 in South Australia would be the same as a magnitude 3.0 in Victoria, according to the same definition. This is done by taking into account variations in local attenuation. Local attenuation is determined by measuring earthquakes over a wide range of distances, from near the epicentre to hundreds of kilometres. Since seismographs still exceed full scale near to even small earthquakes, this requires a dense array of strong motion accelerographs.

AS2121

Background History of Strong Motion Monitoring in Australia

The unprecedented damage caused by the Meckering WA earthquake of 14 October 1968 prompted engineers in Australia to write the first Earthquake Code for this country, published as AS1170.4. The code writers knew little about Australian ground shaking, its frequency, amplitude and duration, data essential for constructing new earthquake-resistant buildings. They lobbied the Australian Government to install some accelerographs in seismically active areas of Australia to compare spectra and attenuation with those from equivalent sized earthquakes overseas, principally California where such instrumentation had been operating since late 1932.

Three New Zealand-made analogue MO-2 recorders were installed in southwest Western Australia during 1971 (Gregson, 1972), three years before any were installed in Eastern Australia. Paradoxically the first Australian accelerogram was recorded in Eastern Australia on a farm near the Oolong NSW railway siding about 50 km from Canberra where a nearby magnitude 3.1 earthquake triggered the US-made Kinemetrics SMA-1 triaxial analogue recorder there on 23 November 1976 (Smith and McEwin, 1980).

These analogue recorders were quite insensitive, triggering when the intensity reached about MMI V (by observation) or PGA reached 0.01g (nominal). Recorded time was relative to the trigger time; real time was one parameter not considered necessary by the engineers or equipment designers at the time. The West Australian Water Authority and South Australian Government purchased several MO-2 analogue recorders in the late 1970s, but the South Australian instruments were never triggered by an earthquake.

From the mid 1980s, several types of digital accelerograph were chosen to replace the analogue recorders that had proved difficult to maintain, and whose records were even more difficult to process. In WA the MO-2s were replaced by the US-built A700. The six A700 recorders purchased by the Queensland Government never worked. In Eastern Australia, the Melbourne-made Yerilla recorder was chosen which, like the A700, used magnetic tapes for data storage. The Yerilla was soon replaced with the next generation Kelunji that introduced solid-state memory and more functionality.

Local manufacture solved several problems with overseas equipment. Recorders returned to the US or New Zealand for repair could take more than a year for the round trip, and customs clearance added to the time delays. Local design and support meant that faults could be quickly fixed, and feedback incorporated into equipment and software upgrades.

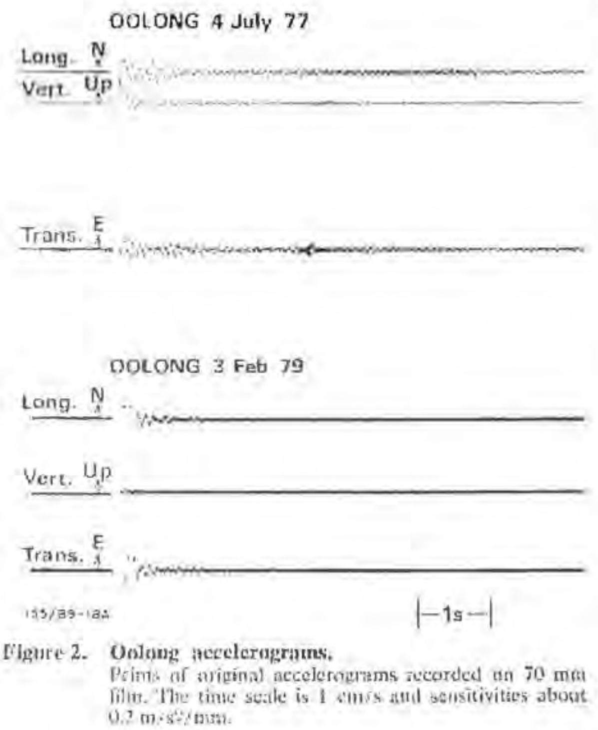


Figure 2. **Oolong accelerograms.** Prints of original accelerograms recorded on 70 mm film. The time scale is 1 cm/s and sensitivities about 0.2 m/s²/mm.

Figure 2: Reproductions of analogue accelerograms recorded at Oolong NSW, 1977 & 1979 (after Smith and McEwin, 1980).

Subsequent recordings at the same site in 1984 (Figure 3) show that the maximum part of the coda had been lost during start up as Smith and McEwin suspected.

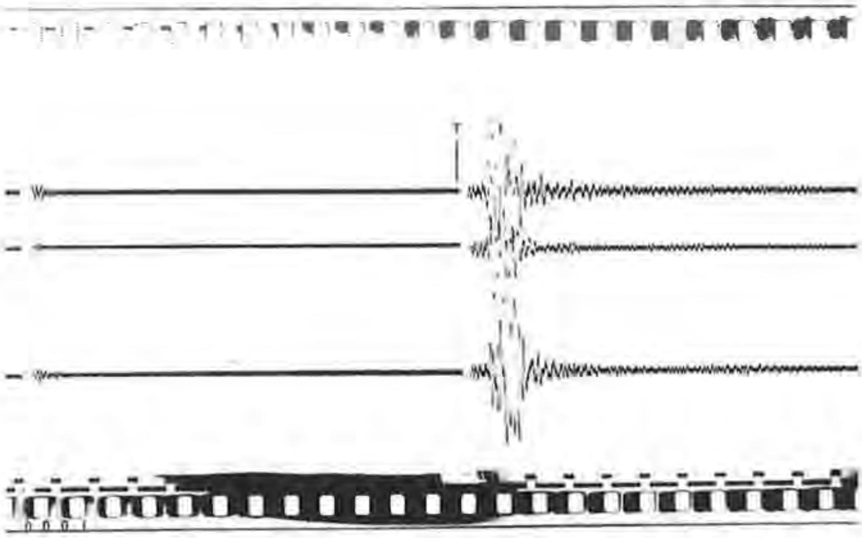


Figure 3: Oolong 9 August 1984 accelerogram on 70 mm wide film. SMA-1 recorder. T is the trigger time. The central trace is the vertical component, the lowest the time mark trace, offset twice per second. The SMA-1 used here had a 1/2 g transducer (i.e. ~38mm/g) sensitivity, and recorded PGA (horizontal) was ~ 0.25g.

The timely societal focus on risk and the advent of digital recording led to monitoring of assets for utilities such as dam owners by Seismology Research Centre (now part of ES&S). Consequently, many more accelerograms were collected, mostly near small earthquakes or at large distances from moderate to large earthquakes. By now, accurate timing was deemed necessary by the users and designers, mostly seismologists, so that the instruments could be used not only for ground motion and structural response monitoring, but also for earthquake location.

Modern Digital Accelerographs

The digital accelerographs first introduced in the 1980's, and significantly improved each decade since, provide features not possible with the earlier analogue instruments (McCue and others 1988, Wesson and Bricker 1996a, Wesson and Bricker 1996b). The first is a much larger dynamic range – that is the range from the smallest to the largest useful signal. For most analogue instruments, this was a range not significantly greater than 100:1, which means that the instruments could only usefully record a range of two units of magnitude at a given distance (e.g. ML 5 to ML 7). By comparison, modern digital instruments have a dynamic range of greater than 1,000,000:1 meaning that they can record earthquakes covering a range of six magnitude units for a given distance (e.g. ML 1 to ML 7). In practise this means that they will record many more earthquakes than the earlier instruments. In addition to a larger dynamic range, modern accelerographs are capable of recording a wider range of frequencies, typically from DC (0 Hz) up to 50Hz or 100Hz.

The second major feature of digital accelerographs is the ease with which the recorded signals can be further processed. In many cases, the recorded signal is used as input to an engineering design where a (digital) computer must process the data. With the analogue instruments this meant digitising the analogue signal leading to many numerical problems, particularly at longer periods (greater than 0.25 seconds), and major problems on records where the traces overlapped. With digital recordings, it is much easier to process the recorded signal to determine the time history of ground velocity or displacement.

The third major feature of modern accelerographs is data accessibility. Modern instruments can be connected directly to the Internet, allowing them to send data to a processing centre in close to real time, and providing suitably authorised operators with immediate access to the data. This can provide a significant reduction in the cost of operating instruments, and makes it easy to monitor the state-of-health of instruments to ensure that they are operating correctly. For applications where immediate data access is less important, this ease of communication may be offset by increased reliability and data capacity allowing visits for manual servicing at infrequent intervals (perhaps one or two per year), or after key earthquakes.

The fourth and most significant feature is the lower capital cost of modern accelerographs. Technological advances over the last few decades have significantly reduced the cost of high precision motion sensing devices and the electronics associated with these necessary to provide a complete accelerograph. In absolute dollar terms, a typically modern accelerograph is only about one third the cost of a comparable instrument twenty years ago. Allowing for inflation over that time, the difference in real terms is considerably greater.

Australian Accelerograms

As mentioned above, there were only a handful of accelerograms recorded in Australia on the early analogue accelerographs. Similarly, the early digital accelerographs purchased overseas provided only a small number of recordings. The vast majority of accelerograms recorded in Australia have been on the Yerilla and Kelunji instruments manufactured by ES&S and its predecessors. These used sensors manufactured by either Sprengnether Instruments in the USA or Guralp Systems in the UK.

A complete catalogue of Australian accelerograms has not yet been gathered, but we estimate that about a thousand accelerograms have now been recorded in Australia. These are from earthquakes ranging in magnitude from about ML 0 up to ML 8.2 and for distances ranging from about one kilometre up to thousands of kilometres. The highest acceleration recorded in Australia to date was just under 1g from the magnitude ML 4.2 earthquake that was the largest event in the swarm of events near the town of

Eugowra in central NSW in 1994. The site was less than two kilometres from the hypocentre of the event.

Conclusion

Important advances have been made in the recording of strong earthquake motions in Australia over the last few decades. These advances have made it possible to record more events, with better quality recordings at lower cost than ever before. These recordings are the key raw data required for all seismic hazard estimates performed in Australia.

References

Allen, T.I., 2004: *Spectral attenuation and earthquake source parameters from recorded ground motion*, PhD. Thesis, Monash University.

Gregson, P.J., 1972 - Mundaring Geophysical Observatory Annual Report 1971. BMR Record 1972/48.

Lee, W.H.K., Kanamori, H., Jennings, P. and Kisslinger, C., 2003 - (Editors) Earthquake and Engineering Seismology, Part B. Academic Press.

McCue, K., Gibson, G. & Wesson, V., 1988: *Intraplate Recording of Strong Motion in Southeastern Australia*, Proc 9th World Conf on Earthquake Eng, Tokyo, July 1988

Smith, R.S. & McEwin, A.J., 1980 – Earthquake accelerograms and attenuation of seismic waves at Oolong, New South Wales. BMR Jnl of Aust Geol. & Geophys, 5, 63-67.

Wesson, V.C. & Bricker, C., 1996a: *Power management for a portable digital seismograph*, "Portable by Design" Electronic Design Conf, March 1996

Wesson, V.C. & Bricker, C., 1996b: *Implementing PC cards in an embedded system*, "Portable by Design" Electronic Design Conf, March 1996, Santa Clara

MONITORING OF EARTHQUAKES IN THE FLINDERS RANGES, SOUTH AUSTRALIA, USING A TEMPORARY SEISMOMETER DEPLOYMENT

PHIL CUMMINS, CLIVE COLLINS, AND ANGELA BULLOCK
GEOSCIENCE AUSTRALIA
DAVID LOVE
PRIMARY INDUSTRIES AND RESOURCES, SOUTH AUSTRALIA

AUTHORS:

Phil Cummins is leader of the Earthquake Hazard and Neotectonics Project at GA. He oversees GA's earthquake monitoring activities and earthquake hazard research.

Clive Collins, is manager of the Earthquake Hazard and Neotectonics Project at GA.

Angela Bullock is the senior earthquake data analyst at GA.

David Love is senior seismologist with PIRSA, and has been managing the network in SA since 1986.

ABSTRACT:

We describe the use of a temporary seismometer deployment to monitor local earthquakes in the Flinders Ranges, South Australia. 16 seismograph stations were deployed over a 200 x 100 km area, which is one of the most seismically active regions in Australia. The instrumentation consisted of short-period and broadband Guralp seismometers combined with Reftek and Kelunji data loggers, which sample data continuously at 100-200 sps. Analysis of data from the period Sept.-Dec., 2003, resulted in the determination of hypocentres for over 175 earthquakes, most of which could not be located using PIRSA's permanent network. 54 of these earthquakes had depths resolved at 10 km or greater, and the proportion of deep events appears to increase from the southern to the northern part of the Flinders Ranges. The largest earthquake, $M_L \approx 4$, occurred near Hawker on 22 November, 2003, and has a depth of 17 ± 2 km, and a well-resolved normal focal mechanism.

INTRODUCTION:

Compared to the rest of continental Australia, the Flinders Ranges region of South Australia stands out not only because of its high topographic relief, but also because of its high seismicity and high fault density. The high density of faults combined with their relatively high Quaternary slip rates (Sandiford, 2004) indicate that the Flinders and Mt. Lofty Ranges comprise a region of pronounced neotectonic activity. Applying the accumulating body of neotectonic evidence to an assessment of earthquake hazard in South Australia requires determining what relationship exists, if any, between the earthquakes and faults. If it can be shown that the seismicity tends to cluster along faults, then neotectonic and paleoseismological studies focused on these active faults can help constrain the size and frequency of earthquakes. If there is no evidence of clustering, then an assessment of earthquake hazard will rely more heavily on the very short history of recorded earthquakes.

At present, the low accuracy of earthquake hypocenter determination makes it difficult to tell whether the events cluster along major fault lines. To address this question we conducted a dense deployment of seismometers to improve the precision with which earthquakes are located. We carried out a deployment of about 18 stations, with approximately 30 km inter-station spacing, which exploited some of the permanent stations in the region (Fig. 1). Over the 2-year course of the deployment we hope to record 300-600 locatable earthquakes. Although the main scientific goal of the experiment is to obtain precise hypocentre estimates, the largest events will also be used to assemble composite record sections, whose interpretation should yield an improved velocity structure, which can then be used to obtain an iterative improvement in the accuracy of hypocentre determinations. This improved velocity model can also be used to improve the accuracy of existing seismicity catalogues as well as future hypocentre determinations using the permanent seismograph networks.

The waveform data recorded by this experiment will also be used to establish a dataset for ground motion studies to assess the propagation characteristics of local earthquake signals in the SA region. Because so few data exist which can be used to infer these characteristics, all current models for seismic risk in Australia rely on ground motion models obtained elsewhere (typically eastern North America). Since the signal propagation characteristics are one of the most important components of a seismic risk analysis, use of inappropriate ground motion models may seriously bias seismic risk estimates. It is thus hoped that the data from this experiment will provide the basis for an unbiased seismic risk estimate for South Australia.

Finally, in contrast to the sparse coverage provided in South Australia by the permanent seismograph networks, the data from the relatively dense deployment of seismometers proposed here should provide enough azimuthal coverage of earthquakes in the Flinders Ranges to obtain well-constrained focal mechanisms estimates. This will not only aid in the identification of active faults, but will also provide a basis for better constraining the regional stress field. While the direction of maximum horizontal compressive stress in South Australia is thought to be roughly EW, there is considerable uncertainty in this estimate (azimuth 83°±30°). Furthermore, the concentration of seismicity in the Flinders Ranges suggests that it may be a low-strength feature capable of modifying the regional tectonic stress field, and the focal mechanism data obtained by the experiment proposed here may be used to test this hypothesis.

THE FLINDERS RANGES SEISMOMETER DEPLOYMENT:

The plan for a high-density seismometer deployment in the Flinders Ranges envisioned a network of about 20 stations with approximately 30 km station spacing distributed over an approximately 200 x 100 km area centred on the northern Flinders Ranges. Eventually, 16 stations were actually deployed (Fig. 2). All of these stations are sited on or near rock outcrop, and are using continuous 100sps (in a few cases 200 sps) recording. At 8 of the stations, Australian National Seismic Imaging Resource (ANSIR) instrument packages are deployed, consisting of Reftek data loggers (four 16-bit R72A-02s and four 24-bit R72A07s) and Guralp CMG-40T sensors. At the remaining 8 stations, Kelunji + Guralp CMG40T-1 seismometers were used.

The Reftek data loggers are manufactured in the USA and have been used in Australia since the 1990's to record broadband data from distant earthquakes, primarily for tomography and receiver function studies. The Kelunji is a data logger manufactured by Environmental Systems & Services, Melbourne, which is commonly used for recording local Australian earthquake data. To our knowledge, this project was the first attempt to use both Refteks and Kelunjis together in a single experiment (see Fig. 1). Also, the project was the first in Australia to use the Reftek recorders at high sample rates of 100 and 200 sps (the latter requiring a 2nd 60W solar panel to cope with more frequent disk access), and to use the Kelunjis in continuous recording mode as part of a temporary deployment.

Figure 1. Comparison of deployment styles for Kelunji (left, station FR03) and Reftek (right, station FR05) equipment. Both sites are east of Hawker, SA, near the eastern margin of the Flinders Ranges. Seismometers are buried in either case.

Each station is visited at approximately 6-weekly intervals, during which time it typically produces about 2 Gbytes of data (4 Gbytes for the 200 sps stations), so that the total data rate is over 20 Gbytes/month. Thus far, both types of instrumentation have performed well, with a data return of approximately 90%. Both Kelunji and Reftek data are converted to CSS3.0 format, and processed using the Antelope software package for environmental monitoring. STA/LTA detectors are run on high-pass filtered data streams, and events are automatically associated using a suite of P-wave travel times calculated using the SH01 model (Shackleford & Sutton, 1981) on a 3-D grid encompassing the region of the deployment. Wider grids are also used to identify regional and teleseismic events. Finally, these events are manually scanned to discriminate local earthquakes from teleseismic events and quarry blasts, and refined estimates of local earthquake hypocentres are obtained using both P-wave and S-wave picks.

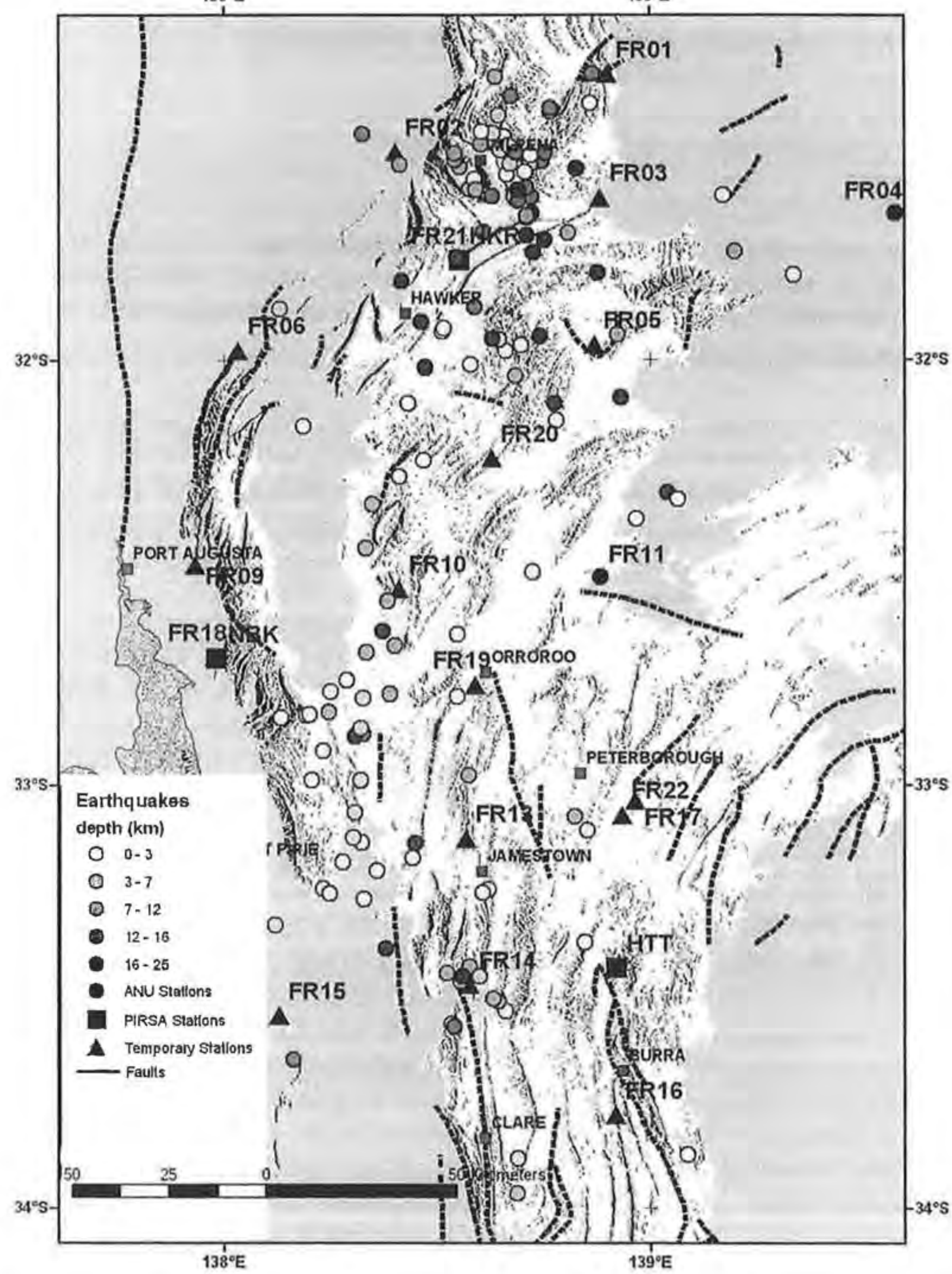


Figure 2. Hypocenters of earthquakes recorded by the Flinders Ranges temporary deployment from Sept-Dec 2003.

EARTHQUAKES RECORDED IN LAST QUARTER OF 2003:

For the period 28/9/03 to 31/12/03, hypocentres for 194 local and regional earthquakes were determined (Fig. 2). A comparison of the GA and Adelaide (ADE) catalogues (without regard to position) showed that out of the first GA list of 175 local events:

8 listed as quarry blast by ADE
47 located by ADE
10 events from Innamincka sequence (induced by well injection)
39 not located by ADE because too few stations
69 ignored or not recognised by ADE in scanning stage
(3 possible errors, and smaller quarry blasts)
2 uncertain

Within the region of interest, ADE located 51 events, compared with 123 events located by GA. This is an additional 140%. Within the region of interest, ADE located 51 events, compared with 123 events located by GA. This is an additional 140% (GA's permanent network only located 5 of these events).

Within the region of interest, both data sets show generally the same distribution of activity. The GA set shows a considerable increase in activity north of Hawker. Within this area ADE has been unable to locate smaller events. In both sets there is a sudden reduction of activity going north between Oraparinna and Blinman. There is a slightly less active area in the Ranges from Wilmington to Hawker, and slightly more activity between Wilmington and Spalding.

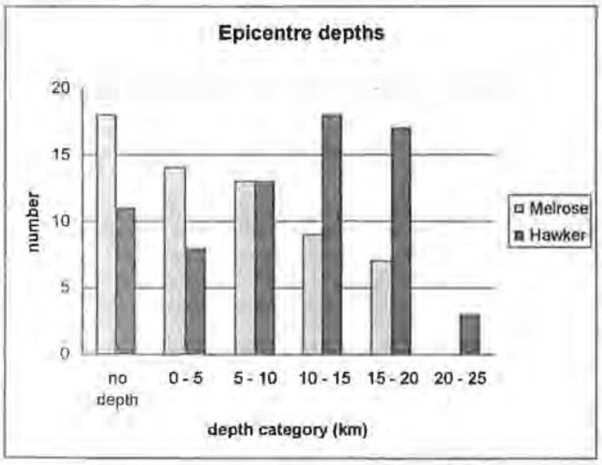


Figure 3. Histograms of the number of earthquakes as a function of depth, with the northern (Hawker) compared to the southern (Melrose) Flinders Ranges.

The GA data set does not show clear lineations relating to faults, however this may still change as more data becomes available and data quality is investigated in more detail.

After removing events for which a depth was not calculated, the Hawker area shows a predominance of events between 10 and 20 km, with a few over 20kms (Fig. 3). By contrast the Melrose area shows a predominance of depths under 10 km with none over 20km. Despite the inaccuracies inherent in the broadly spaced permanent network, these 2 plots show considerable similarity to those in Greenhalgh et al (1994). After more data are available and a review of quality, it will be interesting to see if the same result still applies. In particular, the high proportion of hypocenters with depths > 10 km may suggest that the depth to brittle-ductile transition in the Flinders Ranges is deeper than has been inferred from heat flow data (Pulford & Braun, 2004).

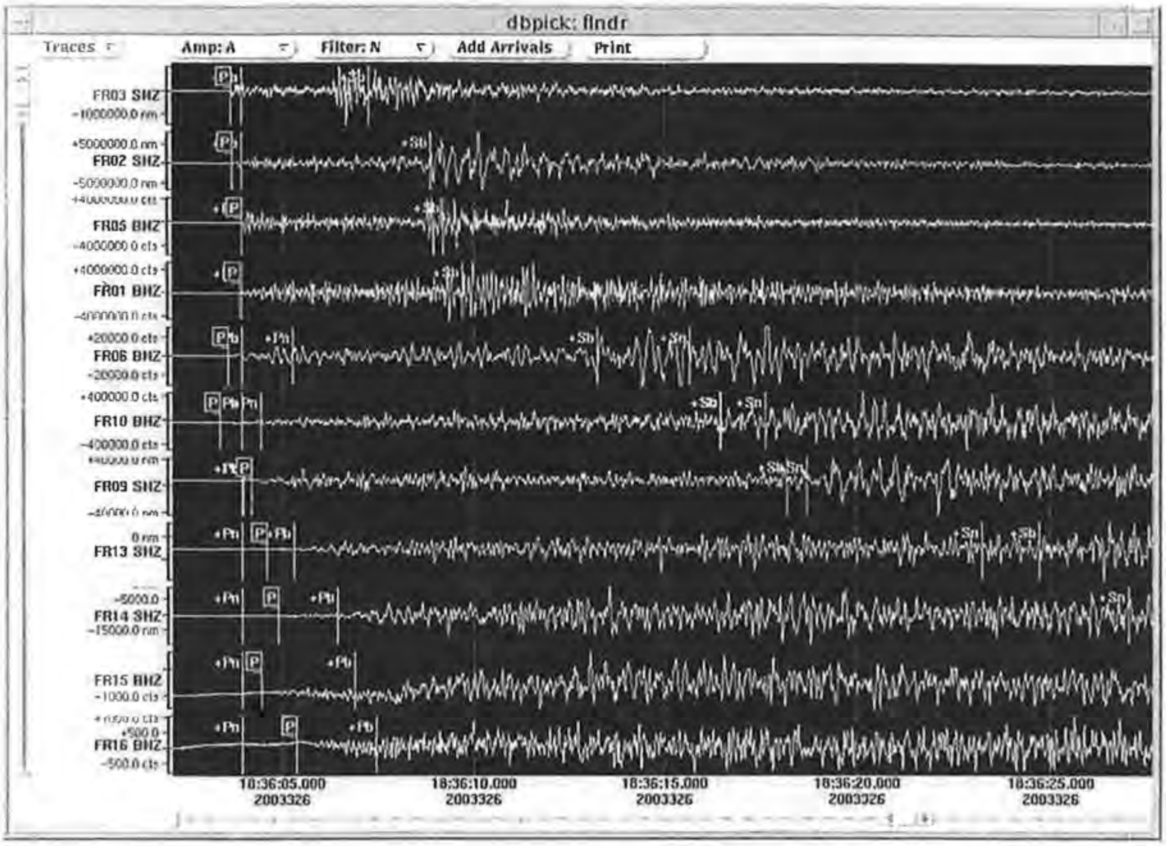


Figure 4. Vertical-component waveforms recorded during the 22 November, 2003 ML 3.9 earthquake near Hawker, SA.

THE HAWKER ML 3.9 EARTHQUAKE OF 22 NOVEMBER, 2003:

The largest earthquake recorded during the course of the experiment, which was widely felt near Hawker, SA, was the ML 3.9 (GA's original estimate was ML 4.2) Hawker earthquake of 22 November, 2003. This earthquake was widely felt in the Hawker region, and it was well recorded at over 11 stations (Fig. 4) of the temporary seismometer deployment. Most significantly, the depth was determined to be 17 km and the focal mechanism (Fig. 5) was normal. Such normal faulting events have occurred in the Flinders Ranges in the past, but their mechanisms and depths have been poorly constrained.

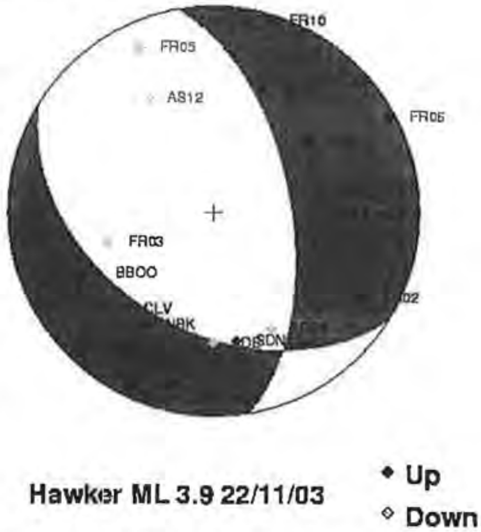


Figure 5. Hawker event focal mechanism.

The extensional focal mechanism calculated from the Hawker earthquake raises important questions about the stress in the area, and its heterogeneity. This was the largest earthquake of the year, the depth of 17± 2 km was moderately well determined and the mechanism well constrained. Such normal faulting events may have occurred in the Flinders Ranges in the past, but their mechanisms have been very poorly constrained. Given the number of other epicentres at considerable depth, it should be possible over the course of the experiment to construct a number of other focal mechanism solutions, which may provide important constraints on the stress field.

CONCLUSIONS:

Geoscience Australia and Primary Industries and Resources, SA are collaborating in a temporary seismometer deployment to monitor earthquake activity in the Flinders Ranges, SA. The 16 stations deployed over an area of approximately 200 x 100 km with 30 km station spacing are using continuous, 100-200 sps recording of 3 component seismometers. For the period Sept.-Dec. 2003, these data have resulted in hypocenter determinations for over 175 local earthquakes, most of which were not resolved by the existing, permanent seismograph network.

Over 30% of the earthquakes located so far occur at depths greater than 10 km, with the proportion of deep events increasing from south to north in the Flinders Ranges, and a few events appear to have focal depths greater than 20 km. This suggests that the depth to brittle-ductile transition in the Flinders Ranges may be deeper than has been inferred from heat flow data (Pulford & Braun, 2004).

The largest earthquake recorded during the course of the experiment was the ML 3.9 Hawker earthquake of 22 November, 2003. This earthquake had an estimated depth of 17 km, and a well-constrained normal focal mechanism. Since the stress regime in this part of Australia is widely thought to be thrust (Hillis & Reynolds, 2000; Burbidge, 2004; Pulford & Braun, 2004), and the only evidence of Quaternary fault movement is attributed to thrust faults (Sandiford 2002), the occurrence of normal faulting earthquakes may have interesting implications for the heterogeneity of stress and the nature of the response of the crust to the regional stress field.

Finally, we note that much work still needs to be done, not only in analysing the more recent data recorded during the course of the experiment, but also in more sophisticated analysis of the 2003 data. In future we hope to model these data using synthetic seismograms, to infer source properties such as stress drop and faulting mechanism. Also, it is hoped that eventually these data can be used as a basis for developing a spectral attenuation model for ground motion in South Australia.

REFERENCES:

Greenhalgh, S.A., Love, D., Malpas, K. & McDougall, R. (1994). South Australian seismicity, 1980-1990, Aust. J. Earth Sci., Vol 41, pp 483-495.

Burbidge, D. (2004). Thin-plate neotectonic models of the Australian plate, J. Geophys. Res., in press.

Hillis, R., and Reynolds, S. (2000). The Australian stress map, J. Geol. Soc. London, Vol 157, pp 915-921.

Pulford, A., and Braun, J. (2004). Rheological Model of Australia, in preparation.

Sandiford, M. (2002). Neotectonics of southeastern Australia: linking the Quaternary faulting record with seismicity and in situ stress, Geol. Soc. Australia Special Publication, *Evolution and dynamics of the Australian Plate*, Vol 22, pp 101-113.

Shackleford, P. R. J., and Sutton, D. J. (1981). A first interpretation of crustal structure in the Adelaide Geosyncline in South Australia using quarry blasts, J. Geol. Soc. Australia, Vol 28, pp 491-500.

SHAKE TABLE TESTING OF UNREINFORCED AND LIGHTLY REINFORCED U-SHAPED ADOBE-MUDBRICK WALL UNITS

DOWLING, D.M., SAMALI, B., AND LI, J.
UNIVERSITY OF TECHNOLOGY, SYDNEY

AUTHORS:

Dominic Dowling is a PhD research candidate at the University of Technology, Sydney. His research is focused on methods to improve the seismic resistance of adobe-mudbrick houses, with a specific interest in low-cost, low-tech solutions for developing countries. Dowling was awarded an AEES Earthquake Engineering Research Scholarship in 2003. <dominic.m.dowling@uts.edu.au>

Prof. Bijan Samali is the current Head of Infrastructure and the Environment at the University of Technology, Sydney. Prof. Samali is a recognised authority on structural dynamics, including wind and earthquake engineering with special interest in structural control, dynamic measurement and analysis of buildings and bridges. <bijan.samali@uts.edu.au>

Dr Jianchun Li is a senior research fellow / senior lecturer in the Faculty of Engineering at the University of Technology, Sydney. Dr Li has over 18 years extensive research experience covering a variety of disciplines, including aeronautical, mechanical, civil, structural and earthquake engineering. <l.jinchun@uts.edu.au>

ABSTRACT

Traditional adobe (mudbrick) houses are highly susceptible to damage and destruction during seismic events, with vertical corner cracking at the intersection of orthogonal walls one of the major damage patterns. In order to assess the performance of different improvement systems a series of tests are being undertaken at the University of Technology, Sydney (UTS), with support from the Australian Earthquake Engineering Society (AEES). Scale model (1:2) u-shaped wall panels are being subjected to transient dynamic loading using a shake table to evaluate the response to out-of-plane seismic forces. The force-displacement characteristics and failure mechanisms of different structural configurations are being studied to determine the resistance capacity of each system. This paper presents the results of two tests in this phase of the research: the dynamic testing and analysis of a traditional (unreinforced) wall unit and a lightly reinforced (internal horizontal chicken wire mesh) wall unit.

Keywords: Aseismic design; adobe mudbricks; experimental testing and analysis; shake table testing

1. INTRODUCTION

Major earthquakes continue to detrimentally affect millions of people living in mudbrick (adobe) houses around the world. This project aims to evaluate appropriate technologies to improve the seismic resistance of mudbrick houses, focussing on low-cost, low-tech solutions for developing countries. A major component of the research is the shake table testing of u-shaped wall units to assess the dynamic structural response and relative performance of different improvement systems.

2. DESCRIPTION OF SPECIMENS

The predominant failure modes of traditional mudbrick houses subject to earthquake loads are vertical corner cracking at the intersection of orthogonal walls, and mid-span vertical cracking due to out-of-plane flexure (Zegarra et al., 1997, Flores et al., 2001, Dowling, 2004). In order to assess the capacity of different improvement systems to reduce such failure, a series of shake table tests of 1:2 scale u-shaped adobe (mudbrick) wall units (Fig. 1) are being undertaken at the University of Technology, Sydney (UTS). This paper presents the results of two tests: one representing a traditional (unreinforced) structure; and the second with horizontal chicken wire mesh reinforcement laid in the mortar joints (between courses 2-3, 6-7, 9-10, 12-13, 14-15, 16-17, 17-18 and 18-19). In each case a downward restraining force was applied to the tops of the 'wing' walls (acting as in-plane shear walls) to simulate the restraint provided by a continuous wall, and to prevent overturning of the complete unit. This restraint effectively transfers the bulk of the seismic loading to the areas of main interest: the vulnerable out-of-plane long wall, and the corner connections.

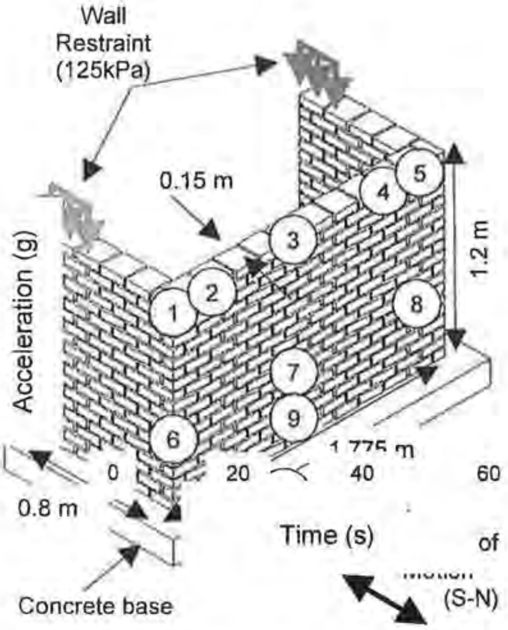


Figure 1. Specimen dimensions and location of LVDT displacement sensors (numbered in circles). Mortar joints = 12-13 mm.

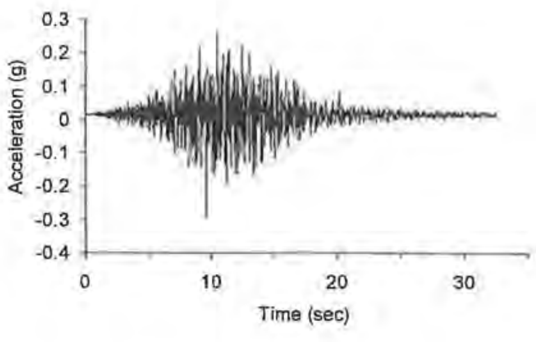


Figure 2. January 13, 2001 El Salvador earthquake, unscaled input time history.

3. INPUT TIME HISTORY FOR SHAKE TABLE SIMULATIONS

The dynamic testing is being undertaken on the 10-tonne capacity, 3m x 3m MTS uni-axial shake table at the University of Technology, Sydney. The shake table is capable of high fidelity seismic simulations. In this study, the input time history from the M_w 7.7 January 13, 2001 El Salvador earthquake is being used (Fig. 2). This input time history was chosen for a number of reasons: Firstly, this earthquake, in combination with a M_w 6.6 earthquake on February 13, 2001, in the same area, caused the destruction of over 110,000 adobe houses (DIGESTYC, 2001). Secondly, the time history has high frequency content, which means stiffer specimens are more likely to experience severe and damaging conditions. Thirdly, the response spectrum of the input time history envelops the proposed design spectrum (for maximum acceleration of 1.5g, presented by Samali et al, 2004) in the high frequency resonant range (6.7 – 15.9 Hz), which is typical of adobe wall units.

Other researchers (e.g. Zegarra et al, 1997; Yamin et al, 2003) have conducted dynamic testing of adobe mudbrick specimens, however they have tended to use the same input spectra for each specimen, discounting the important relationship between the dominant frequency of the input spectra and the natural frequency of each specimen. Such research has produced interesting results from a qualitative perspective, but is limited in depth and quantitative value. Testing at UTS has confirmed the importance of dynamic similitude in shake table testing in order to produce consistent and damaging conditions. Results between different specimens can be more accurately compared because the conditions of dynamic similitude have been applied and the dynamic response of each specimen can be evaluated.

By means of modal analysis, the Frequency Response Function (FRF) of each specimen was identified. For each specimen an 'input spectra time scaling factor' was calculated based on the relationship between the first resonant frequency (natural frequency) of the specimen and the 'target frequency zone' (14.3 – 15.9 Hz) of the input spectra. The 'target frequency zone' was chosen to induce the most damaging (resonance) conditions for the target maximum acceleration response of 1.5g (obtained from the proposed design spectrum, see Samali et al, 2004 for a detailed account of this process). This time-scaling creates the required conditions for dynamic similitude. Table 1 shows the natural frequency and the input spectra time scaling factor for each specimen.

Each specimen was subjected to a series of shake table simulations. Firstly, the unscaled input time history was used, with increasing intensity from 20% to 50% to 100%. The relevant input spectra time scaling factor was then applied for each specimen to obtain the scaled input time history. This scaled input spectra was then used, with increasing intensity until failure (25%, 50%, 75%, 100%).

Table 1. Features of u-shaped adobe wall units 3A and 3C

Specimen	System	Natural Frequency	Input Spectra Time Scaling Factor
3A	Traditional (unreinforced)	29.6 Hz	2.0
3C	Horizontal wire mesh reinforcement	33.0 Hz	2.2

4. RESULTS

For each specimen, the series of shake table simulations using the unscaled input spectra (20%, 50% and 100% intensity) caused no detectable damage to the specimen, due to the natural frequency of the specimens being markedly away from damaging ground motion frequencies. This outcome confirms the need to apply an appropriate input spectra scaling factor in order to produce damaging resonance conditions in the specimen.

Unreinforced specimen 3A failed in a sudden, brittle manner when subjected to the time-scaled simulation at 75% intensity (Fig. 3). The inclusion of horizontal reinforcement (specimen 3C) provided significant ductility to the structure, which performed significantly better (minor hairline cracking) when subjected to the time-scaled simulation at 75% intensity (Fig. 4). Figure 5 shows the relative displacement at the mid-span of the 'long' wall at 75% intensity (time-scaled). The graph clearly shows the failure of specimen 3A, inducing large relative displacements (up to 50mm), whereas the relative displacements of the unfailed specimen 3C were less than ±6.5mm. Specimen 3C eventually failed when subjected to the 100% intensity, time-scaled simulation.

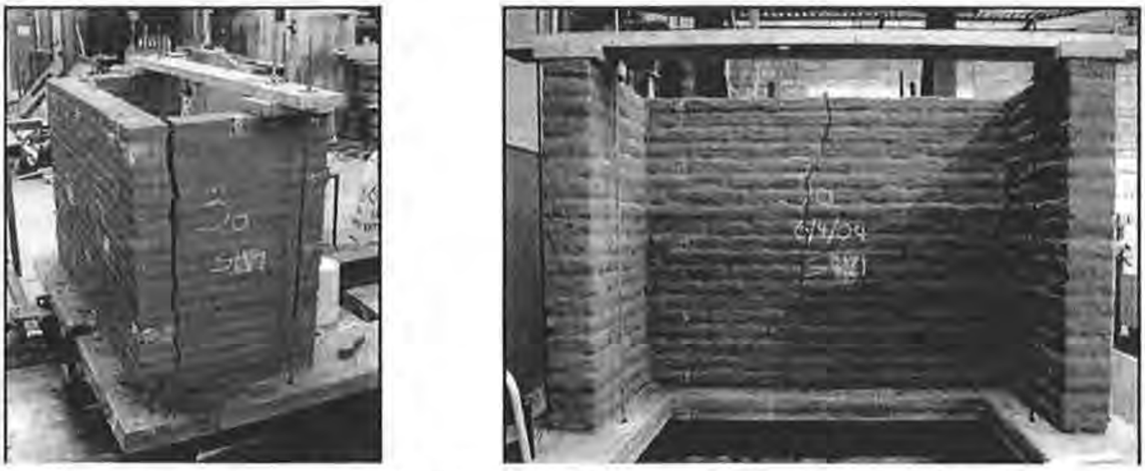


Figure 3a&b. Specimen 3A (unreinforced) failed at 75% intensity of time-scaled simulation.

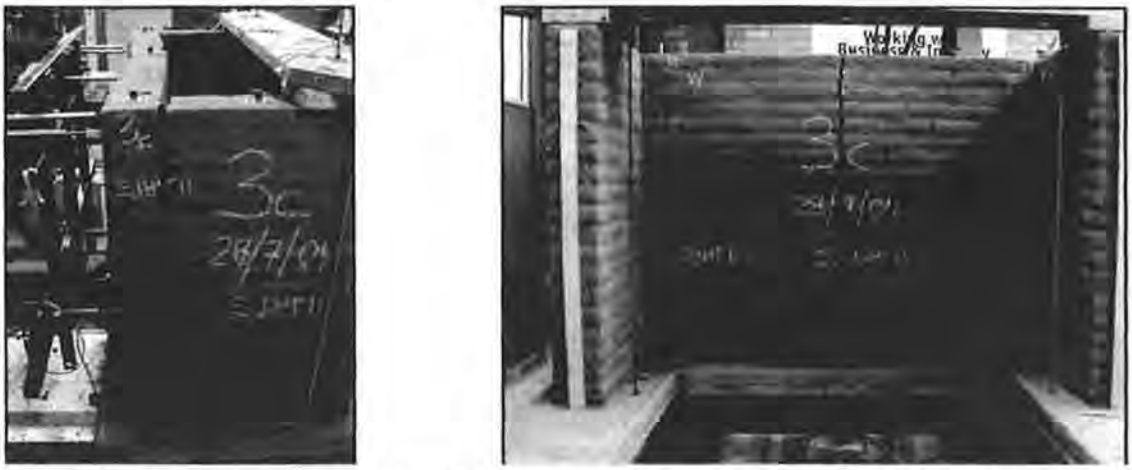


Figure 4a&b. Specimen 3C (with horizontal mesh reinforcement) failed at 100% intensity of time-scaled simulation.

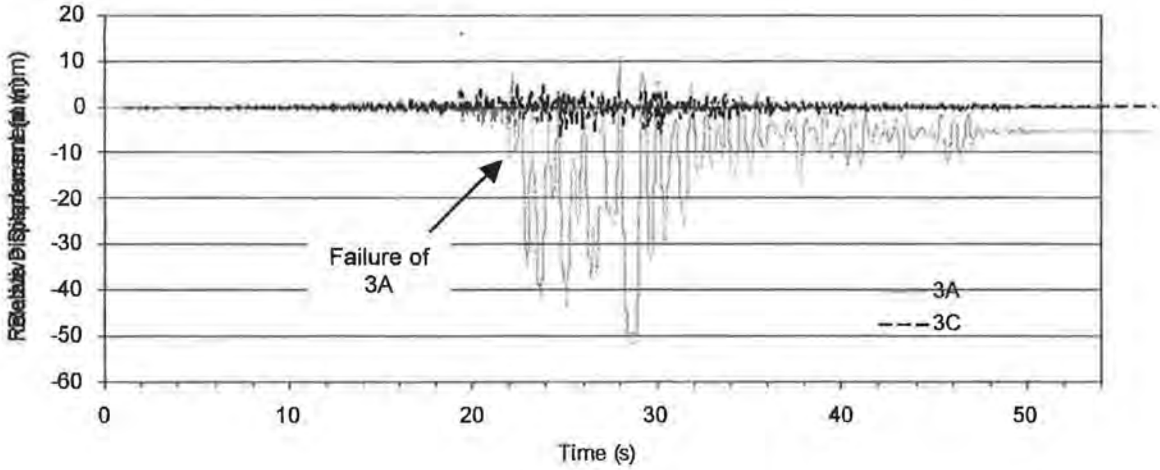


Figure 5. Relative displacement of mid-span of 'long' wall for specimens 3A (grey, solid line) and 3C (black, dashed line) for 75% intensity, time-scaled simulations. (In order to plot and compare results for different specimens each time-displacement graph has been multiplied by the respective input spectra time scaling factor.)

In each specimen the major failure modes (Figs 3 – 4) were vertical cracking in the mid-span of the out-of-plane 'long' wall (induced by flexure), and vertical corner cracking at the intersection of the orthogonal walls due to the large relative displacement between the stiff in-plane shear wall ('wing' wall) and the 'flexible' out-of-plane 'long' wall.

5. CONCLUSIONS

The test results clearly indicate the need to reinforce both the corners and the mid-span of each out-of-plane wall. In each case the specimens exhibited distinct and classic failure patterns, indicative of damage to real houses subject to actual earthquakes. This feature confirms that the selected specimen configuration, boundary conditions and test response spectrum (inclusive of time scaling) are acceptable for this type of experiment. Future testing will consider the costs, complexity and aseismic contribution of a variety of improvement systems including vertical and horizontal reinforcement (e.g bamboo, wire, chicken wire mesh, PVC piping), wider walls and ring beams.

6. ACKNOWLEDGEMENTS

This research is being funded by the generous support of the Australian Earthquake Engineering Society (AEES) and the University of Technology, Sydney (UTS).

7. REFERENCES

DIGESTYC, Dirección General de Estadísticas y Censos (2001) Censo de viviendas afectadas por la actividad sísmica del año 2001. San Salvador: DIGESTYC, Ministerio de Economía, El Salvador.

Dowling D.M. (2004) Adobe housing in El Salvador: Earthquake performance and seismic improvement. In Rose, W.I. et al. (eds), GSA Special Paper 375: Natural Hazards in El Salvador: 281-301. Geological Society of America.

Flores, L.E., Pacheco, M.A. and Reyes, C. (2001) Algunos estudios sobre el comportamiento y rehabilitación de la vivienda rural de adobe, CENAPRED IEG/03/01, CENAPRED México.

Samali, B., Dowling D.M. and Li, J. (2004) Dynamic testing of unreinforced u-shaped adobe-mudbrick wall unit, Proceedings of the 18th Australasian Conference on the Mechanics of Structures and Materials, ACMSM 18, Perth, WA, 1-3 December 2004 (paper accepted for publication).

Yamin L.E., Rodríguez, A.E., Fonseca, L.R., Reyes, J.C. and Phillips, C.A. (2003) Comportamiento sísmico y alternativas de rehabilitación de edificaciones en adobe y tapia pisada con base en modelos a escala reducida ensayado en mesa vibratoria, Proceedings of the 2nd Colombian National Earthquake Engineering Conference, Medellín, November 2003.

Zegarra L., Quiun, D., San Bartlomé, A. & Gisecke, A. 1997. Reforzamiento de Viviendas de Adobe Existentes. Primera Parte: Ensayos Sísmicos de Muros 'U'. Perú: Pontificia Universidad Católica del Perú.