ACARP

Australian Coal Association Research Program

FINAL REPORT

Structure Response to Blast Vibration

C9040

November 2002

ACARP

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STRUCTURE RESPONSE TO BLAST VIBRATION

ACARP REFERENCE NO. C9040

REPORT TO: ACARP

REPORT ON: STRUCTURE RESPONSE TO BLAST VIBRATION

PREPARED FOR: PROJECT STEERING COMMITTEE

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1. SUMMARY

- The structure response of brick veneer test houses in the Muswellbrook and Singleton areas to blasting has been measured and the strength of their structural elements have been determined.

- Analysis has shown that the stresses due to blast vibration that are within currently enforced environmental limits are well below damage levels.

- The vibration levels at which observable damage to houses occurred from blasting compares to the level determined from structural response and strength of materials considerations.

- The structural response effect of ‘natural factors', such as ground movement and rainfall, has been determined and compared to the strength of materials and found to be significant in the formation and propagation of cracks in buildings.

- The type of structural defects observed in the test houses have been observed in reference houses not exposed to blast vibration or mine subsidence.

- The results of this investigation regarding blast vibration levels, structure response and observed damage is consistent with authoritative overseas studies.

- A rational and conservative method has been developed for estimating the dynamic vibration induced strains in houses and comparing these strains with cracking strains of building materials and strains resulting from ‘natural events’ to enable a cause weighting to be determined (if appropriate) without the need of a full structural response investigation.

2. INTRODUCTION

Humans are particularly sensitive to blast vibration and people become concerned about damage to their houses at vibration levels which are well below damage levels.

Because of community sensitivity to airblast and ground vibration, there is a tendency for regulatory authorities, especially those concerned with the environment, to impose increasingly lower limits on blast vibration levels in response to community pressures.

The cost of unnecessarily low limits is a burden that reduces the competitiveness of Australian industry, and this cost must ultimately be borne by the Australian community. The cost of complying also includes the unnecessary sterilisation of resources when establishing and maintaining buffer zones around mines.

There is a tendency for people, after feeling blast vibration, to search for and find defects in their houses, which they consequently attribute to blasting. Natural causes of defects, such as material shrinkage, foundation movement, and temperature variations are generally not recognised because of their gradual nature.

This investigation provides a comprehensive, disciplined investigation into the response of structures to blast vibration, and the causes of defects in buildings that are the subject of blast vibration complaints.

3. PROJECT OBJECTIVES

The objectives of the project are to:

- Provide a disciplined assessment of the effects of both airblast overpressure and ground vibration resulting from blasting in Australian conditions on Australian brick veneer houses.

- Compare these effects with those due to natural phenomena, such as shrinkage and foundation movement.

- Develop a sound methodology for use in the investigation of complaints of blast vibration damage.

This information will then be available for use by authorities when setting airblast and ground vibration limits, and by mining companies and structural investigators when responding to complaints and damage claims.

4. RESEARCH PERSONNEL

The project was carried out by research personnel from the University of Melbourne – Department of Civil and Environmental Engineering, the University of Newcastle – Department of Civil, Surveying and Environmental Engineering, and Terrock Consulting Engineers.

The University of Melbourne team (Associate Professor John Wilson and Dr Emad Gad) were responsible for the dynamic assessment of the effects of blast vibration on the test houses.

The University of Newcastle team (Professor Adrian Page, Dr Stephen Fityus, and Mr Goran Simundic) were responsible for the assessment of the strengths of masonry and other materials used in the structure of the houses, and the assessment of non-blasting factors, such as subsidence and reactive soils.

Terrock Consulting Engineers (Messrs Alan Richards, Adrian Moore and Thomas Lewandowski) were responsible for the installation and operation of monitoring equipment, and for the overall coordination of the project.

5. PROJECT DESCRIPTION

The elements of the research project were:

- Literature search of previously published material on related subjects.
- Measurement of the blast vibration inputs into the test houses.
- Determination of the stresses that the blast vibration induces in the houses.
- Determination of the strength of the structural components of the houses (ie. what stresses can they withstand).
- Assessment of the stresses imposed on houses from non-blasting causes ie. natural events.

Three houses were selected as 'typical' representative brick veneer houses, subject to a range of air and ground vibration from coal mine blasting operations.

The following investigations were conducted:

- Ground and structural vibration measurements.
- Airblast measurements.
- Structural acceleration measurements.
- Direct structural strain measurements.
- Foundation investigation and geotechnical investigation of the foundation soil.
- Structural condition inspection and progressive monitoring.
- Crack width and growth monitoring.
- Level loop surveys of brickwork.
- Determination of characteristic strengths of bricks, mortar and plasterboard.
- Rainfall records.

5.1 Ground and Structural Vibration Measurement

Triaxial vibration measurements were recorded by geophones placed on the ground and at various locations on and in the houses to measure the peak particle velocity, structure response amplification and the vibration frequency spectrum. The recorded velocity was then converted to acceleration and displacement for comparison to the accelerometer measurements and direct strain measurements.

The measurement of peak particle velocity in the ground near the houses is the procedure by which ground vibration is controlled by the regulatory authorities.

5.2 Acceleration Measurement

Accelerometers were placed at various locations on the houses to measure and record accelerations at ground and ceiling levels. These measurements were used to obtain mid wall (out-of-plane) and racking (in-plane) responses and amplifications, both on the external brickwork and internal plaster. The accelerations were then converted to velocity and displacement for comparison to the geophone measurements. The strains induced in the house fabric by the ground vibration could then be determined and compared to direct strain measurements.

5.3 Air Vibration Measurement

The air vibration from each blast was measured outside the house and the response of the structure to the air vibration determined. The air vibration and ground vibration have different transmission velocities and separate with distance. However, at close distances the ground and air vibrations do not separate sufficiently for separate structural responses to be recorded.

At greater distances, where separate responses can be isolated, the levels of both air and ground vibration attenuate to low levels and structural response was difficult to identify and measure. For the reasons outlined, the houses selected for the study did not give enough data for a separate detailed analysis, however, enough air vibration structure responses were observed for comparisons to be made to a more detailed overseas study and valid conclusions drawn.

5.4 Direct Strain Measurement

Strain gauges were placed at locations on the external walls of houses to directly measure the strains induced by the structure response to ground vibration. The measured strains were then compared to those derived from accelerometer and geophone response analysis.

5.5 Foundation Investigation

For each house, a hand excavation was made to measure the depth of the concrete footings. This was compared to the requirements of Australian Standard (AS) 2870-1996 – Residential Slabs and Footings – Construction, for compliance. The operating Standard at the time of construction of two of the test houses permitted a lower standard of footing to be used, which was found to be inadequate for the limitation of cracks because of footing movement, when compared to the current standard.

5.6 Geotechnical Investigation of Foundation Soil

Samples were taken of the foundation soil at a number of locations around each of the test houses and submitted for testing. At Test House No. 3 the soil was also evaluated using a cone penetrometer test rig from the University of Newcastle. Samples were collected by hand auguring and core sampling. The characteristics of the soil were determined and of particular interest was the potential for swelling/shrinking of the surface with moisture variation. A soil classification was determined for each site. The soil classification was then used to determine footing requirements from the code (AS2870-1996) and compared to that found in Investigation 5.5.

5.7 Structural Condition Inspection and Record

The structural condition inspection and report is the primary method in the investigation of complaints of blast vibration damage. In conjunction with the initial crack survey, a floor plan of the house was produced, the construction materials recorded and structural defects, especially cracks, recorded. The floor plan was used to produce a floor/wall/ceiling plan for each room, which was the basis for recording the crack locations. The presence of cracks often required further detailed investigation to explain their cause. The cracks were recorded on a sketch plan supplemented by photographs. Torch, magnifying glass, a carpenter's level and a ladder were essential equipment.

During this inspection it was important to note the provision for drainage of roof and surface run-off and the proximity of paving, garden beds and large trees to the house foundations.

5.8 Crack Recording and Growth Monitoring

During the initial inspection the position, description, width and length of all cracks were noted, recorded on a sketch plan and photographed, where appropriate. At unoccupied houses, the ends of cracks were progressively marked and the dates noted. Before and after major blasts and at convenient intervals, the crack survey was updated and the new crack extensions marked. A crack development history was thus obtained and compared to blasting and non-blasting events.

5.9 Crack Width Survey

Permanent targets were established so the width of selected existing cracks could be accurately measured with a DEMEC gauge. The width of the cracks was measured before and after major blasts and at convenient intervals and a crack width history developed. This was then compared to the occurrence of blasting and non-blasting events, particularly rainfall. A strain gauge placed across a crack for a number of blasts was also used to measure the instantaneous change of crack width due to the dynamic loading during vibration events.

5.10 Masonry Level Survey (Level Loop Survey)

A level survey of a course of bricks around the house was conducted as a check on possible movement since construction. The accuracy of the initial construction is also measured with this survey with a construction tolerance of ±5 mm in any 10 metre length around the circumference of a house, permitted by AS3700-1998 (Masonry Structures). The survey was repeated at later dates to measure subsequent movement. This survey is simple to do and, though by its nature not exact, rapidly shows if substantial footing movements have occurred since construction and over time intervals. The presence of cracks in brickwork and plasterboard are often related to footing movements shown by a loop survey and indicate that further geotechnical investigation of the footings may be warranted.

5.11 Strength of Bricks, Mortar and Plasterboard

The brickwork at each house was tested in accordance with AS3700-1998. The characteristic compressive strength of the bricks and masonry and the characteristic flexural strength of the masonry were determined.

The flexural testing by bond wrench test involved the isolation of at least ten in situ bricks, starting at an opening, such as a vent, and testing the bond between the isolated bricks and the mortar to bond release by the application of a moment arm. These bricks were re-mortared in place after testing. The bond wrench test proves a useful means of comparing different masonry samples but does not provide a flexural strength indicator consistent with possible failure modes.

The compressive strength of the bricks was determined at the laboratory of the University of Newcastle and involved destructive testing of bricks sampled from the houses. The characteristic strength of masonry was estimated conservatively from the AS3700 relationship using the measured brick compressive strength and nominal mortar type.

The cracking strength of plasterboard was established from previous full scale wall tests at the University of Melbourne. This was further verified by findings from similar international research.

6. SUMMARY OF INVESTIGATIONS

6.1 Test House No. 1 (Rix's Creek, near Singleton, N.S.W.)

6.1.1 Rix's Creek Blasting Operations

Rix's Creek Colliery is located 9 km north-west of Singleton township, and blasting operations are carried out as close as 4 km to the closest urban areas and at distances varying between 50 metres to 1000 metres from the test house. The current level of production requires the removal of 8 million bank cubic metres of overburden to produce 1.2 million tonnes of coal.

Typical overburden blasting specifications are shown in **Table 1**.

Table 1 - Typical Rix's Creek blast specification

Blasthole Diameter:	229 mm
Burden:	5 m
Spacing:	8 m
Inclination:	15^o
Face Height:	35 m
Stemming Height:	5 m
Stemming Material:	Drill cuttings or 10mm agg.
Charge Mass/Delay:	1000 kg
No. of Rows:	12
Hole Pattern:	Rectangular
Control Row Delay:	25 ms
Echelon Row Delay:	65 ms

6.1.2 Description of Construction

The test house is of conventional brick veneer construction, with a timber frame, 10 mm plasterboard internal lining, tiled roof, timber floor boards and aluminium framed windows. The house is approximately 25 to 30 years old, having been constructed in the early 1970s. The brickwork is supported by strip footings, whilst the timber floor is supported by timber floor joists and bearers and masonry piers.

The garage floor is an on-ground concrete slab while the laundry, bathroom and toilet have suspended concrete slab floors. There were no expansion or articulation joints in the brick veneer to control movement, as required by the current Standard.

A floor plan of the house is shown in **Figure 1** and a side view of the house is shown in **Figure 2**.

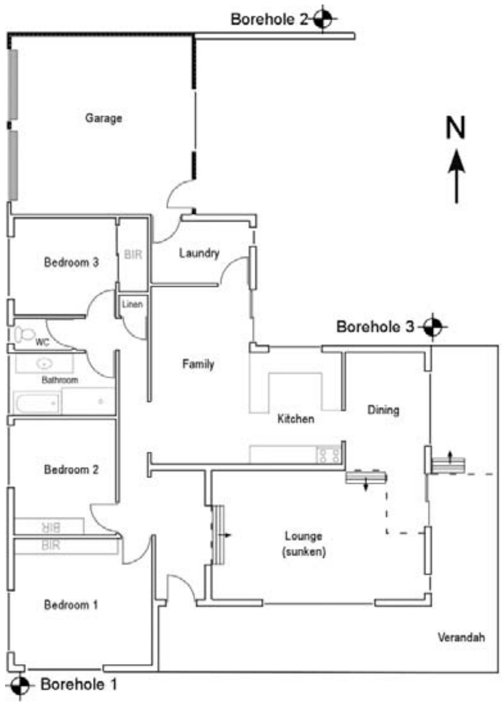


Figure 1 - House floor plan

Figure 2 – South-east view of Rix's Creek test house

At the commencement of the project, the condition of the house was reasonable and consistent with its age and history. The plasterboard and brickwork had a number of cracks ranging in size from hairline (<0.2 mm) to noticeable but easily filled (<5 mm) in the brick veneer and plaster walls. The main cracks were found to be due to foundation movement and poor building practices – which were discovered after detailed inspection. The cracks and defects in the house at the commencement of the project are shown in **Figures 3a** to **3r**.

The house was demolished in early May 2001 before final crack recording could be completed.

Figure 3a - Stepped brick crack at south-west corner below DPC

Figure 3b - Stepped brick cracks below DPC at south-west corner

Figure 3c – Horizontal crack at DPC - separation of steps from verandah

Figure 3d - Cracked brickwork (stepped and vertical) at kitchen window

Figure 3e - Horizontal mortar crack between toilet and bathroom windows

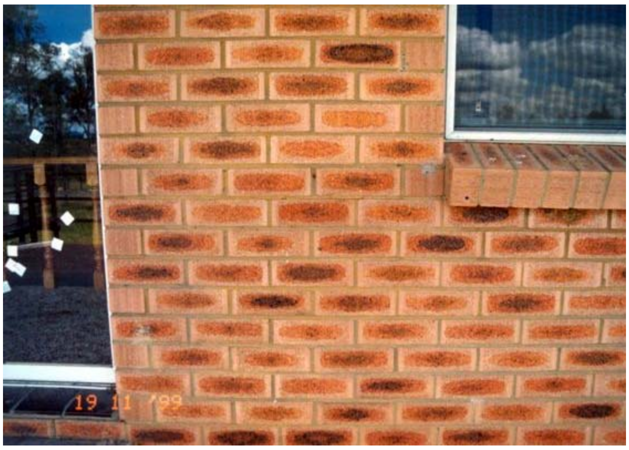


Figure 3f - Horizontal brick crack from dining room window

Figure 3g - Freestanding wall moves independently of the garage

Figure 3h – Horizontal cracked brickwork in garage at end of steel lintel

Figure 3i – Horizontal cracked brickwork in garage pier

Figure 3j - Concrete shrinkage cracks in garage apron

Figure 3k - Separation of path from wall

Figure 3l - Shrinkage crack in concrete verandah slab

Figure 3m - Inclined plaster crack in kitchen from window to encased beam

Figure 3n - Cracked cornice and ceiling plaster sheet join, dining room

Figure 3o - Vertical masonry crack in kitchen

Figure 3p - Cracked cornice in kitchen near beam

Figure 3q - 45° plaster crack in laundry

Figure 3r - Cracked cornice/wall and plaster sheet join, laundry

6.1.3 Recorded Blast Vibration Levels

The peak particle velocity measured on the ground near the house and the peak airblast measured during this investigation are listed in **Table 2**.

Table 2 - Blast vibration measurement summary

Blast No.	Date	Charge Mass (kg)	Distance (m)	PPV (mm/s)	Air Vibration (dBL)
1	01/03/00	300	231	18.4	128
2	20/03/00	1300	450	16.1	140
3	22/03/00	300	268	14.2	127
4	27/03/00	200	363	4.5	125
5	11/04/00	85	395	4.5	129
6	13/04/00	150	306	7.3	124
7	13/04/00	150	306	6.3	127
8	04/05/00	1000	401	17.4	125
9	09/05/00	250	255	11.3	126
10	12/05/00	80	280	1.7	124
11	12/05/00	250	280	15.0	127
12	22/05/00	1000	260	20.5	128
13	31/05/00	300	380	9.3	120
14	31/05/00	50	408	3.0	117
15	14/06/00	50	425	1.5	124
16	14/07/00	50	247	4.7	136
17	18/07/00	150	418	6.9	133
18	25/07/00	-	-	8.0	130
19	28/07/00	30	-	4.9	135
20	07/08/00	200	214	16.6	124
21	07/08/00	50	447	2.9	120
22	08/08/00	1100	810	9.7	128
23	11/08/00	350	106	71.2	131
24	11/08/00	250	333	9.6	123
25	17/08/00	300	106	17.3	128
26	02/11/00	150	252	10.4	128
27	06/11/00	200	-	13.0	126
28	27/11/00	150	166	36.2	134
29	28/11/00	300	135	73.6	129
30	30/11/00	110	106	44.4	>145
31	07/12/00	350	55	190.0	136
32	20/12/00	330	50	222.0	145
33	21/12/00	1100	400	41.4	124
34	19/01/01	150	250	9.3	-

The proximity of blasting to the house is shown in **Figure 4**.

Another strip closer to the house was blasted after this photograph was taken, resulting in blasting to within 50 metres of the house.

Figure 4 - Blasting in close proximity to the test house – 11/08/00

6.1.4 Structural Tests

The masonry has been constructed from extruded, fired clay bricks. The brickwork (10 bricks) was tested to the requirements of AS3700-1998 and gave the results listed in **Table 3**.

Table 3 - Brick and masonry test results

Unconfined compressive strength of bricks:	22.4 MPa (mean)
	20.6 – 25.3 MPa (range)
Characteristic compressive strength of bricks:	17.7 MPa
Characteristic strength of masonry*:	6.1 MPa
Flexural strength of masonry (bond wrench):	0.25 MPa (mean)
	0.06 – 0.58 MPa (range)
Characteristic flexural strength of masonry:	0.03 MPa

***** *estimated from the provisions of AS3700*

The bricks are of reasonable quality. The bond wrench strength is considered low, but is typical of domestic construction where the standard of workmanship is highly variable.

6.1.5 Foundations and Geotechnical Report

The house is constructed on a gently sloping, elevated area which has been cleared to leave only occasional mature Iron-Bark and Eucalypt trees. Most trees are remote from the house, except for a tree of moderate height on the western side. Three boreholes were drilled adjacent to the house to investigate the soil properties (refer **Figure 1**).

The sub-surface conditions are characterised as residual clays derived from sandstones. The total depth of soil overlying the sandstone varies between about 0.6 metres and 0.9 metres, and the profile may be summarised as follows:

- Up to 150 mm (CL-MH): silty CLAY/clayey SILT topsoil, low to medium plasticity.
- 150-500 mm (CH): CLAY, high plasticity.
- 500-800 mm (CL-CH): silty CLAY, medium to high plasticity.
- >800 mm: red-purple sandstone, medium grained, highly weathered – non expansive.

Based on the methods and classification criteria of AS2870-1996, the characteristic surface movement is estimated to be 25 mm for the average sub-surface site conditions and the site is classified as Class M, or moderately reactive.

Characteristic surface movements at each borehole location are estimated to be 23, 12 and 29 mm for boreholes 1, 2 and 3, respectively.

The strip footings beneath the structure were examined in an excavation adjacent to borehole 1. This excavation revealed a strip footing of between 370 mm and 400 mm deep. A footing depth of 400 mm is consistent with the requirements for a masonry veneer dwelling on a Class S site, as outlined in AS2870-1996, but is less than the 500 mm depth required for a masonry veneer dwelling on a Class M site.

The footings of this house may be regarded as being inadequate to limit damage due to the effects of the reactive soil foundation at this site. It may thus be expected that the performance of this house, in regard to the effects of a reactive soil foundation, may fall outside the bounds considered as satisfactory in Appendix C of AS2870-1966.

The performance of the sub-standard footings is exacerbated by the lack of provision for drainage of the overflow of concrete water storage tanks. The tanks overflowed frequently, due to the house being unoccupied. Overflowing water ponds at several locations on the surface before flowing beneath the garage floor slab and ponding beneath the floor of the house below the south-west corner. The two areas of external brickwork cracks correspond to the areas of water ponding.

The soil profile ranges from fully saturated to practically dry over a period of time, with considerable lateral variation. When the profile dries out, the soil shrinks considerably, as indicated by the geotechnical testing. **Figure 5** shows a pier which has shrunk away from the floor joist by about 20 mm and **Figure 6** shows 25 mm wide cracks between the soil and the footings.

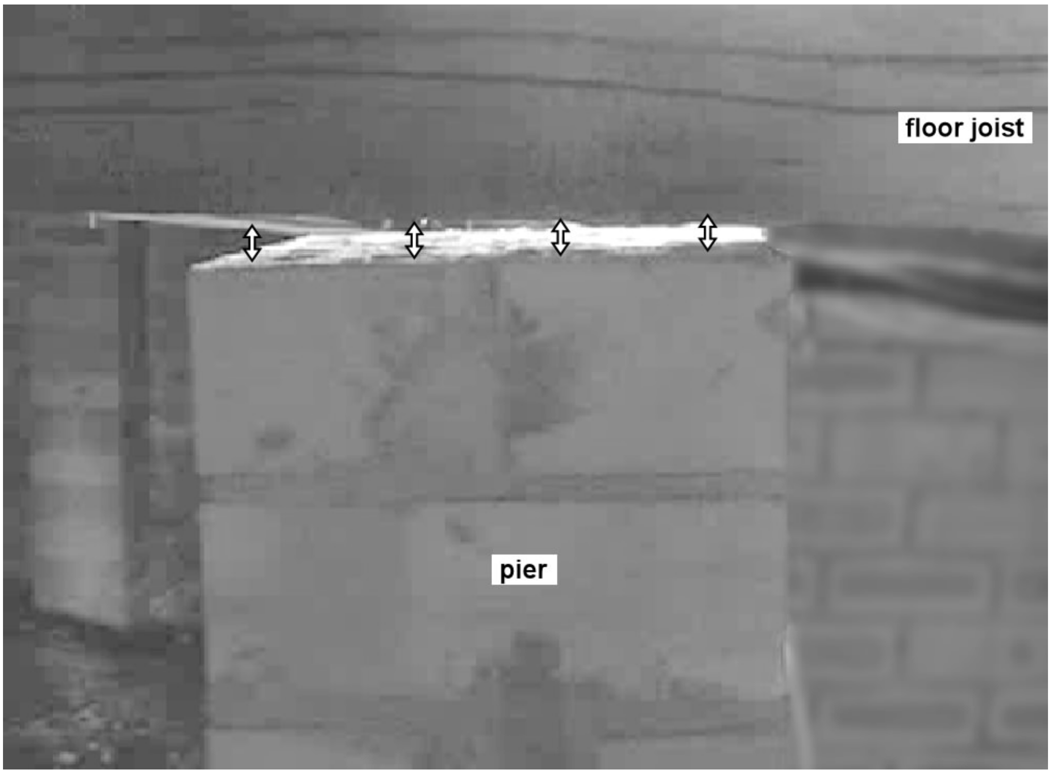


Figure 5 - 20 mm gap between brick pier and floor joist

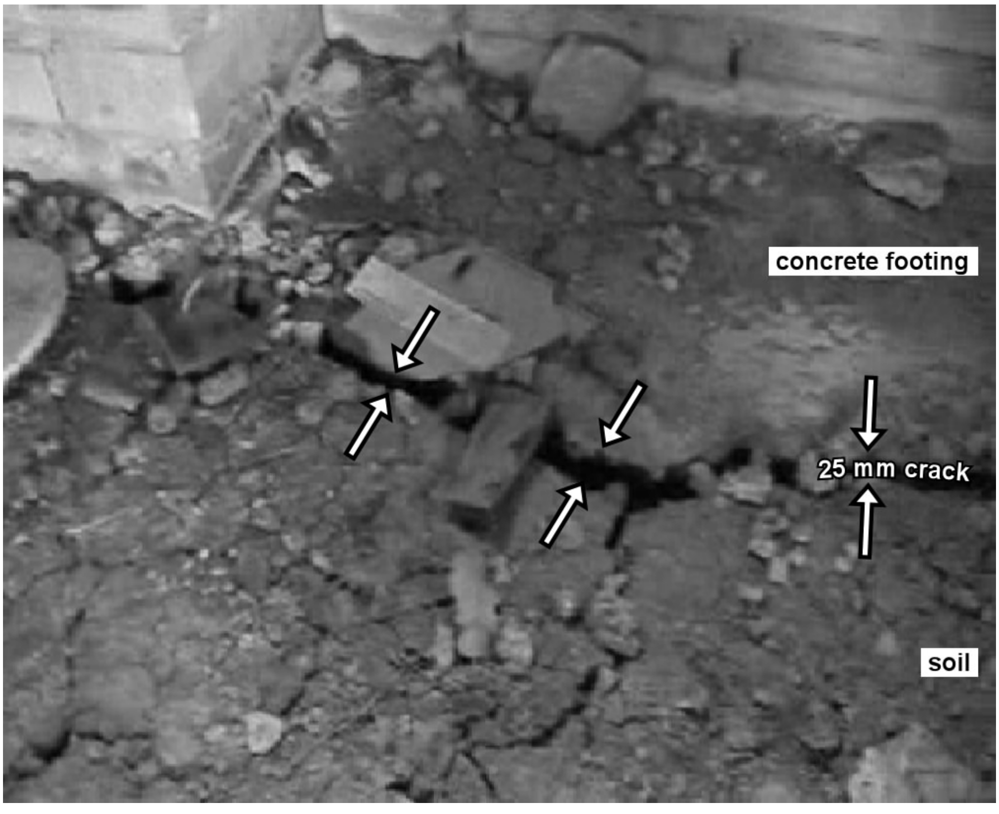


Figure 6 - 25 mm crack between footing and soil beneath house

6.1.6 House Condition Report

6.1.6.1 Crack Width Survey

Prior to the project, a number of cracks had developed over the life of the house comprising of open cracks in plaster and open, vertical, horizontal and zigzag cracking following mortar courses in masonry. The crack patterns are generally consistent with loss of foundation support and settlement of footings, although some related directly to poor building practices.

The growth pattern of all cracks was recorded between March 2000 and April 2001. The width of seven cracks in the house structure have been accurately monitored with DEMEC gauge measurements between installed targets. Photographs of the monitored cracks are shown in **Figures 3a** to **3r** and are listed in **Table 4**.

The monitored cracks are described in **Table 4** together with the maximum movement during blasting and movement after blasting ceased. Crack width movement diagrams are shown in **Figures 7** and **8** and cumulative rainfall total, together with peak ground vibration measurements and daily rainfall totals.

Table 4 - Crack description and width movement summary

No.	Location	Photo No.	Internal	External	Description	Movement during blasting (mm)	Movement without blasting (mm)
1	Living Room		✓		Stepped crack in non-structural brickwork.	0.3	1.3
2	Kitchen	3d		✓	Stepped crack in veneer brickwork.	0.12	2.2
3	Kitchen	3o	✓		Vertical crack in structural brickwork.	<0.1	0.5
3	Kitchen	3m	✓		Horizontal crack in plaster.	<0.1	0.6
5	Laundry	3q	✓		Sub-horizontal crack in plaster.	<0.1	0.5
6	Bedroom	3a		✓	Horizontal crack in veneer brickwork.	<0.1	0.2
7	Bedroom	3a		✓	Vertical crack in veneer brickwork.	<0.1	0.6

The external crack showing the most movement was adjacent to where the tank overflow water ponded against the kitchen wall. The extremes of width behaviour of all cracks over the monitoring period shows similar complex movement that is unrelated to blasting but is related to periodic variations in rainfall. Limited width movement of some cracks is related to ground vibration for the 4 blasts in excess of 70 mm/s.

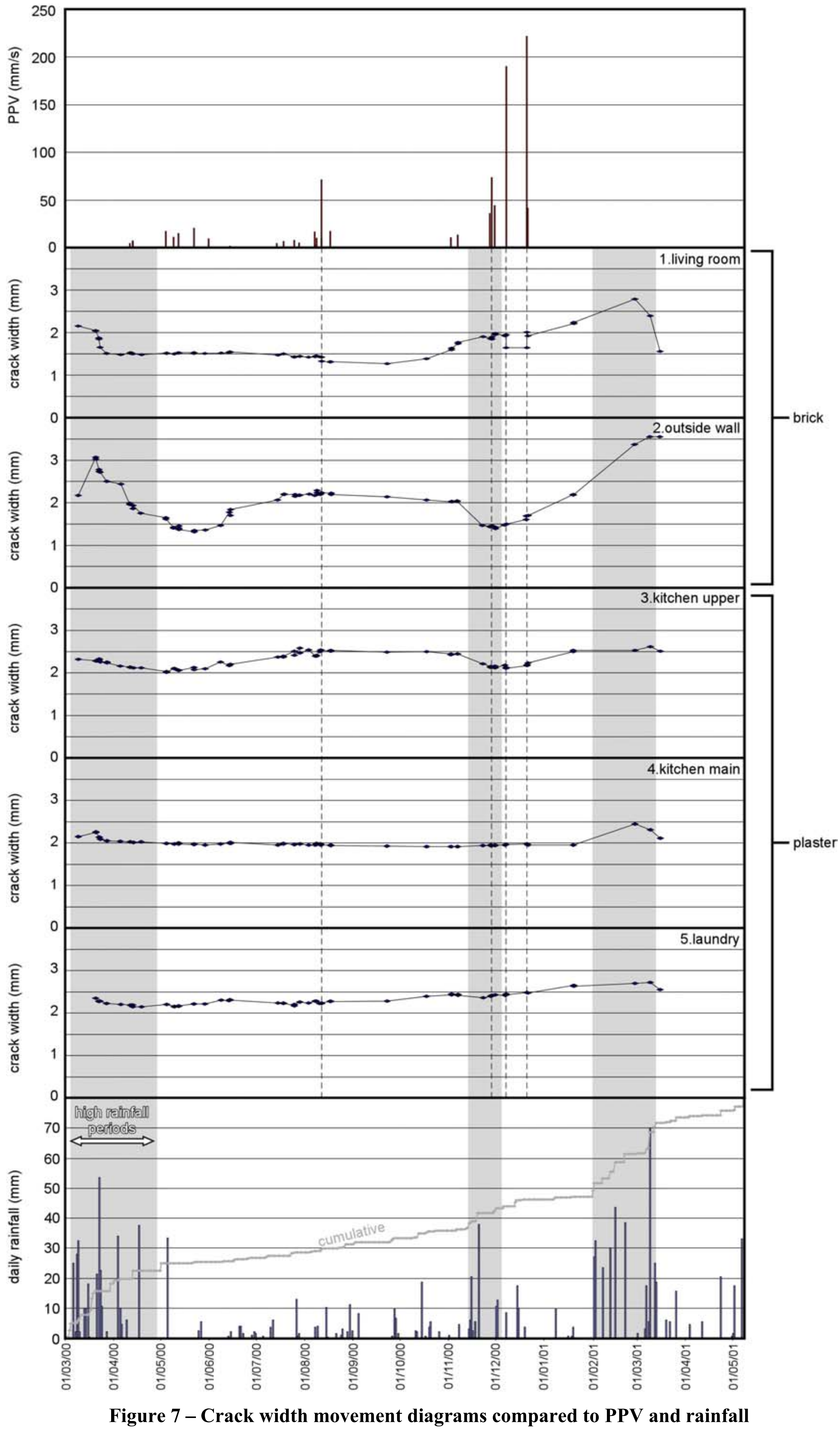


Figure 7 – Crack width movement diagrams compared to PPV and rainfall

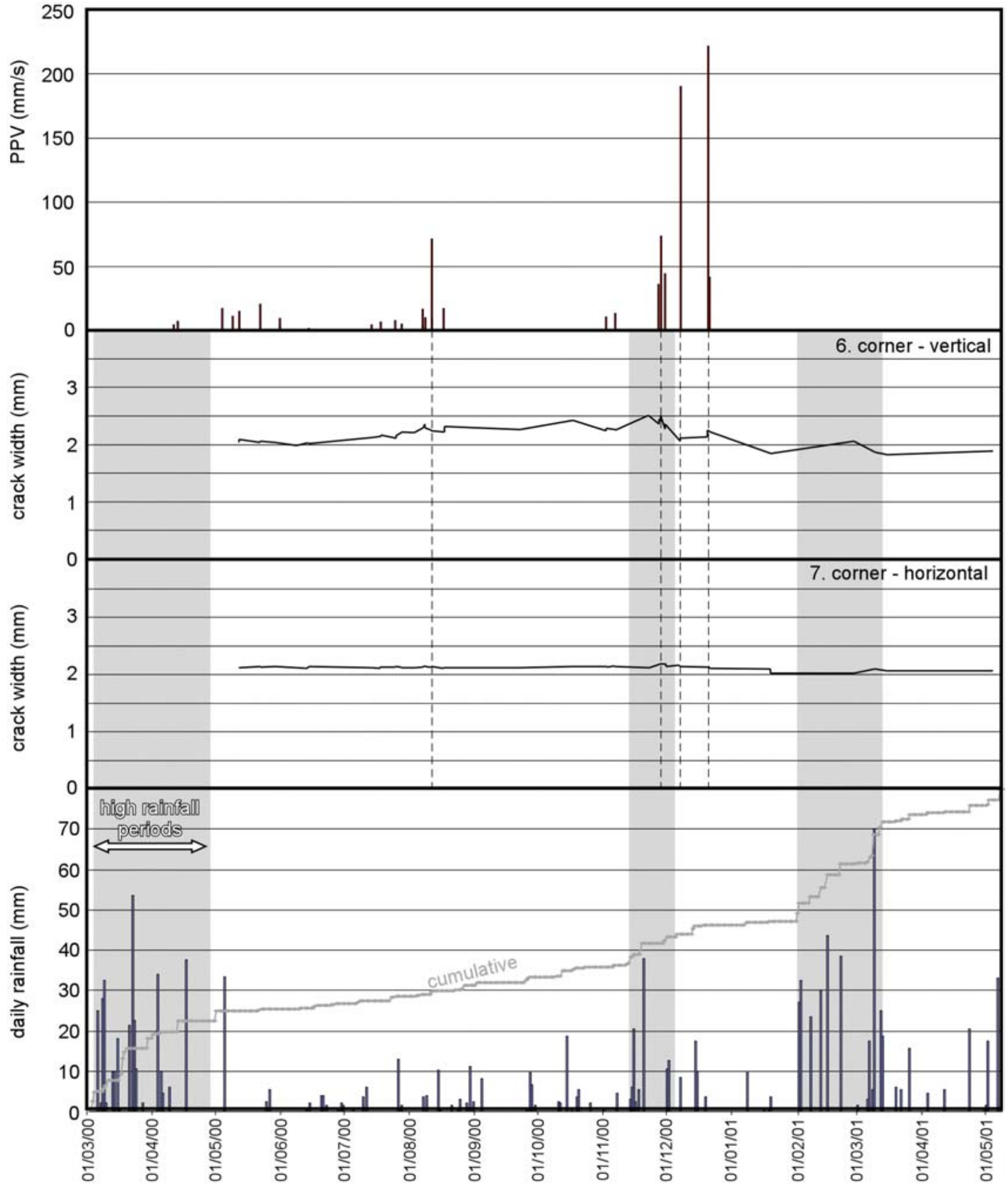


Figure 8 – Crack width movement diagrams compared to PPV and rainfall (continued)

The internal crack showing the most width movement was located in a low (700 mm high), non-structural brick wall in the sunken lounge room. Sub-floor inspection disclosed that the foundations of this wall were extremely sub-standard, being mortar spread on the natural ground surface. The variations in width of the lounge room (crack No. 1 refer **Table 4**), between March 2000 and May 2001, is plotted in detail in **Figure 9**.

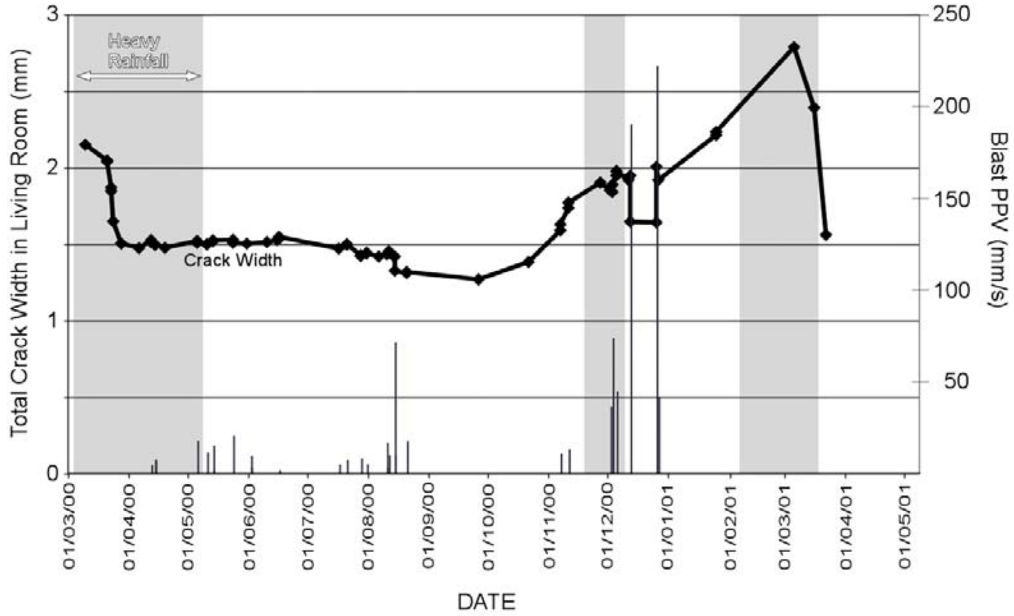


Figure 9 – Detail crack width movements of crack No. 1

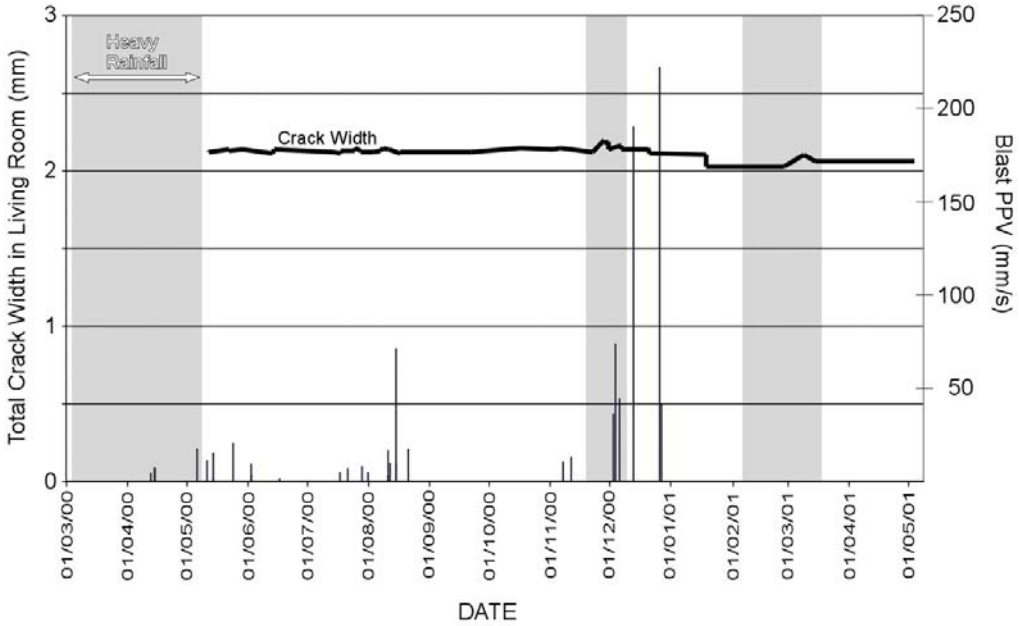


Figure 10 – Detail movements of crack No. 6

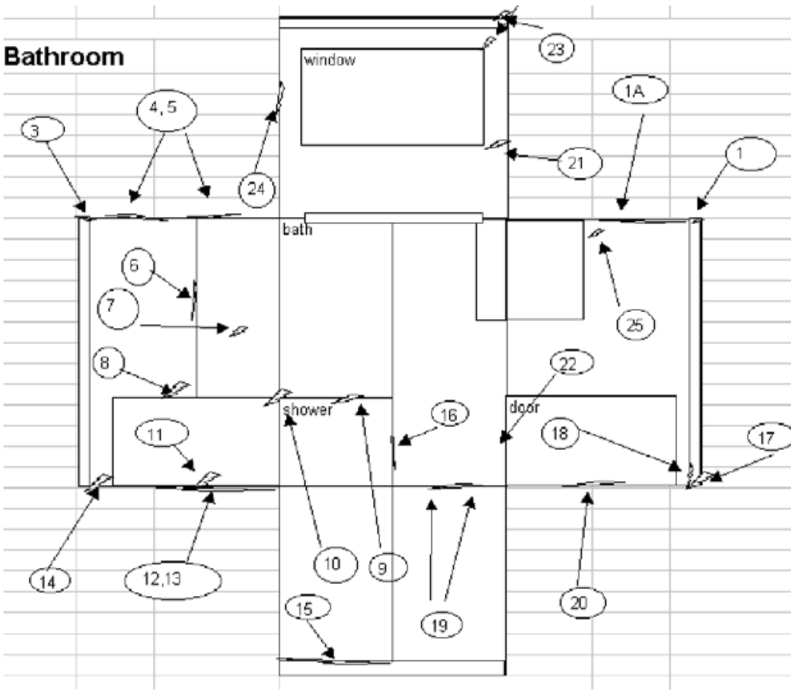
The width of crack No. 1 did not appear to be effected by the numerous blasts with PPV less than 20 mm/s over the period 1st April to 10th August 2000. A permanent offset of 0.1 mm was measured following Blast No. 23 on 11th August 2000, with a peak ground velocity of 70 mm/s. In contrast, the crack closed from a width of 2.2 mm (5th March 2000) to 1.5 mm (25th March 2000), following a period of heavy rainfall, and then remained constant at 1.5 mm up to July 2000. Between July and October 2000 the crack width reduced to 1.3 mm and then widened to 1.6 mm by November 2000. The blast of 7th December 2000 (190 mm/s) caused the crack to close from 2.0 mm to 1.7 mm and the next blast (220 mm/s) caused the crack to re-open to 2.0 mm. The crack widened to 2.8 mm following a period of heavy rain after the end of the close blasting and monitoring period and then closed to 1.6 mm by mid March 2001.

As a contrast, crack No. 6 in the external brickwork (**Table 4**), detailed in **Figure 10**, was relatively unaffected by blasting and soil moisture variations.

From **Figures 7** and **8** it can be seen that the rainfall pattern shows a strong correlation with the variation in width of some cracks. These cracks have responded to a high rainfall period after the cessation of blasting. Permanent widening of cracks listed as 1 and 2 in **Table 4** were recorded after receiving a ground vibration measurement of 222 mm/s. The widening was 0.3 mm and 0.12 mm respectively. However, crack movements of 1.3 mm and 2.2 mm were recorded in the same cracks after blasting ceased and are due to the effects of ground moisture change.

6.1.6.2 Crack Length Survey

The location, size, length and configuration of the cracks in all rooms were mapped periodically before and after the larger blasts and changes noted. As an example of the crack monitoring technique adopted in this study, **Figure 11** shows the bathroom with all the cracks numbered on the walls and ceiling.



No	Width	Length
1	0.75	150
1A	HL	1500
2	0.1	175
3	0.25	153
4	HL	40
5	0.5	25
6	HL	45
7	0.5	250
8	<0.1	150
9	0.5	1500
10	0.75	200
11	0.25	71.6
12	0.5	1700
13	0.5	46.6
14	0.25	91.5, 197
15	0.25-paint	1600
16	cracked door	
17	HL	150
18	HL	47
19	missing tiles	
20	HL	20
21	0.2	110
22	nail head	
23	HL	20
24	HL	60
25	chipped	60mm^2

updated on 07/12/00

Figure 11 – Typical survey of the bathroom showing all cracks on the walls and ceiling

Similar plots have been used to record the crack patterns in all other rooms. The total length of the cracks in each room were plotted and the growth of the cracks compared to ground vibration and rainfall. **Figure 12** shows the crack growth for five rooms until the end of the monitoring program. A final crack length survey of the laundry was conducted in May 2001 as the house was being demolished.

The four blasts with vibration levels in excess of 70 mm/s resulted in additional cracks in the plaster of all rooms, but the bathroom was relatively unaffected.

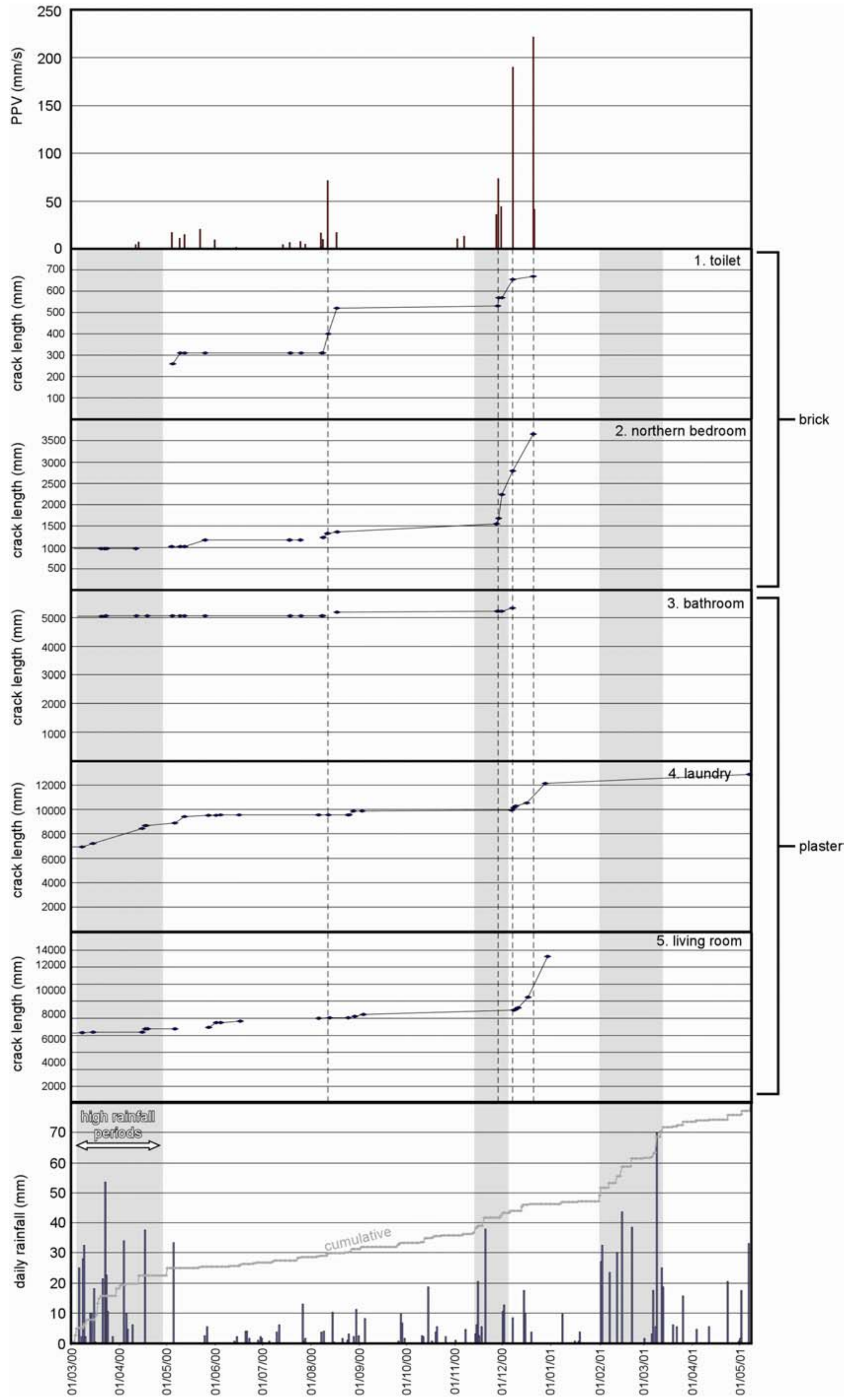
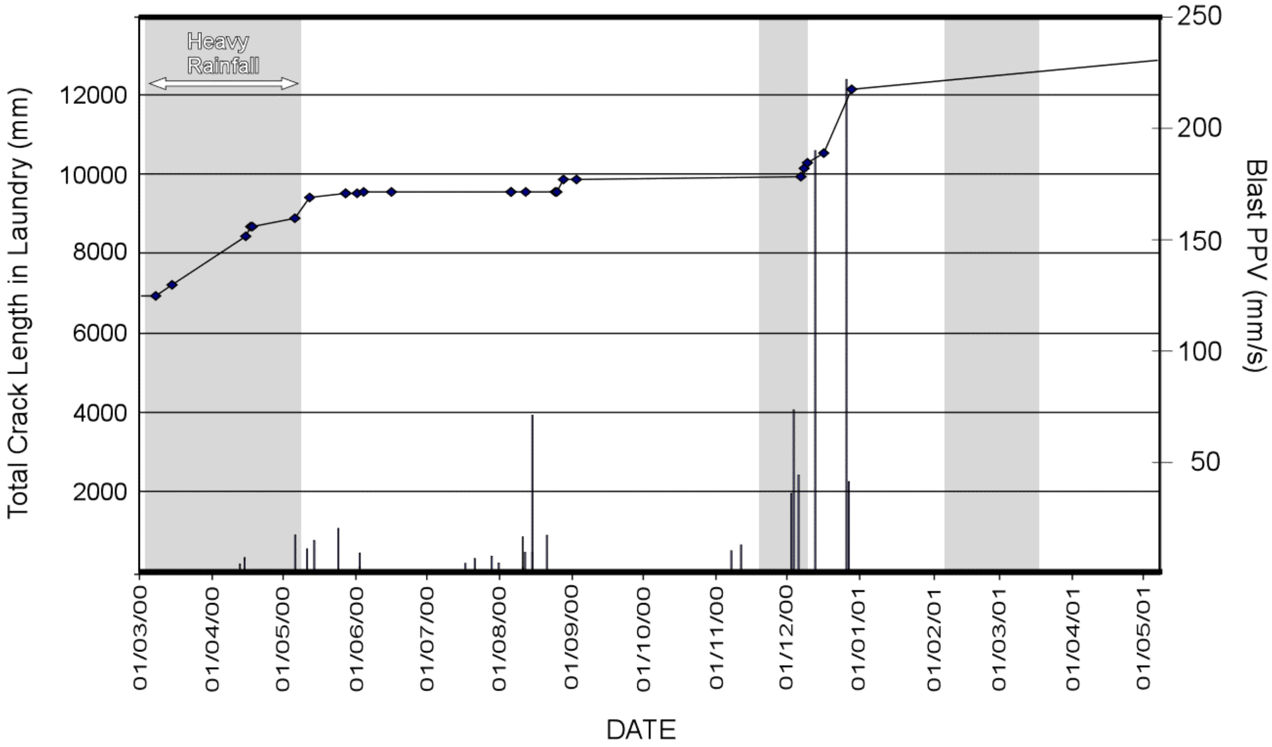


Figure 12 - Total crack length for 5 rooms compared to rainfall and PPV



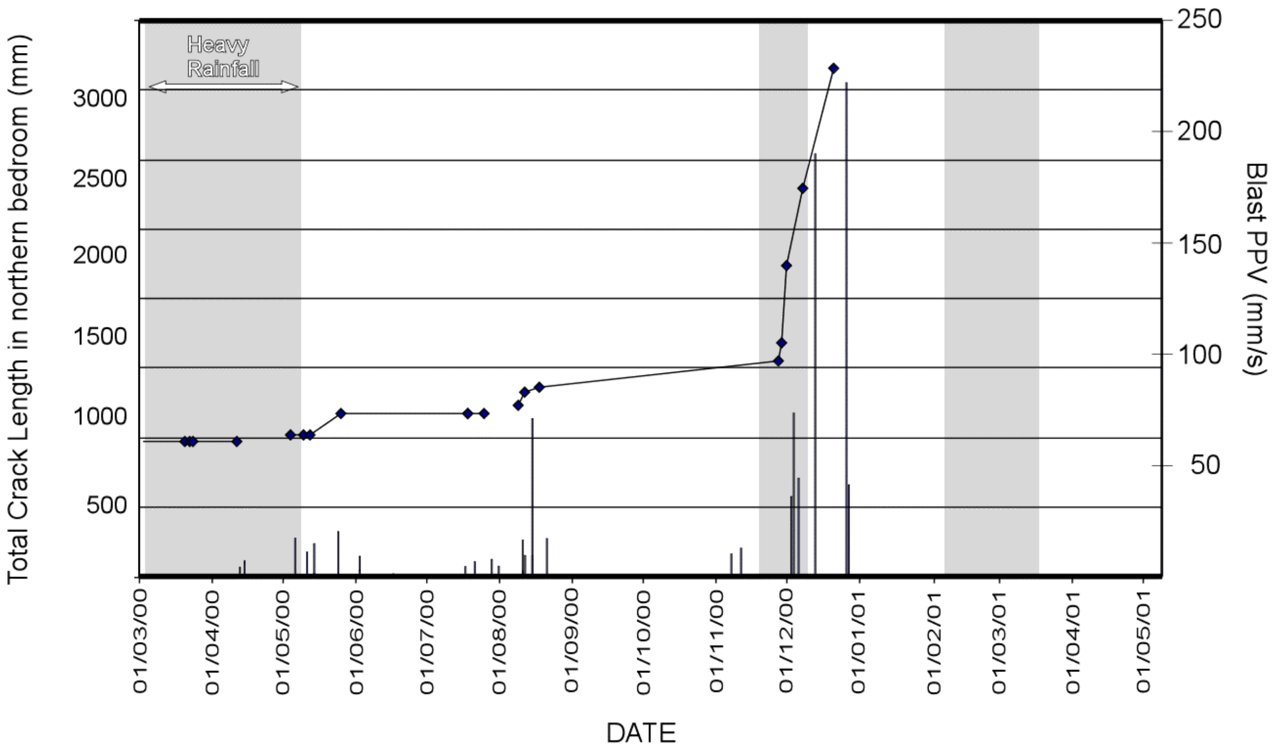


Figure 13 – Total crack lengths for the laundry and bedroom 3 and blast history for 2000

Figures 12 and **13** demonstrate that the total crack lengths between May and August 2000 did not increase, despite some 12 blasts with PPV in the range of 2 mm/s to 20 mm/s.

An explicit change in crack lengths of some rooms was observed on 11th August and 28th November 2000 associated with blasts with ground PPV in the order of 70 mm/s. Further significant change to the total crack lengths was observed following blasting on 28th November 2000, which produced a ground PPV of 73.6 mm/s and subsequent blasts recording 44.4 mm/s, 190.0 mm/s, 222.0 mm/s and 41.4 mm/s. It may be concluded that vibration levels of 70 mm/s and above can result in cosmetic damage to plaster. The crack width behaviour in the laundry is evidence that the effects of rainfall can also cause the lengthening of plaster cracks.

6.1.6.3 Dynamic Crack Movement

The horizontal strain gauge placed across the brickwork crack below the kitchen window in Test House No. 1 (see **Figure 3d**) enabled the changing width of the crack to be measured in response to ground vibration. The strain gauge length was 300 mm. Strain cannot be transmitted across the crack so the gauge measured the changing crack width under dynamic load.

The change of crack width is the horizontal strain multiplied by 300 mm and is listed in **Table 5** for the blasts noted. The crack was approximately 6 mm wide and returned to its original width after the blasts because there was no permanent deformation noted in the strain traces.

Table 5 – Change of crack width

Blast Date	PPV (mm/s)	ε_H ($\mu\varepsilon$)	Increase in Crack Width (mm)
23/12/99	7.3	37.8	0.011
21/01/00	13.2	112.0	0.033
08/02/00	12.5	130.8	0.039
15/02/00	21.5	175.3	0.052
18/02/00	20.8	282.4	0.084
27/03/00	4.4	23.7	0.007
11/04/00	4.5	46.5	0.013
13/04/00	6.3	34.1	0.010
13/04/00	7.3	89.6	0.026

Figure 14 shows the plot of increased crack width versus PPV. There is a close correlation and it may be reasonably concluded that a ground PPV of 20 mm/s results in cracks temporarily opening between 0.05 mm and 0.08 mm.

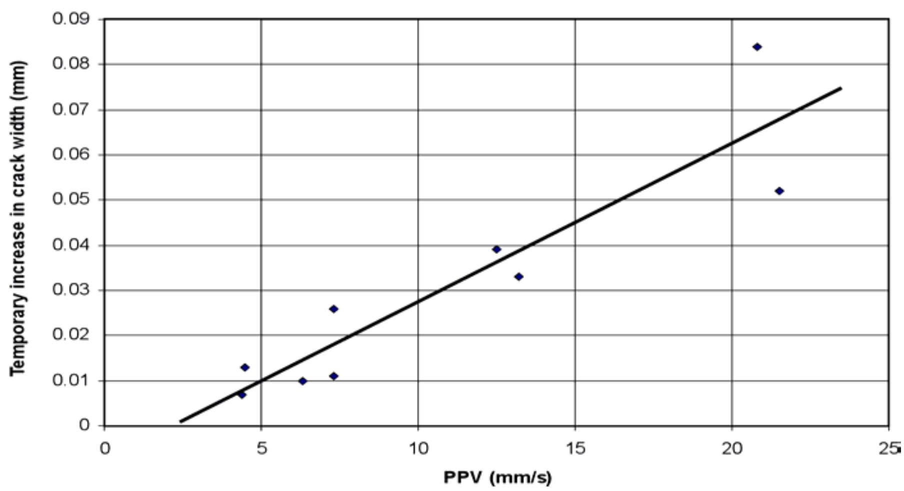


Figure 14 - Change of crack width versus PPV – crack No. 4

The DEMEC gauge crack survey (Section 6.1.6.1) showed that a permanent widening of crack No. 1 of 0.12 mm occurred after the blasts resulting in PPVs of 71.2 mm/s and 0.3 mm occurred after 222 mm/s.

This further reinforces the observation that at vibration levels below 20 mm/s, the brick wall is behaving elastically and returns to its original position. At vibration levels above 70 mm/s, there is permanent widening but there is also partial closure from the widest opening, indicating a remaining degree of elasticity.

6.1.6.4 Observed Damage due to Blasting

For the first 22 blasts during the monitoring period, the maximum ground vibration measured was 20.5 mm/s and only minor damage, such as slight lengthening of existing hairline cracks in the plaster, was observed (see **Figure 12**).

Following Blast No. 23 on 11th August 2000, where the peak ground vibration was 71.2 mm/s, it was noticed that a section of the plaster ceiling in the lounge room had sagged about 30 mm at the hallway end (refer **Figures 15a** and **15b**). Subsequent investigation showed that the plasterer had not used adhesive on the two ceiling joists nearest the western wall and the nails had pulled through the plaster and backing paper. The remainder of the ceilings throughout the house were unaffected. The sagging plaster was re-attached to the joists with plaster adhesive by propping from beneath until the adhesive set. The entire ceiling remained intact for the duration of the investigation.

Figure 15a - Sagging lounge room ceiling after a PPV of 70 mm/s

Figure 15b – Lounge room ceiling - nails pulled through plaster and backing paper causing the ceiling to sag

The remainder of the blasts, which resulted in peak ground vibration ranging from 9.6 mm/s to 222 mm/s, caused minor damage to the plaster, such as cracks around nail heads, lengthening of existing hairline cracks and the opening of new hairline cracks.

The crack growth survey of bedroom three (see **Figure 12**) is typical of the crack growths measured. Blast No. 23, where a level of 71.2 mm/s was recorded at the test house, resulted in an extra 250 mm of hairline cracks. The series of blasts in excess of 35 mm/s from 27th November 2000 (including 190 mm/s and 222 mm/s) resulted in an additional 2200 mm of cracks.

The peak air vibration exceeded 145 dBL, but no glass damage resulted in windows or sliding doors.

The observed damage was confined to the plaster. There was no damage to the ceramic wall and floor tiles in the hallway, bathroom or laundry. There were no additional cracks in the concrete floor slabs of the garage, verandah or pathways. There was no additional damage in the external brickwork, roof tiles, ridge capping or concrete water tanks. The observed plaster damage could be described by AS2870 (Residential Slabs and Footings) Wall Damage Classification Criteria (see **Table 9**) as Category 2 *'cracks noticeable but easily filled'* with crack width <5 mm.

Variations in crack widths were more likely to occur in periods between major blasts, than during blasting events. Crack width variations commonly coincided with high rainfall periods that would result in variations in ground moisture. A permanent displacement of 0.3 mm was recorded in the living room crack after the blast vibration of 222 mm/s. Minor displacements of <0.1 mm were recorded in three cracks after the four blasts exceeding 70 mm/s. After cessation of blasting, following a high rainfall period, crack width movements of 2.0 mm were recorded at crack No. 2.

The vibration levels in excess of 70 mm/s clearly have had an effect on the crack lengths. Vibration levels less than 25 mm/s have had no perceptible effect on crack widths or lengths. This is consistent with the guide values of BS7385: Part 2: 1993 to prevent cosmetic damage, reproduced as **Figure 16** and including the vibration and damage observations from this investigation.

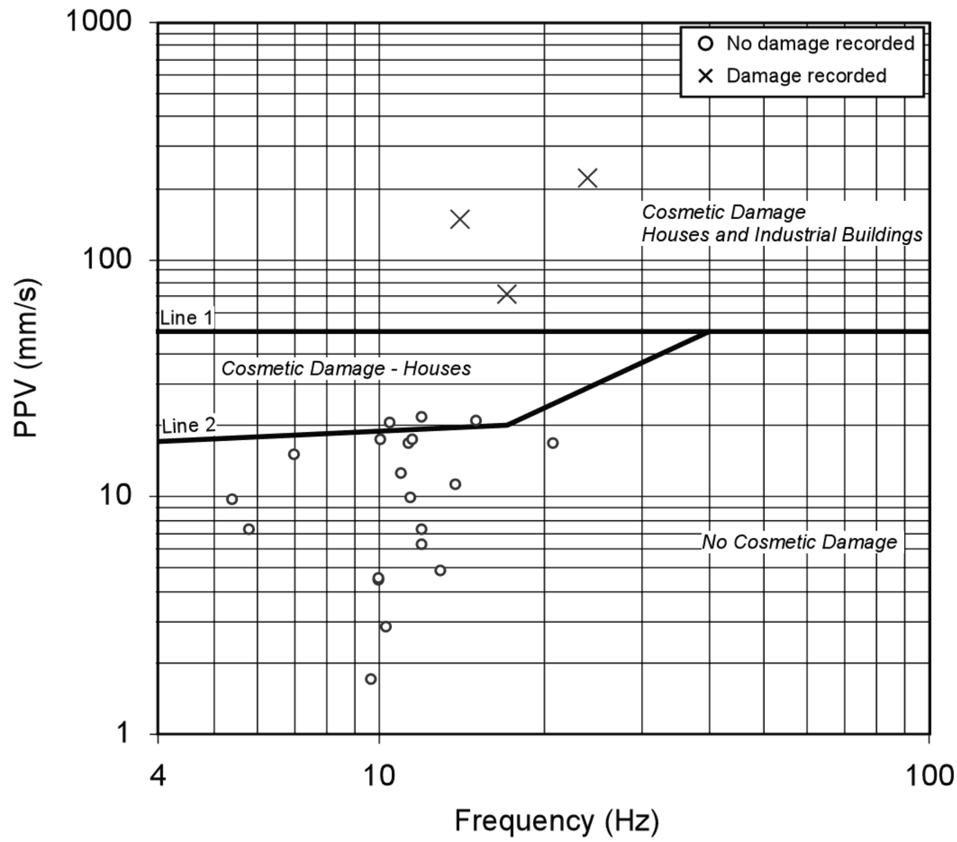


Figure 16 – Transient vibration guide values for cosmetic damage (BS7385: Part 2: 1993) and ACARP Investigation C9040 damage observations

6.1.6.5 Level Loop Survey

The relative levels of a course of brickwork around the house were measured on 8th June and 22nd November 2000 and indicated some differential movement over this period, as shown in **Figure 17**.

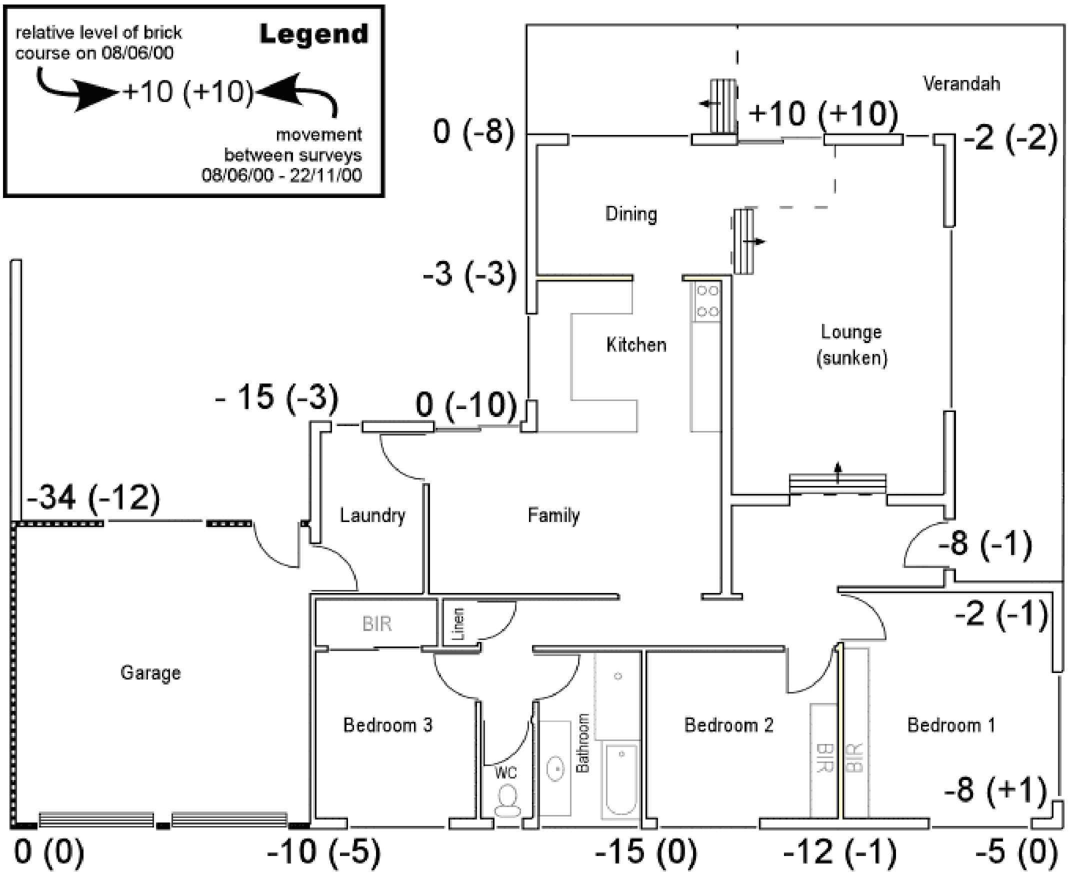


Figure 17 – Level loop survey - 8th June and 22nd November 2000

The maximum variation between surveys was –12 mm in the south-east corner of the garage and a 10 mm rise in the middle of the eastern wall, near the sliding doors. The levels measured by individual surveys included variations in tolerance of the original bricklaying, which could be ±5 mm. Relative measurements between surveys indicated footing movement.

The period of assessment included only minor rainfall events and only one significant blast with a PPV exceeding 50 mm/s. The changes of level are likely to be a function of soil drying conditions.

6.1.6.6 Structural Response Monitoring

Test House No. 1 experienced some 33 blasts between 1st March 2000 and 21st December 2000, with charge masses varying from 50 kg to 1300 kg, at distances between 50 metres and 1000 metres. The peak particle velocity (PPV) measured on the ground, adjacent to the house, varied between 1.5 mm/s and 222 mm/s. **Table 2** summarises peak vibration levels recorded, together with distance and maximum charge mass.

There were four events in excess of 50 mm/s and two events in excess of 100 mm/s. Blasting at the mine continued after the 21st December 2000 at more remote locations.

Fifteen accelerometers were used to measure vibrations in different locations in the house, as shown in **Figure 18**.

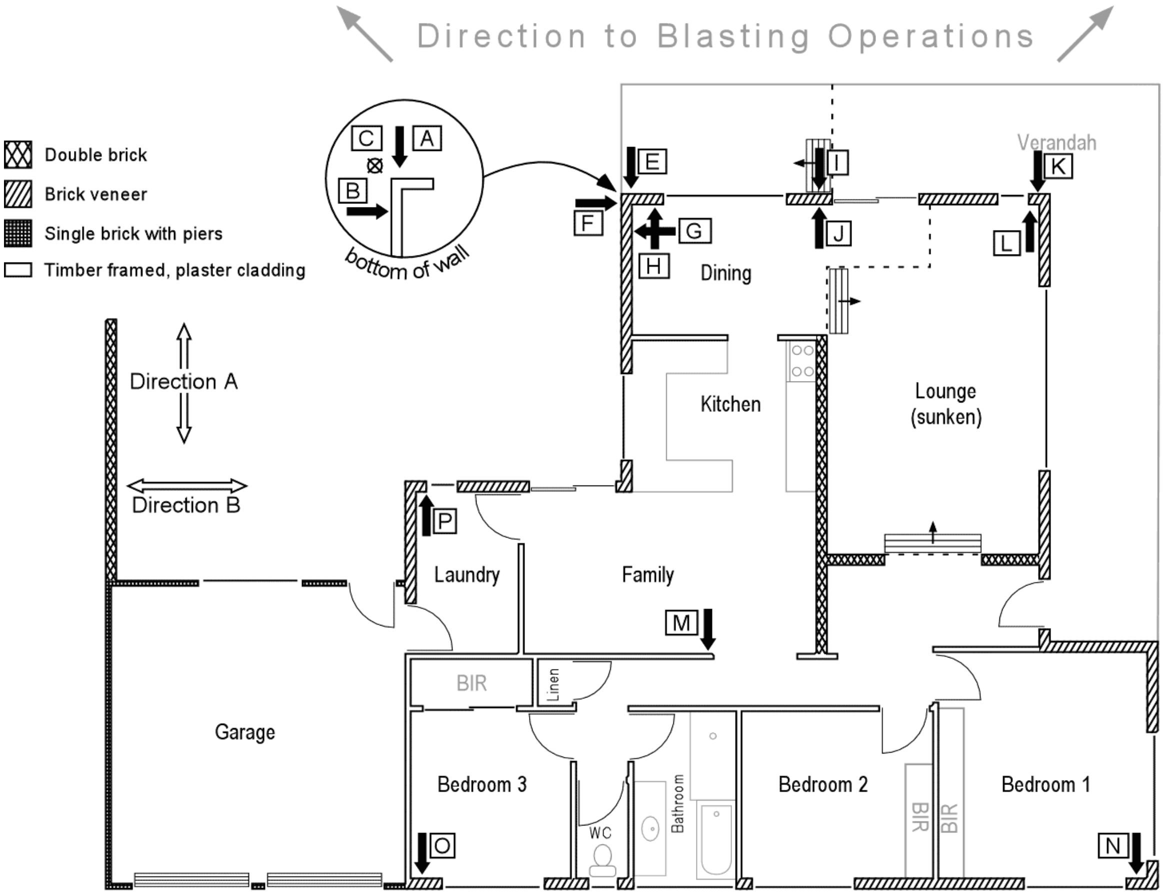


Figure 18 – Locations of the accelerometers

Three accelerometers (A, B and C) were located at ground level to measure the two horizontal and one vertical component of acceleration. The remaining twelve accelerometers were orientated in the horizontal direction at ceiling level (approximately 2.4 metres above floor level) with six on the external brick veneer walls (E, F, H, I, K, P) and eight on the internal plasterboard (G, J, L, M, N, O).

Ambient vibration tests indicated that the natural frequency of the house was in the order of 10 Hz in one direction and 8 Hz to 9 Hz in the other direction, which are typical values for single storey domestic structures.

The acceleration recordings, in both the time and frequency domains, were obtained for each of the 15 channels for the 17 blasts (whole house measurements are shown in **Table 14**). The acceleration time histories were then integrated and double integrated to obtain the velocity and displacement time histories, respectively.

Structural Response Analysis is detailed in Section 7.1 of this report and Direct Strain Measurement Comparisons are made in Section 8.1.

6.1.7 Conclusions

The Rix's Creek test house is of conventional brick veneer construction, typical of its 25 to 30 year age. The quality of brickwork is variable, the foundations are below current requirements, and evidence of sub-standard building practice (which is considered normal) during construction became apparent on close inspection. There were no expansion or articulation joints in the masonry to limit cracking due to movement, as required by the current Standard.

At the commencement of monitoring, the house had a number of minor cracks in the brickwork and plaster, ranging from hairline (<1 mm) to noticeable but easily filled (<5 mm). Inspection showed the cracks resulted from either movement of the sub-standard footings due to reactive clay soil and poor drainage or sub-standard building practice.

Geotechnical investigations of the foundation soil proved the characteristic surface movement to be between 12 mm and 29 mm for three test borehole sites. A brick course level survey showed variation of up to 34 mm in the level of a course of bricks around the house. This is probably a combination of foundation movement and normal bricklaying tolerance.

Lack of provision for the overflow from concrete water storage tanks caused water to flow beneath the garage floor and pond beneath the house. The areas of ponding corresponded to the widest cracks in the external brickwork, ie. external to the kitchen and the south-west corner.

During the study period, the house was subjected to peak ground vibration ranging from 1.5 mm/s to 222 mm/s and peak air vibration in excess of 145 dBL.

The blasting operations resulted in the house suffering minor and easily repairable damage. Blast No. 23 (71.2 mm/s), which caused the highest ground vibration up to that time, resulted in the plaster nails pulling through the backing paper of a small section of ceiling plaster in the lounge room.

Investigation showed the plaster had been installed without adhesive (sub-standard practice) on two joints. The plaster was reconnected to the joists using adhesive and remained intact for the remainder of the project, as did the remainder of the ceilings.

The minor damage observed and measured was the extension of the existing plaster cracks, opening of new hairline cracks and the cracking of plaster around nail heads. The crack width gauges of two cracks showed a permanent crack width increase of 0.12 mm and 0.3 mm, following blasts with vibration of 190 mm/s and 222 mm/s, with the widest movement limited to one crack in a low, poorly constructed, non-structural brick wall in the living room. The permanent crack widening is less than the dynamic movement measured and projected to 222 mm/s, which indicates some closing after being opened by the vibration and some elasticity remaining in the wall. The other five cracks showed no permanent widening.

As a contrast, all cracks widened between 0.2 mm and 2.2 mm in the period after blasting concluded, following a period of high rainfall. The crack length survey shows a demonstrable increase in crack lengths for the four blasts above 70 mm/s.

No damage was observed to ceramic tiles in the hallway, laundry and bathroom. No damage was observed in the concrete floor slabs of the garage and veranda. There was no glass damage. There was no additional damage to the external brickwork. The only additional damage attributable to blasting was minor damage to the internal plaster that could be easily repaired by filling and painting.

At the locations where the crack width was monitored, the movement is complex and relates more closely to ground moisture variation than to blasting events. There was considerable movement of crack widths after the conclusion of blasting that is clearly related to high rainfall episodes.

The lack of observed damage to the masonry following exposure to blast vibration of 220 mm/s should be discussed. Masonry and plasterboard have similar failure stain limits (see Section 10.1) and it may be anticipated that vibration levels that cause failure in plaster would also cause failure in masonry. However, the masonry of the house was cracked by foundation soil movement prior to the commencement of blasting. The cracks effectively articulated the masonry into smaller, stiffer elements, which rendered the whole structure more flexible, in a similar manner to the current code requirements by the provision of control joints.

Because the cracked masonry is flexible, any distortion due to vibration occurs at the articulations rather than being transferred to the intact masonry elements and they are thereby not exposed to strains that will cause further cracking.

Unarticulated and uncracked masonry houses may, therefore, be cracked by vibration levels similar to those that crack plasterboard (found to be 70 mm/s for Test House No. 1). Articulated or cracked masonry houses may be exposed to higher levels of vibration without causing additional crack development. Because plasterboard is attached to a frame, which is a continuous structural element, prior cracking of the plasterboard does not articulate the structure and it is thereby subject to the full vibration induced flexure. Plaster, therefore, cracks at the expected failure strain induced by the vibration, ie. about 70 mm/s.

6.2 Test House No. 2 (Wybong Road, near Bengalla Mine, Muswellbrook, N.S.W.)

Bengalla mine is located 4.5 km west of Muswellbrook township and blasting operations are conducted at distances between 700 metres and 2000 metres from this test house.

Typical overburden blasting specifications are shown in **Table 6**.

Table 6 - Typical Bengalla blast specification

Blasthole Diameter:	200 mm
Burden:	7 – 8 m
Spacing:	6.5 – 7 m
Inclination:	10^{o}
Face Height:	35 m
Stemming Height:	5 – 6 m
Stemming Material:	Angular gravel
Charge Mass/Delay:	800 kg

6.2.1 House Description

Test House No. 2 is approximately 25 years old. Due to the sloping nature of the site, the house has two sections, one is single storey and the other double storey. The single storey section is of conventional brick veneer construction with strip footings and brick piers. The double storey section has a slab on the ground foundation with brick piers and steel beams supporting the second floor. The walls consist of a single skin of brick between the piers. The laundry has a rendered finish. The second storey is conventional brick veneer construction.

The house has a timber frame, 10 mm plasterboard interior lining, tiled roof, wooden floors and aluminium framed windows and sliding doors. A floor plan of the house is shown in **Figure 19a** and a front view of the house in **Figure 20**. There are no expansion or articulation joints in the masonry to control movement, as required by the current Standard.

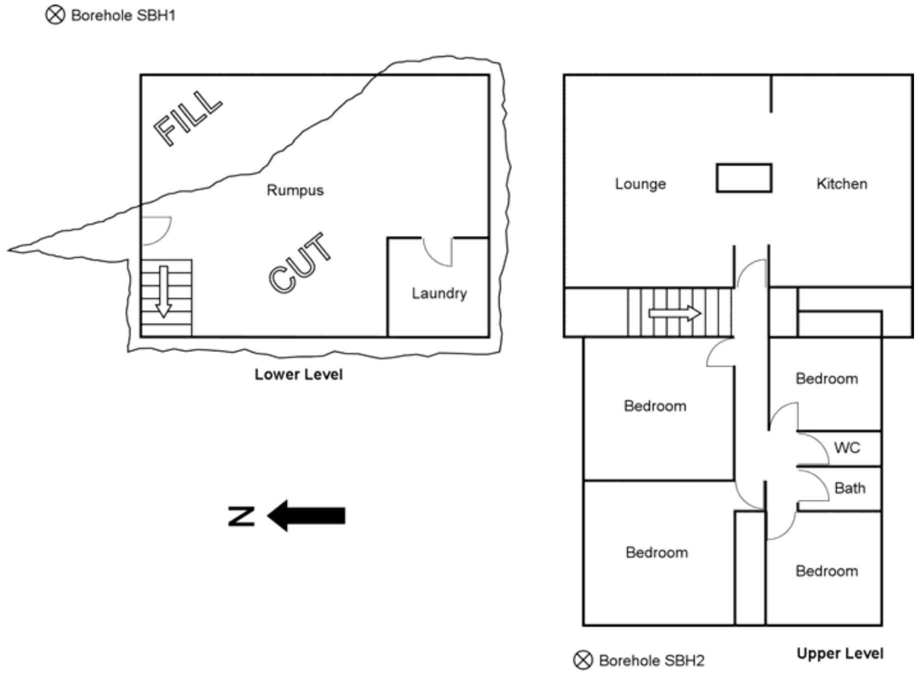
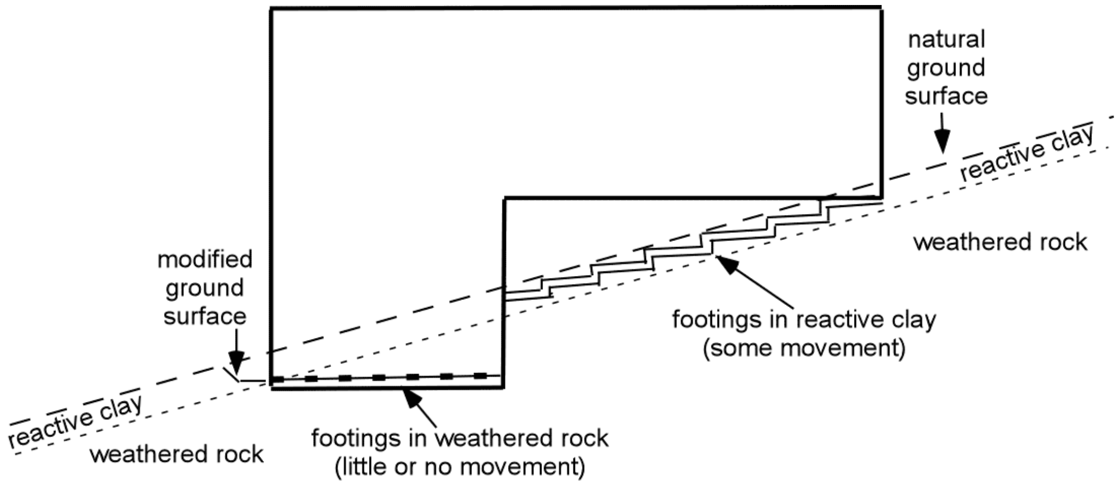


Figure 19a – Floor plan Test House No. 2



19b – Schematic section showing footings

Figure 20 – View of house from north-east

Prior to the commencement of blasting at Bengalla, the house had a number of minor structural defects relating to foundation movement and settlement. Photographs of typical defects are shown in **Figures 21a** to **21f**.

Figure 21a- Bulging plaster above the stairs

Figure 21b – Tapered stepped crack in brickwork at entrance (up to 8 mm wide)

Figure 21c – Close up of Tapered stepped crack in brickwork at entrance (Figure 21b)

Figure 21d – Cracks in brickwork over door in rumpus room

Figure 21e – Cracks in rendered finish in laundry

Figure 21f – Close up of vertical crack in rendered finish in laundry (Figure 21e)

6.2.2 Structural Tests

The brickwork has been constructed from extruded and fired clay bricks. The testing of the bricks (6 bricks) and mortar to AS3700-1998 gave the results listed in **Table 7**.

Table 7 – Brick and masonry test results – Test House No. 2

Unconfined compressive brick strength:	9.6 MPa (mean)
	7.4 – 12.1 MPa (range)
Characteristic compressive strength:	6.4 MPa
Flexural strength of masonry (bond wrench):	1.25 MPa (mean)
	0.92 – 1.45 MPa (range)
Characteristic flexural strength of masonry:	0.59 MPa

The bricks have a lower compressive strength than the other two houses considered in this study but are of reasonable quality. The bond strength is above average for domestic construction and is much better than Test House No. 1 and consistent with Test House No. 3.

6.2.3 Recorded Blast Vibration Levels

A total of 16 blasts were monitored with peak ground vibrations ranging from 0.62 mm/s to 3.02 mm/s and airblast levels up to 119.7 dBL, as listed in **Table 8**.

Table 8 – Blast monitoring Wybong Road House

	Station A		Station B	Station C	Station D
Date	PPV Ground (mm/s)	PAV (dBL)	PPV Verandah (mm/s)	PPV Laundry (mm/s)	PPV Eaves (mm/s)
27/03/01	2.11	104.6	-	1.44	12.7
02/04/01	1.21	98.5	1.23	0.8	6.5
02/04/01	1.5	101.9	1.52	1.15	-
09/04/01	1.06	90.6	0.99	0.75	-
20/04/01	0.65	95.9	1.03	-	-
26/04/01	1.36	101.2	1.13	0.84	7.7
26/04/01	-	-	-	-	7.0
27/04/01	-	-	-	-	3.1
30/04/01	1.31	-	1.39	-	7.7
01/05/01	2.37	109.1	-	-	7.3
09/05/01	0.62	97.8	0.52	0.57	-
05/06/01	-	-	-	-	4.7
29/06/01	2.96	111.3	-	-	11.4
09/07/01	3.02	119.7	-	-	13.5
20/07/01	1.96	-	-	-	-

6.2.4 Foundation and Geotechnical Report

The house is constructed on a sloping elevated area cleared and planted with occasional fruit trees and garden beds. The construction has involved significant cutting of the site to accommodate the lower level rooms.

Two boreholes, located as shown in **Figure 19**, were drilled and sampled. The soil profile can be summarised as:

- Up to 350 mm of (ML) sandy silt topsoil, low plasticity, pale grey with fine sand and a trace of gravels and assessed to be relatively non-expansive, overlying.
- 300 mm to 600 mm of (CH) clay, high plasticity, orange brown with a trace of fine to medium gravels, residual after sandstone but without relict rock structure and a shrink-swell instability index of 3.1%, indicating a medium reactive potential, overlying.
- White sandstone, of variable grain size, extremely weathered.

Based on the methods and classification criteria of AS2870-1996, the characteristic surface movement is estimated to be 21 mm for the average sub-surface site conditions and the site is classified as Class M, or moderately reactive, although it falls at the boundary of the lower classification of Class S (slightly reactive).

The strip footings were examined adjacent to hole SBH2 and were found to be 250 mm to 280 mm deep. By the provision of AS2870-1996, a footing of 400 mm is regarded for a masonry veneer dwelling on a Class S site, and a depth of 500 mm is required at a Class M site. A schematic section of the house showing the footings and excavation is shown in **Figure 19b**.

The foundations of the house may, therefore, be regarded as inadequate with the possibility that the performance of the foundations may fall outside the bounds considered as satisfactory in Appendix C of AS2870-1996. The code nominates the possibility that damage exceeding Category 2 may occur due to reactive clay foundation movements. The damage categories are listed in **Table 9**.

The defect photographs in **Figures 21b** and **21c** show that the masonry damage in the entrance may be considered as Category 3. This crack appears to be caused by lateral expansion of the reactive clay layer pushing the upper section of the bricks towards the south.

Table 9 – AS2870-1996 - classification of damage with reference to walls

Description of Typical Damage and Required Repair	Approximate crack width limit (mm)	Damage Category
Hairline cracks	<0.1	0
Fine cracks which do not need repair	<1.0	1
Cracks noticeable but easily filled. Doors and windows stick slightly.	<5.0	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weather tightness often impaired.	5 – 15 (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted.	15 – 25 (but also depends on number of cracks)	4

6.2.5 Crack Survey

Due to likely reactive soil foundation movements, a number of cracks and other defects had developed in the structure of the house prior to the commencement of blasting at Bengalla on 28th August 1998 and were there when the monitoring aspect of this project began on 27th March 2001. The crack patterns are consistent with foundation movement and general occupation of the house.

The growth of all cracks was monitored and recorded between 27th March 2001 and February 2002. The width of five cracks in the plasterboard and brickwork has been accurately monitored with DEMEC gauge measurements between installed targets. The monitored cracks are described and maximum movements listed in **Table 10**.

Table 10 – Maximum movements of monitored cracks

Target No.	Location	Internal/ External	Description	Movement (mm)
1	Rumpus Room	Internal	Masonry stepped crack near entrance	0.18
2	Laundry	Internal	Vertical crack in rendered plaster	0.12
3	N/E Corner	External	Stepped crack in brick wall	0.10
4	Living Room	Internal	Plaster crack	0.10
5	Living Room Arch	Internal	Plaster crack 45^{o}	0.27

Crack movement diagrams are shown in **Figure 22**, together with ground vibration measurements and daily rainfall totals.

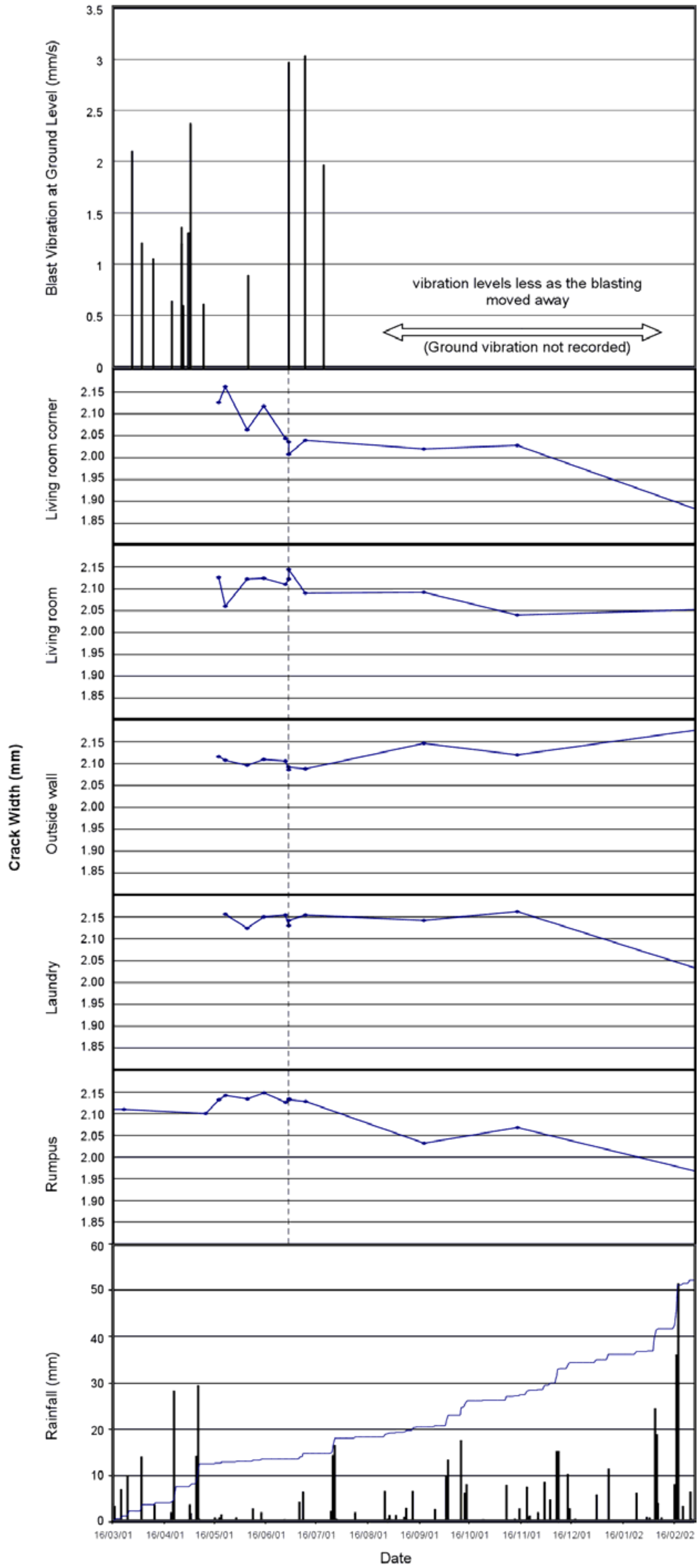


Figure 22 – Crack movement diagram Test House No. 2 (Wybong Road)

The vibration levels were much lower than Test House No. 1 and there was no clear correlation between blast vibration levels and crack growth. There was a period of pronounced crack movement following a high rainfall period in January to February 2002. The blast vibration levels after the monitoring period were less than 3 mm/s because the blasting had moved away from this end of the pit.

Systematic monitoring of blasts was discontinued after July 2001 due to the difficulty in obtain data and the low vibration levels and lack of structural response.

6.2.6 Level Loop Survey

The relative levels of two courses of bricks representing top and ground floors were measured during January 2001 and February 2002. The original survey showed the bottom course of bricks to vary by 16 mm from level and the course above the first floor level to vary by 34 mm (refer **Figure 23**). In between surveys, there was a maximum of 6 mm difference in relative levels on the bottom course and a maximum of 10 mm in the first floor course.

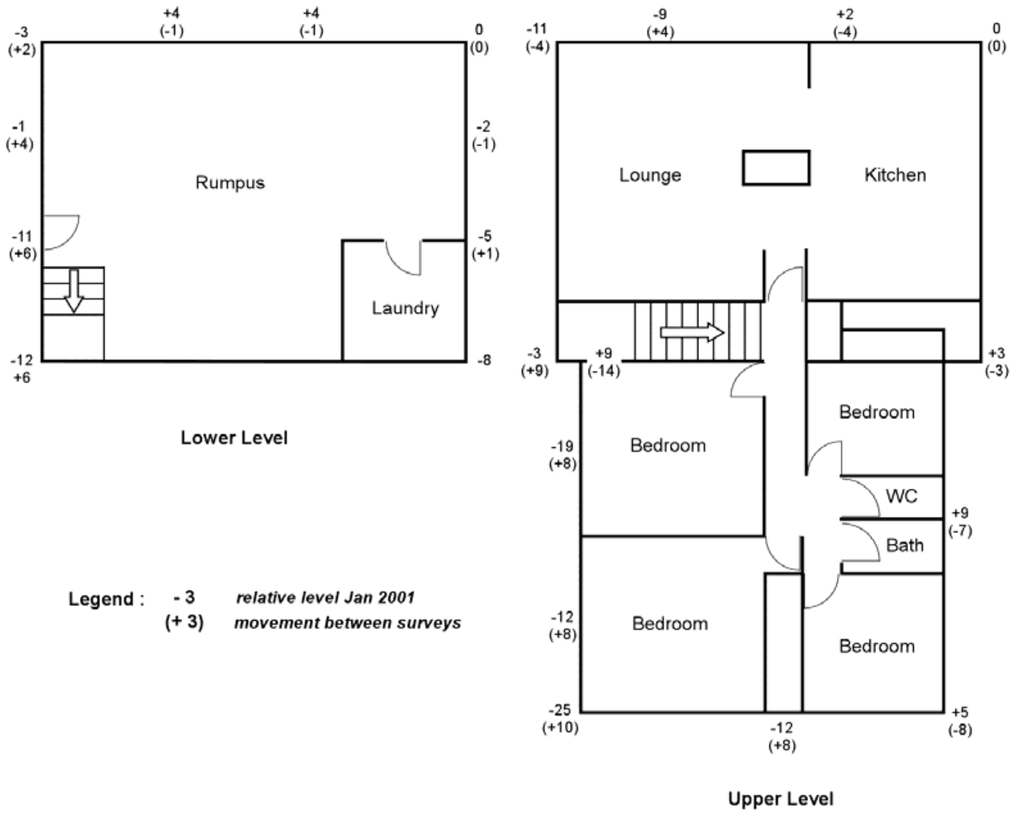


Figure 23 – Level loop survey Test House No. 2 (Wybong Road)

Part of the original variation in level can be explained by the allowable trade bricklaying construction tolerance of ±5 mm/10 metre length (10 mm overall), but the location of the movement in relation to the cut/fill/natural surface suggests that foundation movements are responsible with the amount of variation consistent with that suggested by the geotechnical investigation.

The relative movement between surveys shows an irregular uplifting that is consistent with foundation clays expanding after a high rainfall period in February 2002.

6.2.7 Blast Vibration and Structural Response

The accelerometers, geophones and strain gauges were placed in the positions shown in **Figure 24**. The blast vibrations measured during this investigation are listed in **Table 8**. Refer to Section 7.2 for the Structural Response Analysis and Section 8.2 for Direct Strain Comparisons.

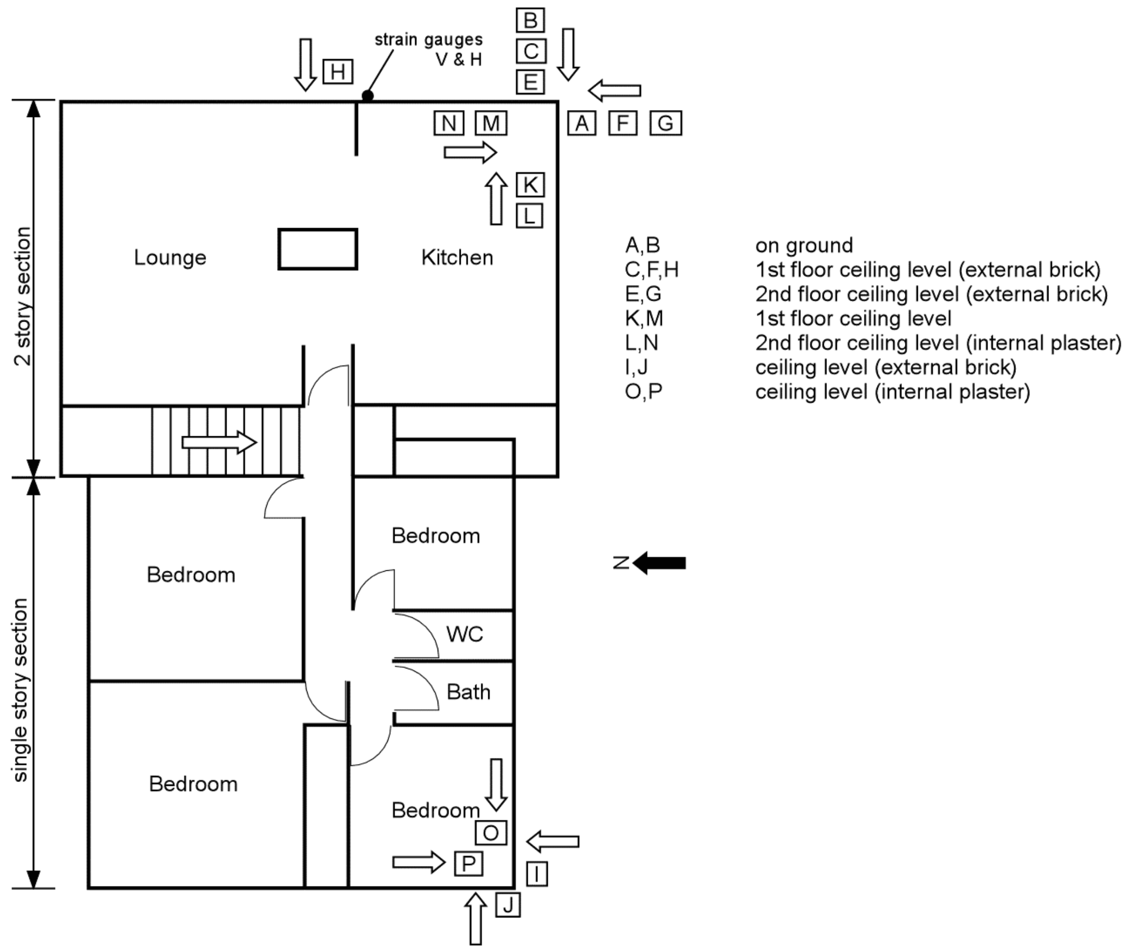


Figure 24 – Instrumentation of Test House No. 2

6.3 Test House No. 3 (Racecourse Road, near Bengalla Mine, Muswellbrook, N.S.W.)

6.3.1 House Description

Test House No. 3 is a fairly new (<5 years old) conventional single storey brick veneer house, located on a slightly elevated terrace within a broad alluvial flat, adjacent to Race Course Road. Blasting operations at the Bengalla mine are conducted from 1700 metres to 2400 metres from this house. The house is a conventional brick veneer construction, with a timber frame and plasterboard lined walls. It has a tiled roof, paved verandah and paved swimming pool surrounds at the rear. The floor plan is shown in **Figure 25a.** The floor is a stiffened concrete slab with deep edge beam footings (see **Figure 25b**). A photograph of the front of the house is shown in **Figure 26**.

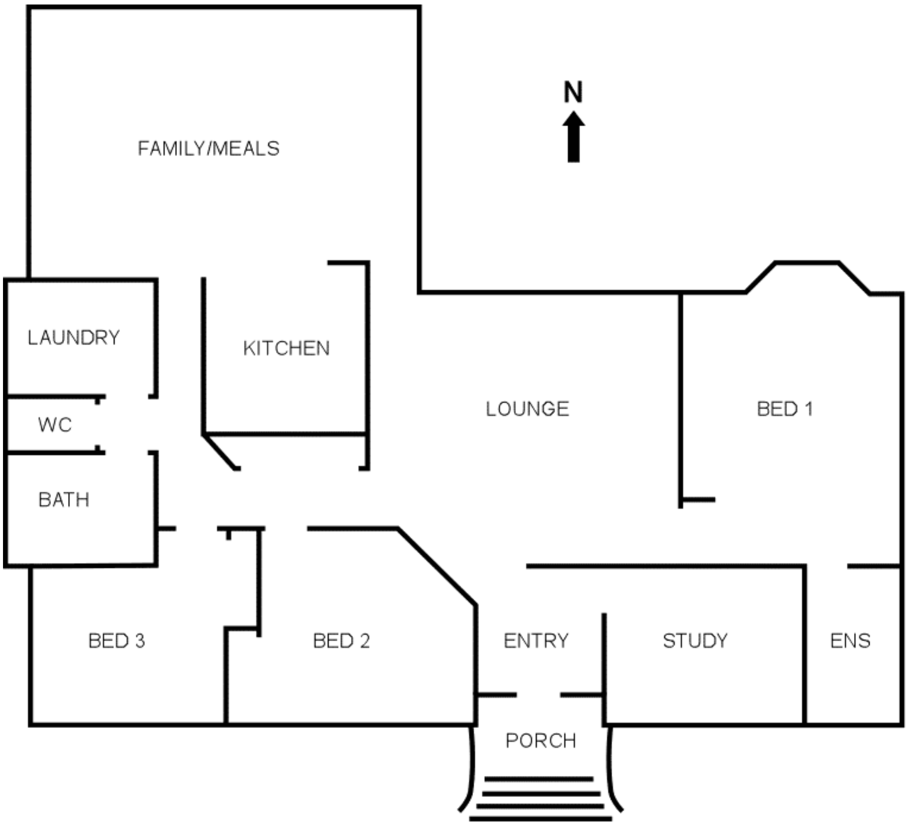


Figure 25a – Floor plant of Test House No. 3 (Racecourse Road)

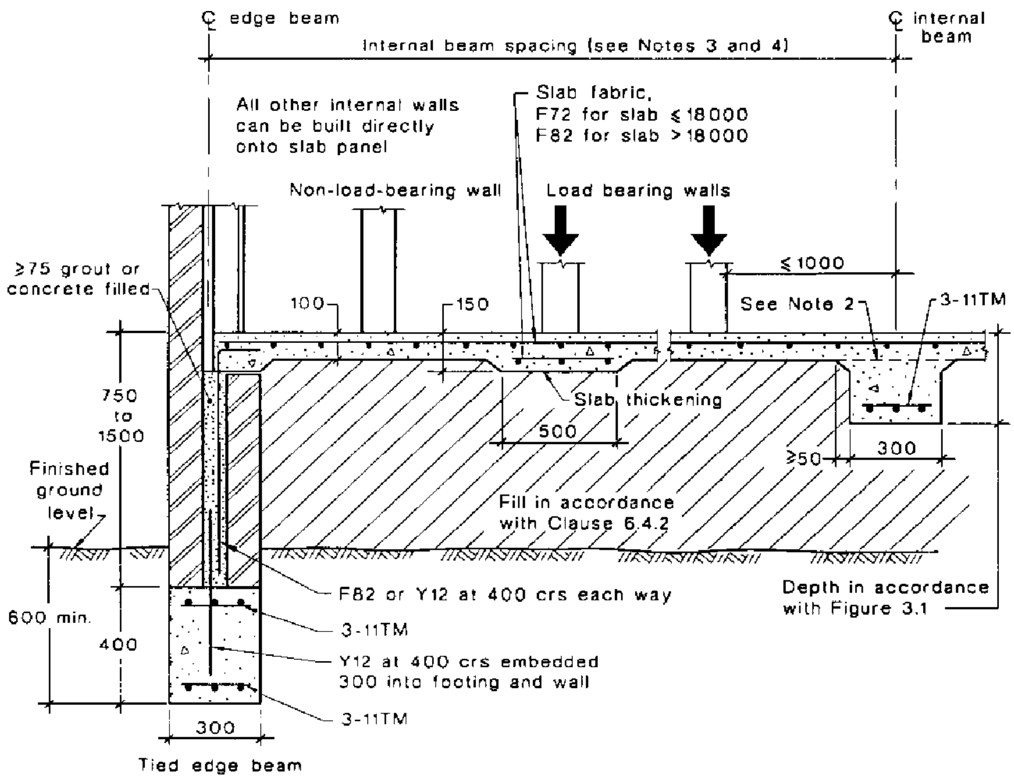


Figure 25b – Schematic of footings

Figure 26 – Front view of Test House No. 3 (Racecourse Road)

The house was selected because it was a typical house that had no visible external brickwork cracks and would permit the monitoring of the structural response of a brick wall to low air and ground vibration levels. It is important to note that the house construction did not include the provision of control joints (to control and limit movements) of the masonry and prevent uncontrolled cracking.

6.3.2 Structural Tests

The house is constructed of extruded and fired clay bricks. The testing of the bricks and mortar to AS3700-1998 gave the results listed in **Table 11**.

Table 11 – Brick and masonry test results

Unconfined compressive brick strength:	26.3 MPa (mean)
	21.4 – 32.2 MPa (range)
	18.4 MPa (characteristic)
Flexural strength of masonry (bond wrench):	1.21 MPa (mean)
	0.84 – 1.51 MPa (range)
	0.54 MPa (characteristic)

The bricks are good quality and the flexural strength higher than average for normal domestic construction.

6.3.3 Recorded Blast Vibration Levels

The blast vibration levels recorded at the Racecourse Road house during this investigation are listed in **Table 12**.

Table 12 – Blast vibration monitoring at Racecourse Road

Date	PPV Ground (m/s)	PPV Eaves (mm/s)	PAV (dBL)
18/05/01	0.29	-	101.3
22/05/01	1.08	-	98.4
31/08/01	0.96	-	101.4
05.10.01	0.9	-	96.5
22/10/01	0.49	0.6	104.0
08/11/01	0.8	0.9	98.0
06/12/01	0.29	0.35	87.7
06/12/01	1.11	-	100.5

6.3.4 Foundation and Geotechnical Report

The foundation soils at Test House No. 3 are irregularly stratified alluvial soils consisting of silty CLAYS, sandy silty CLAYS, clayey SILTS, gravely SANDS, silty SANDS, clayey silty SANDS and silty clayey SANDS that are at least 4.8 metres deep. The gravel content increases with depth after 1.5 metres. At 4.8 metres the Cone Penetrometer Rig met with refusal, indicating a very dense gravel bed.

Tests on the clay rich soil layers returned a shrink-swell index value of 3.7% pF, which by the method of Section 2.2.3 of AS2870-1996 gave a characteristic surface movement of as much as 44 mm. The site is, therefore, classified as Class H (highly reactive), although it falls close to the boundary of the lower Class M (moderately reactive).

The footings used in the construction of the house are shown in **Figure 25b**. The footings are suitable for a Class M site, as required by AS2870-1996, and while they could be considered inadequate for the site Class H category determined, they appear to be performing satisfactorily, as evidenced by the lack of masonry cracks and movement indicated by level loop surveys.

6.3.5 Crack Survey

The observation of cracks at this house was limited by the property owners to external brickwork, where no cracks were noted prior to monitoring and no cracks developed during the period of the project.

6.3.6 Level Loop Survey

Level surveys were conducted around the course of bricks above the damp-proof course in April 2001 and February 2002. The relative levels are shown in **Figure 27**. The maximum variation in the level of the brick course was 6 mm after the original survey, which is within construction tolerance, and 3 mm on the second survey. The maximum movement between surveys was +4 mm to –2 mm. This was insignificant compared to movements at the other test houses.

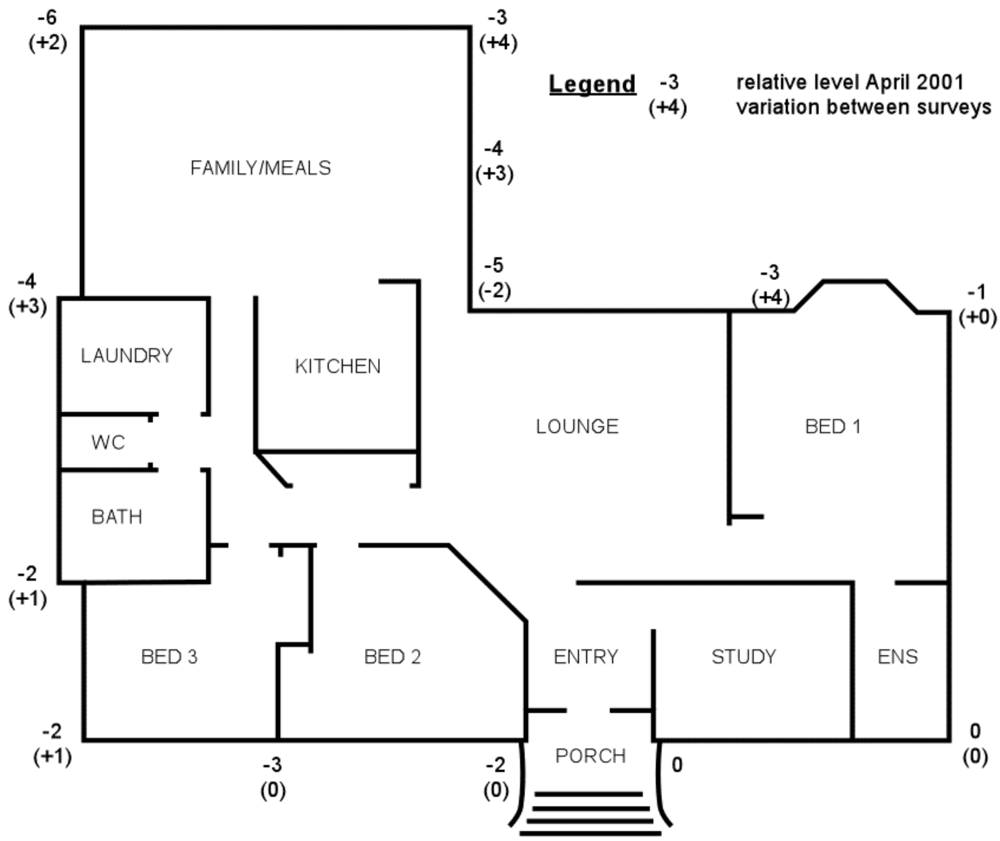


Figure 27 – Level loop survey results

The lack of relative movement shown by the level loop survey is reflected in the lack of brickwork cracking, which demonstrates the adequacy of the footing construction.

6.3.6 Blast Vibration and Structure Response

The accelerometers, geophones and strain gauges were placed on the house in the locations shown in **Figure 28**.

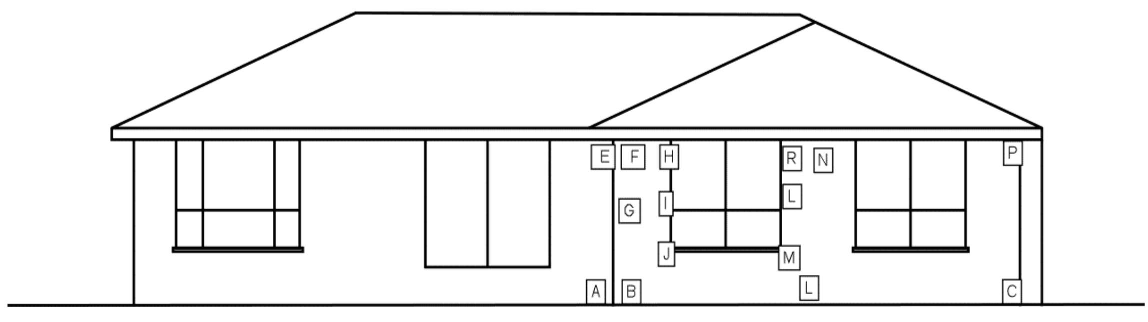


Figure 28 – Instrument locations Test House No. 3

The vibrations levels measured during this investigation are listed in **Table 12**. It should be noted that the level of vibrations recorded at this test house were very small (PPV mostly below 1 mm/s). For such very small vibrations the readings were relatively noisy and the records from the accelerometers were not highly reliable, particularly when the data is integrated and double integrated. Refer to Section 7.3 for the Structural Response Analysis of this house and Section 8.3 for Direct Strain Comparisons.

6.4 Reference Houses

When the project was conceived, it was thought that the behaviour of a control house in an area not subject to mine blast vibration or subsidence might be a useful comparison between houses subject to blast vibration and those not subject to blast vibration. Upon reflection, it was decided that a single control house would not serve the intended purpose.

If a house was chosen that had no cracks and developed no cracks during the investigation period, while those subject to vibration did, it could be interpreted as evidence that exposure to blast vibration (regardless of blasting levels) caused cracking. On the other hand, if a house was chosen which had cracks and those cracks were monitored over a period, it would also not provide conclusive evidence if the cracks did or did not alter. In either case, we could be accused of bias in the selection of a single control house.

After much discussion, it was decided that a number of control houses would be selected from the files of the Mine Subsidence Board, with similar construction to the three test houses. After careful examination of the damage observed in the test houses and damage reported in numerous control houses, it was clear that the type of damage recorded at the test houses at the commencement of this investigation was similar to damage recorded at houses unaffected by mine blasting and determined by the collective experience of Mines Subsidence Board Engineers to be unaffected by mine subsidence.

Photographs of typical defects in the control houses are shown in **Figures 29a** to **29v**.

Figure 29a –Reference House R2 (Edgeworth) – bed joint separation

Figure 29b – Reference House R4 (Fennell Bay 2) – bed joint separation

Figure 29c – Reference House R4 (Fennell Bay 2) – brick crack

Figure 29d – Reference House R7 (Cessnock) – diagonal brick crack

Figure 29e – Reference House R7 (Cessnock) – diagonal brick crack

Figure 29f – Reference House R7 (Cessnock) – diagonal brick crack

Figure 29g – Reference House R8 (Jewell) – diagonal brick crack

Figure 29h – Reference House R4 (Fennell Bay 2) – foundation brick crack

Figure 29i – Reference House R7 (Cessnock) – vertical brick crack

Figure 29j – Reference House R8 (Jewell) – vertical crack brick

Figure 29k – Reference House R4 (Fennell Bay 2) – vertical crack brick

Figure 29l – Reference House R8 (Jewell) – rendered masonry crack

Figure 29m – Reference House R8 (Jewell) – lining and cornice crack

Figure 29n – Reference House R6 (Kotara) – rotated corner

Figure 29o – Reference House R6 (Kotara) – relative movement

Figure 29p – Reference House R7 (Cessnock) – relative movement and slab crack

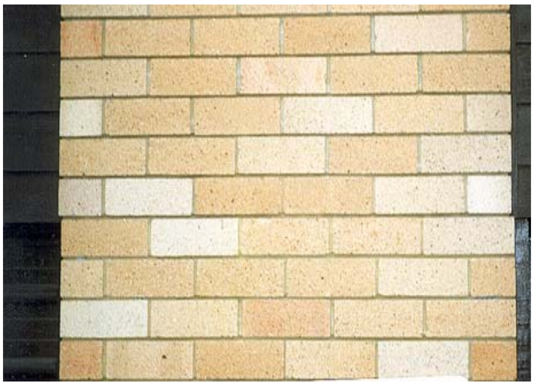


Figure 29q – Reference House R4 (Fennell Bay 2) – shear movement

Figure 29r – Reference House R8 (Jewell) – slab crack

Figure 29s – Reference House R4 (Fennell Bay 2) – slab crack

Figures 29t-v – 25-30 year old solid masonry, single storey house with rendered plaster at New Lampton

Figure 29t

Figure 29u

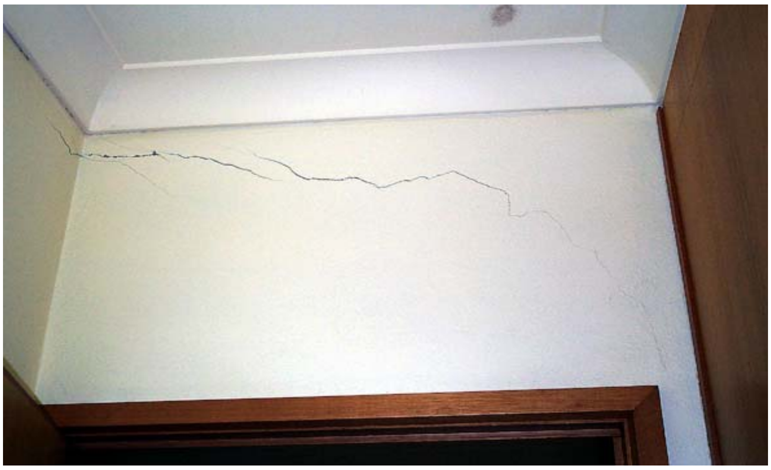


Figure 29v

Comparisons between the test houses and control houses are listed in **Tables 13a**, **13b** and **13c**.

Table 13a

	Test House No. 1	Ref. House R4 (Fennell Bay 2)	Ref. House R5 (Macquarie Hills)	Ref. House R6 (Kotara)
House Type:	Brick Veneer	Brick Veneer	Brick Veneer	Weather Board
Age (years):	30	24	30	50
No. of Storeys:	1	1	1	1
Footing Type:	Strip and Pad Footings	Strip and Pad Footings	Strip and Pad Footings	Strip and Pad Footings
Footing Thickness (m):	0.38			0.18
Founding Depth (m):	0.6			0.32
Footing Class:	S			S
Soil Type:	Residual	Alluvial/ Slopewash		Residual
Depth of Topsoil (m):	0.15	0.2	0.3	0.35
Depth of Clay (m):	0.4-0.7	0.6	>2.3	1.15
Depth to Rock (m):	0.6-0.9			1.5
Rock Type:	Sandstone			Siltstone
Clay Reactivity I_{gs} (%/pF):	2.0-4.4	0.4	0.9-3.6	4.7-6.0
Predicted Ground Movement (mm):	12-29	<20	25	45-60
Site Class:	M	S	M	H
Type of Damage:	1. LC 2. BJS 3. DBC 4. DS	1. LC 2. BJS 3. DBC 4. RM 5. DS	1. DBC 2. DS 3. CR	1. CC 2. VBC 3. FBC 4. RM
Crack/Movement Width (up to, mm):	1. 3 2. 3 3. 10 4. 25	1. - 2. 10 3. 4 4. 7 5. 40	1. 2-3 2. 40 3. -	1. - 2. 10 3. 6-8 4. 15-20
Damage Classification:	Slight	Slight	Moderate	Severe

M	=	corresponds to characteristic ground surface movement between 20 and 30 mm
S	=	corresponds to characteristic ground surface movement <20 mm
H	=	corresponds to characteristic ground surface movement between 40 and 70 mm
LC	=	cracking of linings (undifferentiated)
BJS	=	bed joint separations
DBC	=	diagonal cracking in brickwork at openings
RM	=	relative movement between structural components
DS	=	differential settlements
CR	=	corner rotations
CC	=	cornice cracking
VBC	=	vertical cracking in brickwork
FBC	=	cracking in foundation work

Table 13b

	Test House No. 2	Ref. House R1 (Fennell Bay 1)	Ref. House R7 (Cessnock)	Ref. House R8 (Jewell)
House Type:	Brick Veneer	Brick Veneer	Brick Veneer	Cavity Brick/ Brick Veneer
Age (years):	30	22	22	12
No. of Storeys	Split Level	Split Level	2	2
Footing Type:	Strip and Pad Footings	Strip and Pad Footings	-	Strip and Pad Footings
Footing Thickness (m):	0.28	0.35		0.3
Founding Depth (m):	0.35	0.50		0.9
Footing Class:	A	A/S		A
Soil Type:	Residual	Residual	Residual	Residual
Depth of Topsoil (m):	0.35	0.15	0.25	0.3
Depth of Clay (m):	0.3-0.6	0.8-0.9	0.7	1.0
Depth to Rock (m):	0.6-0.9	0.9	1.0	1.5
Rock Type:	Sandstone	Siltstone	Sandstone	Siltstone
Clay Reactivity I_{gs} (%/pF):	3.1	3.4	1.4	6.0
Predicted Ground Movement (mm):	21	-	15-20	65
Site Class:	M	H	S	H/E
Type of Damage:	1. LC 2. BJS 3. VBC 4. DBC 5. CC 6. DS 7. BP	1. LC 2. FBC 3. CR 4. DWJ	1. DLC 2. BJS 3. DBC 4. DWJ	1. CC 2. LC 3. DBC 4. DS
Crack/Movement Width (up to, mm):	1. 3 2. 1 3. 3 4. 10 5. 1 6. 25 7. 25	1. - 2. 15 3. - 4. -	1. – 2. – 3. 3 4. -	1. – 2. – 3. 5 4. 30
Damage Classification:	Moderate	Moderate	Slight	Severe

A	=	corresponds to little or no characteristic ground surface movement
S	=	corresponds to characteristic ground surface movement <20 mm
M	=	corresponds to characteristic ground surface movement between 20 and 30 mm
H	=	corresponds to characteristic ground surface movement between 40 and 70 mm
LC	=	cracking of linings (undifferentiated)
BJS	=	bed joint separations
VBC	=	vertical cracking in brickwork
FBC	=	cracking in foundation work
CR	=	corner rotations
DWJ	=	doors and windows jamming
BP	=	bulging plaster
CC	=	cornice cracking
DS	=	differential settlement

Table 13c

	Test House No. 3	Ref. House R2 (Edgeworth)	Ref. House R3 (Chain Valley Bay)
House Type:	Brick Veneer	Brick Veneer	Brick Veneer
Age (years):	5	8	14
No. of Storeys:	1	1	1
Footing Type:	Suspended Slab with Reinforced Edge Beams	Strip Footings/Infill Slabs	Strip Footings/Infill Slabs
Footing Thickness (m):	0.4	0.3	
Founding Depth (m):	0.6	0.6	
Footing Class:	M	A	
Soil Type:	Alluvial/Slopewash	Residual	Residual
Depth of Topsoil (m):		0.15	0.35
Depth of Clay (m):	0.6-0.8	>1.35	0.5-0.6
Depth to Rock (m):		>1.5	1.3-1.5
Rock Type:			Sandstone
Clay Reactivity I_{gs} (%/pF):	3.7	3.7	5.0
Predicted Ground Movement (mm):	44	35-45	45-55
Site Class:	H	M/H	H
Type of Damage:	none (external	1. CC 2. BJS 3. RM 4. DWJ	1. CC 2. DLC 3. VBC 4. BJS 5. DS
Crack/Movement Width (up to, mm):		1. - 2. 12 3. 40 4. -	1. - 2. - 3. 2-3 4. – 5. 14
Damage Classification	Insignificant	Severe	Slight

H	=	corresponds to characteristic ground surface movement between 40 and 70 mm
M	=	corresponds to characteristic ground surface movement between 20 and 30 mm
CC	=	cornice cracking
BJS	=	bed joint separations
RM	=	relative movement between structural components
DWJ	=	doors and windows jamming
DLC	=	diagonal cracking in linings at openings
VBC	=	vertical cracking in brickwork
DS	=	differential settlements

The constant theme through the comparisons is that the houses were built with footings inadequately engineered relative to currently adopted standards to withstand the movement caused by the reactivity of the footing soil. It was not determined how representative these houses are or how many houses are not effected by inadequate foundations. Anecdotal evidence by Newcastle University personnel and Mr John Berthon (Berthon & Associates) suggests that reactive clay is common throughout the Hunter Valley. Many houses built in the past to the standards current at the time have been found, from subsequent experience, to have inadequate foundations and have shown damage ranging from slight to severe. On the other hand, houses with footings engineered to current standards are quite stable, as evidenced by the performance of Test House No. 3.

This project does not attempt to compare the number of new houses that crack with the number that do not crack in the absence of blast vibration.

7. STRUCTURAL RESPONSE AND BLAST VIBRATION

7.1 Structural Response

The overall damage in a residential structure due to blasting is directly correlated with the in-plane distortion of the walls between the ceiling and floor. The in-plane distortion is often measured in terms of the drift ratio (γ) which is defined by the horizontal displacement (Δ_1) of the wall at the ceiling level divided by the wall height (H), as shown in **Figure 30**. The ceiling displacement can be estimated from the ground peak component velocity (V_g), the amplification (λ) of the velocity between the ground and ceiling and the dominant frequency (f) of the structure as shown in Equation [1b].

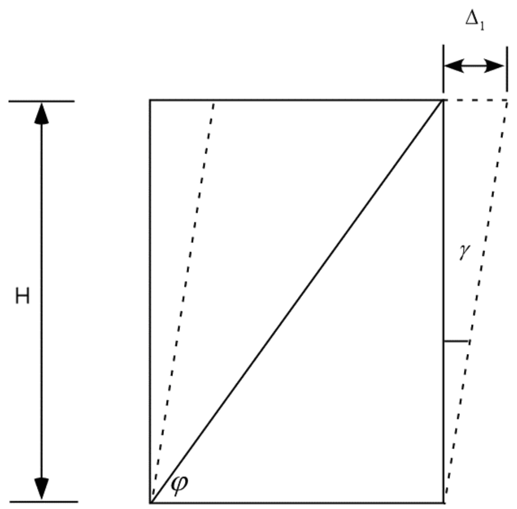


Figure 30 - Illustration of terminology for the drift γ and ceiling racking displacement Δ_1

$$\gamma = \frac{\Delta_1}{H} \qquad [1a]$$

$$\Delta_1 = \frac{V_g}{2\,\pi\,f}\lambda \qquad [1b]$$

$$\gamma = \frac{V_g}{2\,\pi\,f}\frac{\lambda}{H} \qquad [1c]$$

The drift ratio (γ) provides an estimate of the gross shear strain in a wall. However, damage occurs when the principle tensile strain of the material is exceeded and hence rupture occurs. The average principal tensile strain (ε) can be simply estimated from the gross shear strain using basic mechanics of solids principles as follows:

$$\in \;=\; \frac{\Delta_1}{H}\ \mathrm{Sin}\varphi\ \mathrm{Cos}\varphi\ \ (\mathrm{Sin}\varphi\ \mathrm{Cos}\varphi \text{ max when } \varphi \;=\; 45^{\circ}) \;=\; 0.5\,\gamma \qquad [2]$$

The bases for selecting the amplification and frequency values are described in the following sections.

7.2 Structural Monitoring Test House No. 1 – Rix's Creek

Fifteen accelerometers were used to measure vibrations in different locations in the house, as shown in **Figure 18**.

The vibration history, together with those blasts for which whole house measurements were taken, is listed in **Table 14**.

Table 14 – Details of all blasts occurring during the monitoring period – Test House No. 1 (Rix's Creek)

Blast No.	Date	Charge Mass (kg)	Distance (m)	PPV Ground (mm/s)	Whole House Measurements
1	01/03/00	300	231	18.4	x
2	20/03/00	1300	450	16.1	x
3	22/03/00	300	268	14.2	✓
4	27/03/00	200	363	4.5	✓
5	11/04/00	85	395	4.5	✓
6	13/04/00	150	306	7.3	✓
7	13/04/00	150	306	6.3	✓
8	04/05/00	1000	401	17.4	x
9	09/05/00	250	255	11.3	x
10	12/05/00	80	280	1.7	x
11	12/05/00	250	280	15.0	x
12	22/05/00	1000	260	20.5	✓
13	31/05/00	300	380	9.3	x
14	31/05/00	50	408	3.0	x
15	14/06/00	50	425	1.5	✓
66	14/07/00	50	247	4.7	x
17	18/07/00	150	418	6.9	x
18	25/07/00	-	-	8.0	x
19	28/07/00	30	-	4.9	x
20	07/08/00	200	214	16.6	x
21	07/08/00	50	447	2.9	x
22	08/08/00	1100	810	9.7	x
23	11/08/00	350	106	71.2	✓
24	11/08/00	250	333	9.6	x
25	17/08/00	300	106	17.3	✓
26	02/11/00	150	252	10.4	✓
27	06/11/00	200	-	13.0	✓
28	27/11/00	150	166	36.2	✓
29	28/11/00	300	135	73.6	✓
30	30/11/00	110	106	44.4	✓
31	07/12/00	350	55	190.0	✓
32	20/12/00	330	50	222.0	✓
33	21/12/00	1100	400	41.4	x
34	19/01/01	150	250	9.3	✓

Ambient vibration tests indicated that the natural frequency of the house was in the order of 10 Hz in one direction and 8 Hz to 9 Hz in the other direction, which are typical values for single storey domestic structures.

The acceleration recordings in both the time and frequency domains were obtained for each of the 15 channels for 17 blasts (the blasts with whole house measurements are listed in **Table 14**). The acceleration time histories were then integrated and double integrated to obtain the velocity time histories and displacement time histories, respectively.

The ratios of the peak component velocity (V_1) at ceiling level to ground level (V_g) were calculated to estimate the likely vibration amplification effects with height in the structure. The ratios were calculated for both the in-plane (shear) and out-of-plane (bending) directions for both the frame and brick veneer walls. The in-plane measurements are of vital importance from damage level perspective, whilst the out-of-plane records are less structurally significant but do contribute to the overall vibration and noise perception of the occupants. This is demonstrated in **Figure 31**.

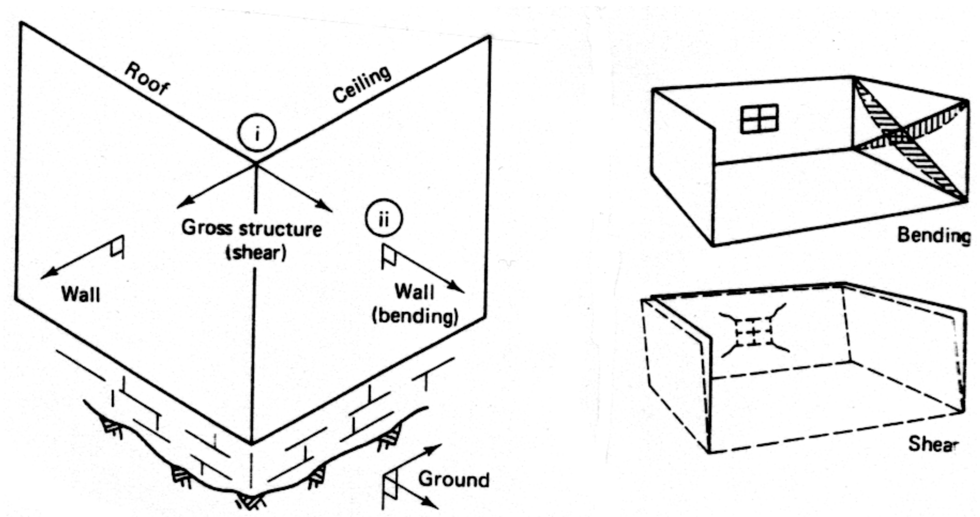


Figure 31 – Superstructure and wall response (Dowding, 1985)

7.2.1 Amplification Effects

The resulting amplification values varied significantly, depending on the level of ground vibration. **Figures 32a** and **32b** plot the in-plane amplification for the framed walls and brick veneer walls versus the ground PPV measured adjacent to the house. It should be noted that the PPV is always greater than the V_g, with the ratio of PPV to V_g typically in the range of 1-2. Most blast related regulations worldwide, including Australia, are based on PPV rather than V_g, and, hence, the values plotted are conservative.

An upper bound envelope has been fitted to the data, so that an approximate and conservative estimate of the amplification effects can be obtained. The amplification envelope can be described by a step function as follows:

$\lambda = 4.0$ for PPV $\leq$ 5mm/s [3a]

$\lambda = 2.0$ for PPV 5-100mm/s [3b]

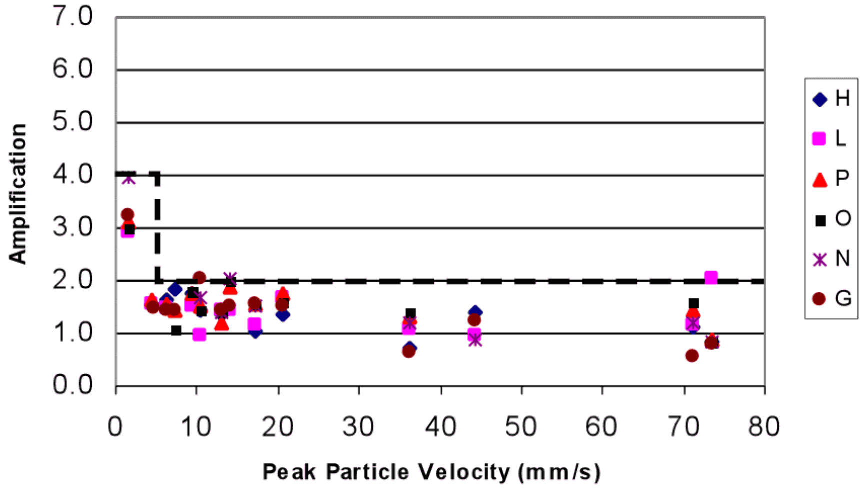
The proposed stepped function suits the data shown in **Figure 32a**, but there are no practical reasons for the step. Further research is required to investigate if a smoother amplification function is appropriate, especially in the range 3 mm/s to 15 mm/s. Beyond 100 mm/s, from the limited data available, an amplification factor of 1.0 is appropriate.

The same stepped function can be conservatively applied to the frame in the out-of-plane direction, as shown in **Figure 33**. The out-of-plane response of the brick veneer is more difficult to generalise, as the amplification is dependent on the geometry, boundary conditions, type and condition of brick ties, and presence of a damp proof course.

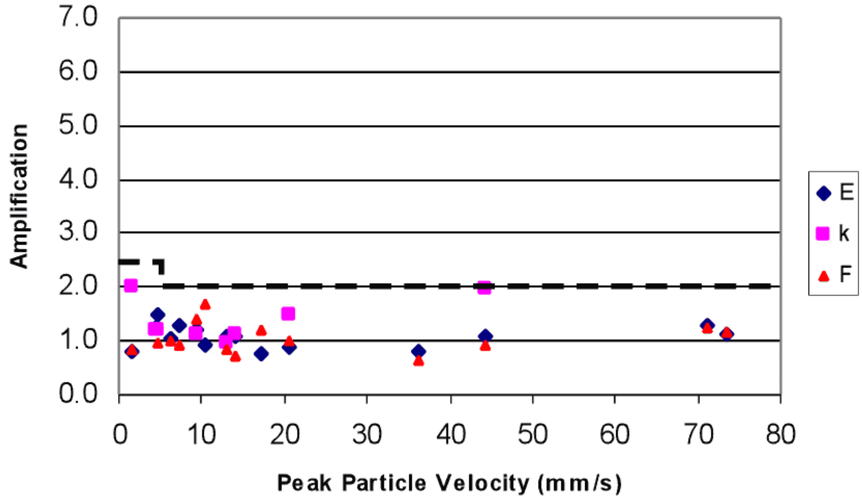
Generally, the highest amplifications are associated with isolated walls such as those bounded by large window/door openings. Such an isolated wall was monitored in the Rix's Creek house and the amplification response is plotted in **Figure 33**. This 'wall' was a narrow masonry panel between a window and a sliding door. This figure shows amplification up to 6 for very low ground PPV values, although damage could not be expected due to the inherent flexibility of the wall configuration. The strains associated with high amplification at low ground PPV levels are still relatively low.

It is expected that for regularly configured brick veneer walls, the step function described in Equation [3] would conservatively envelope the out-of-plane amplification. It should be noted that all these amplification factors have been found to be conservative for very high PPV (190 mm/s and 220 mm/s, refer to **Table 16**). However, these have not been shown in **Figures 32** and **33** for clarity and the fact that these records are extraordinarily high (blasting was only 50 metres from the house).

Figures 32a-b - In-plane velocity amplifications at ceiling level for the Rix's Creek house for different levels of ground vibrations

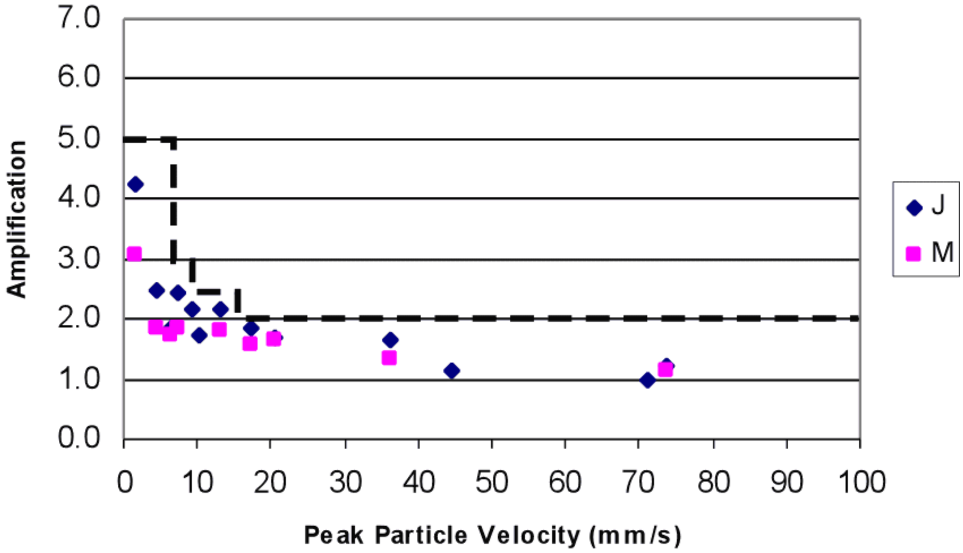


(a) In-plane framed walls

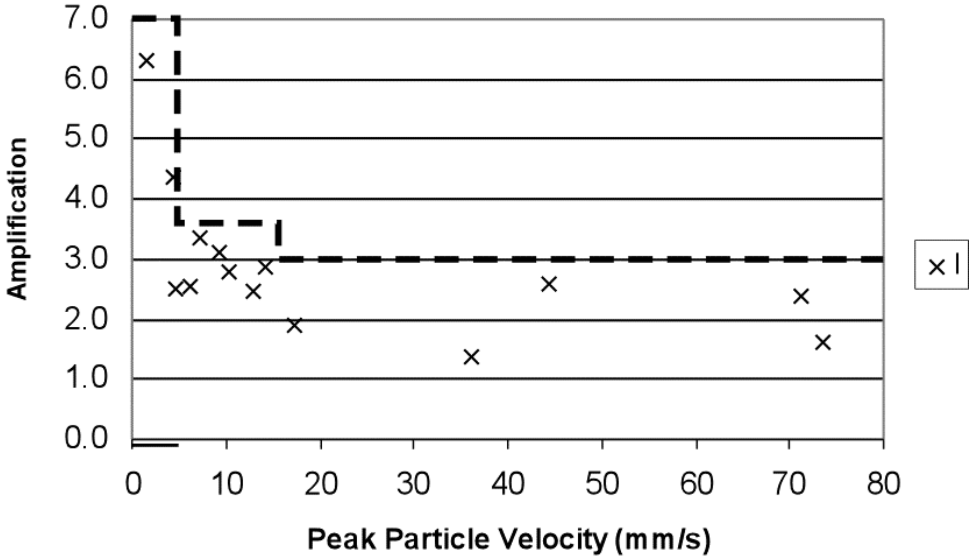


(b) In-plane brick veneer walls

Figures 33a-b - Out-of-plane velocity amplifications at ceiling level for the Rix's Creek



(a) Out-of-plane framed walls



(b) Out-of-plane isolated narrow brick veneer panel

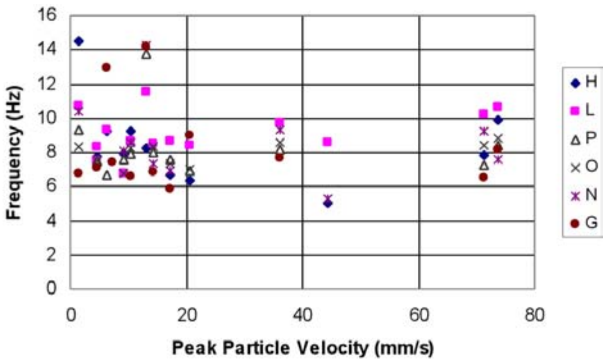
7.2.2 Dominant frequency

For the measuring locations shown in **Figure 18**, the acceleration records were integrated and double integrated to obtain the peak velocity (V_1) and peak displacement at the ceiling level (Δ_1). The dominant frequency (f) was calculated assuming a simple single degree of freedom response as follows:

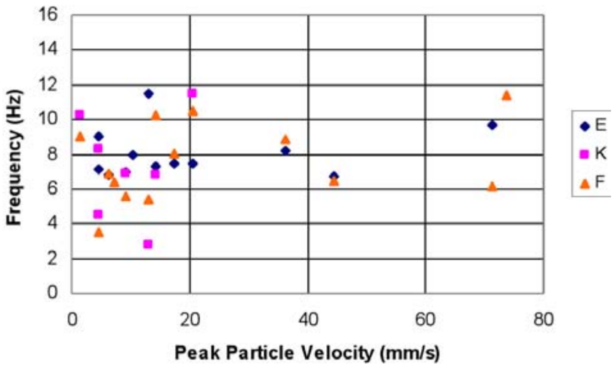
$$f = \frac{V_1}{2\,\pi\,\Delta_1} \qquad [4]$$

It is noted that this is a major simplification, however, the method enables a realistic estimate of Δ_1 and hence the drift to be made. The dominant frequency tends to vary with V_g and hence PPV and is in the range of 6 Hz to 10 Hz as shown in **Figure 34**. A lower bound frequency figure of 6 Hz is recommended so that conservative values of the displacement and drift can be estimated.

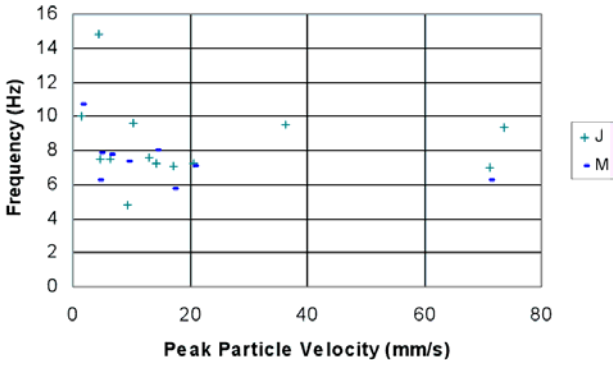
Figures 34a-d - Dominant frequencies calculated from the Rix's Creek house for the estimation of displacements at the ceiling level for different ground vibration levels



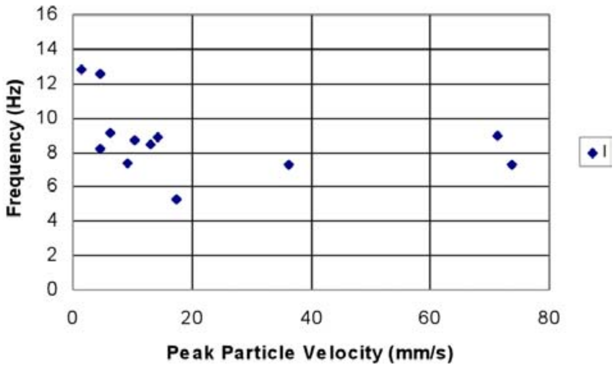
(a) In-plane framed walls



(b) In-plane brick veneer walls



(c) Out-of-plane framed walls



(d) Out-of-plane isolated brick veneer wall

7.3 Test House No. 2 - Wybong Road House

7.3.1 Amplification Effects

The ratios of the peak component velocity at ceiling level to ground level were calculated to estimate the likely vibration amplification effects with the height of the structure. The ratios for in-plane brick veneer and plaster and out-of-plane brick veneer amplifications are shown **in Figures 35a**, **35b, 35c** and **35d**.

The amplification levels obtained from Test House No. 2 are well within the limits developed from Test House No. 1. That is, for PPV less than 5 mm/s, the maximum amplification is 4 for a single storey house. For a double storey house the maximum amplification would be about 5, as shown in **Figure 35b**. Also, narrow slender brick veneer walls subjected to out-of-plane vibrations may experience higher amplification than 4, as shown in **Figures 35c** and **33b**.

However, these walls are unlikely to experience damage as they have a high degree of flexibility in the out-of-plane direction. Furthermore, the very high amplification (in excess of 5) seems to occur at very small PPV (less than 1 mm/s).

Figures 35a-d – Velocity amplifications at ceiling level for the Wybong Road House

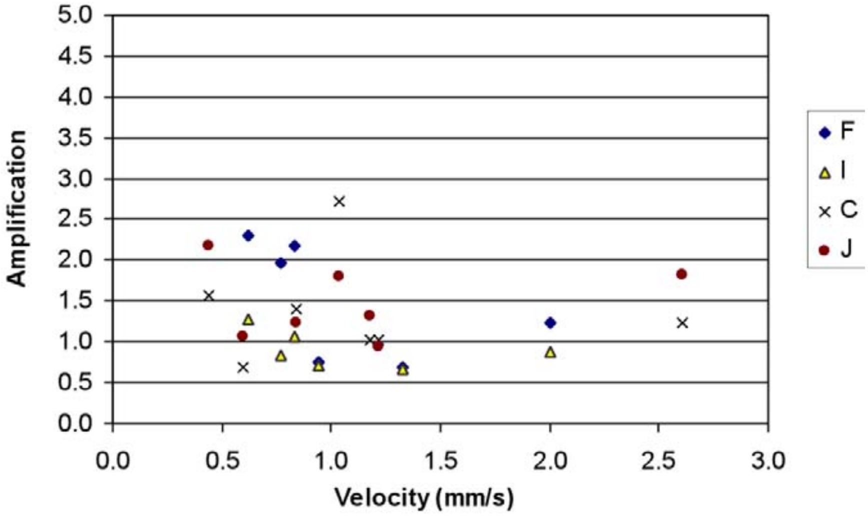


Figure 35a – In-plane brick veneer (1st floor)

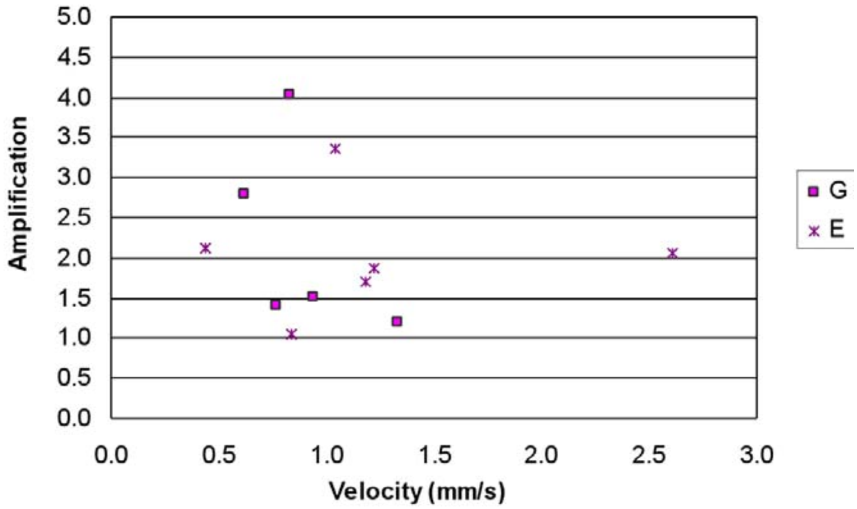


Figure 35b – In-plane brick veneer (2nd floor)

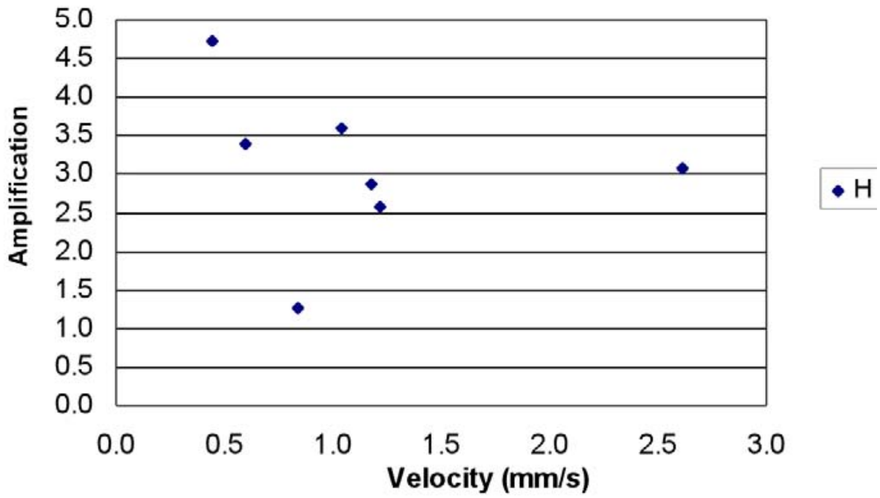


Figure 35c – Out-of-plane slender brick wall

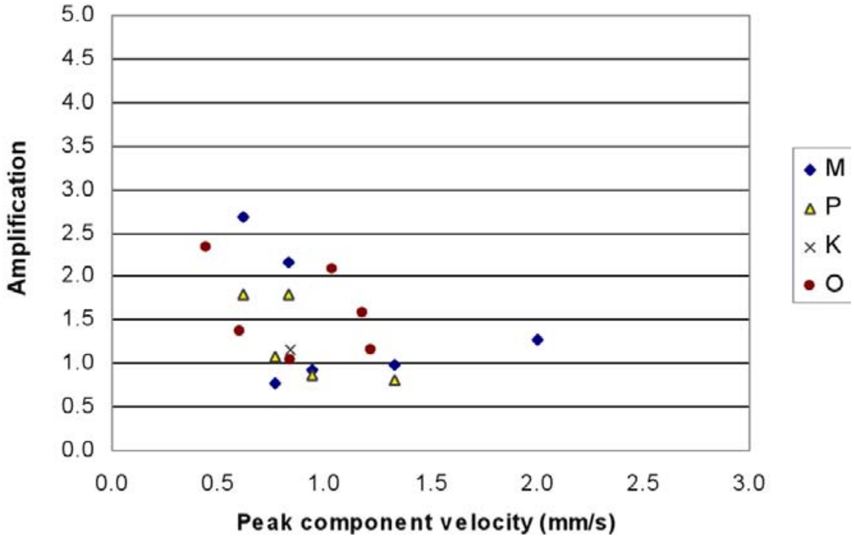


Figure 35d – In-plane frame

7.3.2 Dominant Frequency

Similar to Test House No. 1, the acceleration measurements were integrated and double integrated to obtain the velocity and displacement assuming a simple single degree of freedom system. The dominant natural frequency based on this simplified system was obtained and plotted against the velocity, as shown in **Figures 36a** to **36d**.

Figures 36a-d – Dominant frequencies calculated from Wybong Road house for different ground vibration levels

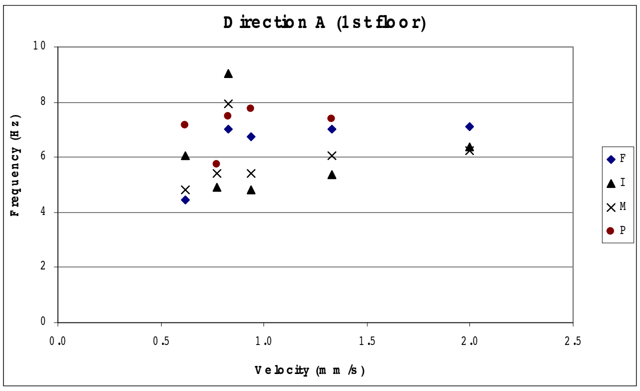


Figure 36a – In-plane

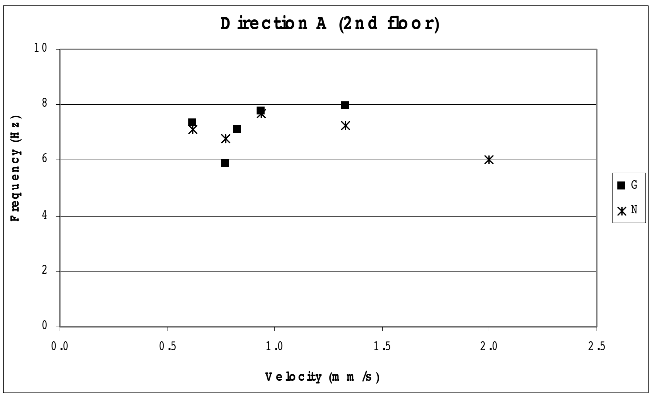


Figure 36b – In-plane

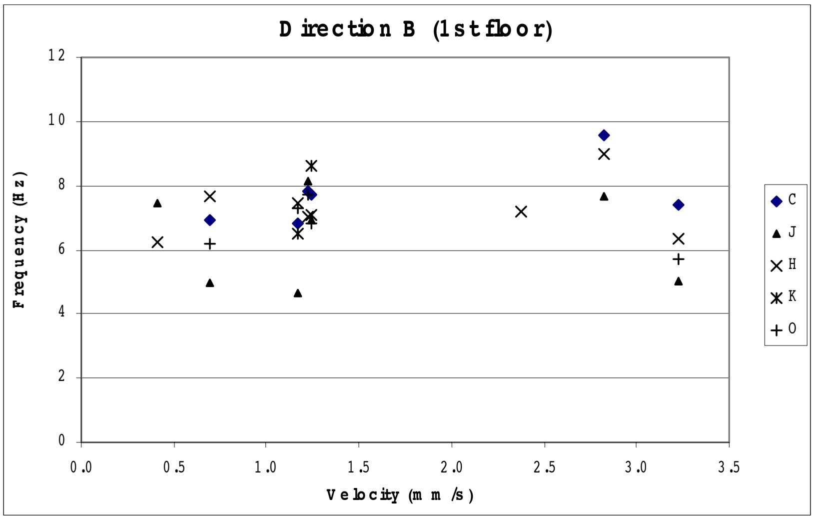


Figure 36c – Out-of-plane

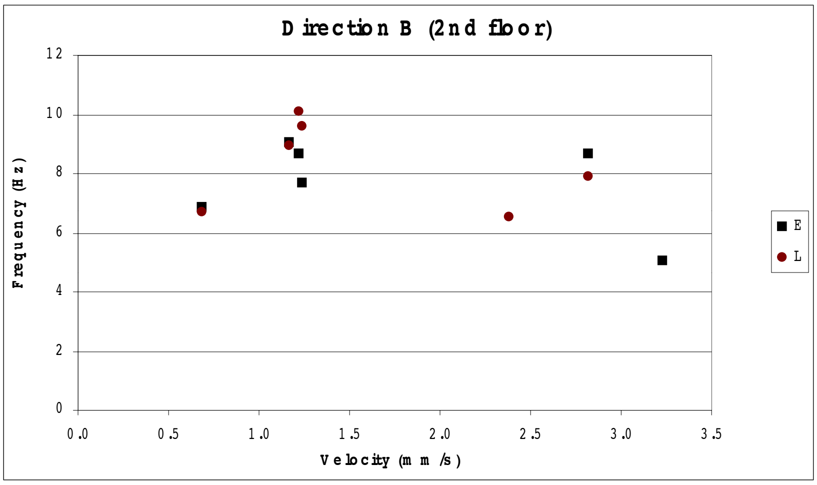


Figure 36d – Out-of-plane

In the lower vibration range measured at this house, the dominant frequencies for both the in-plane plaster and brick veneer range from 4.5 Hz to 10 Hz, with most of the data points in the range of 6 Hz to 10 Hz.

At lower vibration levels, the conservative method developed for determining displacement and drift for Test House No. 1 would still apply and give conservative estimates. The amplification factor of 4 up to 5 mm/s (Test House No. 1) may be high in the range of 1.5 mm/s to 5.0 mm/s, as determined in Test House No. 2, but is compensated for to some extent by the lower frequencies found at low vibration levels.

7.4 Quantification of Damage

The conservative values for the amplification (λ = 2 and 4) and frequency (f = 6 Hz) developed in Section 7.2 have been used to estimate the upper range ceiling displacements (Δ_1) and principal tensile strain (ε) for a single storey house (ceiling height of 2.4 metres), subject to different levels of ground vibration expressed in terms of PPV, as shown in **Table 15**. The lower range of tensile strains based on an amplification of 1.0 and a frequency of 10 Hz are also shown.

It could be argued that an even smaller strain estimate would result if the maximum frequency of 14 Hz established in **Figure 34a** was used. However, amplifications of 1.0 are associated with higher PPVs at the natural frequencies of the building. The adaptation of 10 Hz is, therefore, considered to be a reasonable maximum.

Table 15 – PPV, drift and strain determinations

PPV Ground (mm/s)	Amplification (λ)	Frequency (Hz)	Δ_1 (mm) lower upper	Drift (γ) lower upper	Strain (με) lower upper	Plasterboard Failure/Strain (%) lower upper
1	1 - 4	6 – 10	0.016 - 0.1	1/150000 - 1/24000	3.3 - 22	0.3 – 2.2
5	1 - 4	6 – 10	0.08 - 0.5	1/30000 - 1/4500	17 - 111	1.7 – 11.1
10	1 - 2	6 – 10	0.16 - 0.5	1/15800 - 1/4500	33 - 111	3.3 – 11.1
20	1 - 2	6 – 10	0.32 - 1.1	1/7500 - 1/2250	66 - 221	6.6 – 22.1
25	1 - 2	6 – 10	0.40 - 1.3	1/6000 - 1/1800	83 - 276	8.3 – 27.6
50	1 - 2	6 – 10	0.80 - 2.7	1/3000 - 1/900	165 - 553	16.5 – 55.3
75	1 - 2	6 – 10	1.19 - 4.0	1/2016 - 1/600	245 - 829	24.5. – 83.9
100	1 - 2	6 - 10	1.59 - 5.3	1/1500 - 1/450	330 - 1105	33.0 – 110.0

Most codes of practice around the world recommend drift ratio in the order of 1/300 to 1/500 at the serviceability limit state to prevent damage from wind and earthquake loading. These drift ratios would conservatively correspond to blast vibrations in the order of 100 mm/s for the upper range of induced strain.

The principal tensile failure strains associated with solid plaster are in the order of 200 με to 300 με (Dowding, 1985) compared with 1000 με for plasterboard (Stagg et al, 1984 and Konig, 1989). These principal strains correspond to conservative ground vibration in the order of 25 mm/s and 100 mm/s for solid plaster and plasterboard, respectively. For masonry construction, such correlations are more difficult to establish due to the anisotropic properties of this composite material (bricks and mortar bed joints). The tensile strength of masonry is always quoted in terms of the tensile stress needed to rupture the bond between the bricks and the mortar (the associated tensile strain with rupture is typically in the order of 100 με to 300 με). In contrast, a blast loading, which induces racking displacements in a masonry wall, would result in shear stress and strains (and not principle tensile stresses) at the bricks and mortar interface. The shear strength at this interface is typically stronger than the corresponding tensile strength and a typical range of 250 με to 1000 με is the likely order of shear strength (Stagg et al, 1984). The shear strength of masonry is strongly influenced by the interpretation of when failure occurs because there is no yield point equivalent to ductile materials. If the presence of cracks visible to the naked eye is used as the criteria, the range is 500 με to 1000 με.

The strain levels presented in **Table 15** are all dynamic strains and must be added to any residual or existing strains in the structure. The residual strains could arise from a number of sources, including:

- Foundation movements associated with moisture changes in the soil.
- Thermal movements associated with temperature changes in the material causing shrinkage or expansion.
- Humidity changes resulting in shrinkage and swelling.

- Building age and material deterioration.
- Substandard building construction.
- Human actions, such as slamming doors and out of balance washing machines.

In order to establish an acceptable level of dynamic strain, an understating of the level of residual strains in the structure is required. For example, if it is estimated that existing strains are in the order of 90% of the material rupture strain, then the dynamic strain would need to be limited to 10% of the rupture strain to avoid the onset of cracking.

This would translate to a conservative limiting PPV of 10 mm/s to 25 mm/s for plasterboard assuming that the residual strains are in the order of 900 $\mu\varepsilon$. Similarly, the limiting PPV for plasterboard could be in the order of 50 mm/s to 100 mm/s if the residual strains were estimated to be 50% of rupture.

In the Rix's Creek house, no new damage from blasting was observed for PPV less than 70 mm/s. This suggests that the residual strain in this house were relatively small and in the order of 100 $\mu\varepsilon$ or 10% of the plasterboard rupture strain.

It has been recognised by other researchers that while it is theoretically possible for a house to be subject to almost rupture strain levels from other causes without cracking and the slightest ground vibration will be sufficient to induce cracks to appear, this situation is not found in practice. This investigation reinforces the hypothesis that, whereas houses may theoretically show cracking of plaster after the small dynamic strains from vibration are added to high residual environmental strains, in practice, houses are either cracked or not cracked from environmental strain. At worst, the additional strains added by vibration at the environmental limit of 5mm/s to 10 mm/s can only amount to about 2% to 10% of the strains necessary to cause plaster and unarticulated masonry to crack.

It is recognised that for fatigue to be an issue, the dynamic strain needs to be greater than some limiting threshold value and the material subjected to enough load cycles for fatigue cracks to occur. The dynamic strain associated with the normal PPV limits of blasting are generally small and less than the threshold value to cause fatigue cracking (British Standard BS7385: Part 2: 1993).

8. DIRECT STRAIN MEASUREMENTS

Vertical and horizontal strain measurements were taken on the bricks of the test house walls for comparison with strains predicted by the method developed in Section 6.

8.1 Direct Strain Analysis – Test House No. 1 – Rix's Creek

Initially, the horizontal strain gauge was placed across a crack so that it was measuring the dynamic crack movement instead of the strain. On 4th May 2000 the gauges were moved to measure strain in the outer skin of the brick veneer wall. The strain gauge measurements are listed in **Table 16**, together with ground PPV, the dominant frequency and the PPV measured at eaves level.

The PPV on the ground was compared to the peak longitudinal component velocity at eave level, representing the in-plane peak motion of the wall (refer **Figure 37**). The 1.0, 2.0, 3.0 and 4.0 amplification lines are drawn, together with the in-plane stepped amplification function from **Figure 33a** in the accelerometer response analysis section. The proposed amplification function is in agreement with the geophone in-plane response.

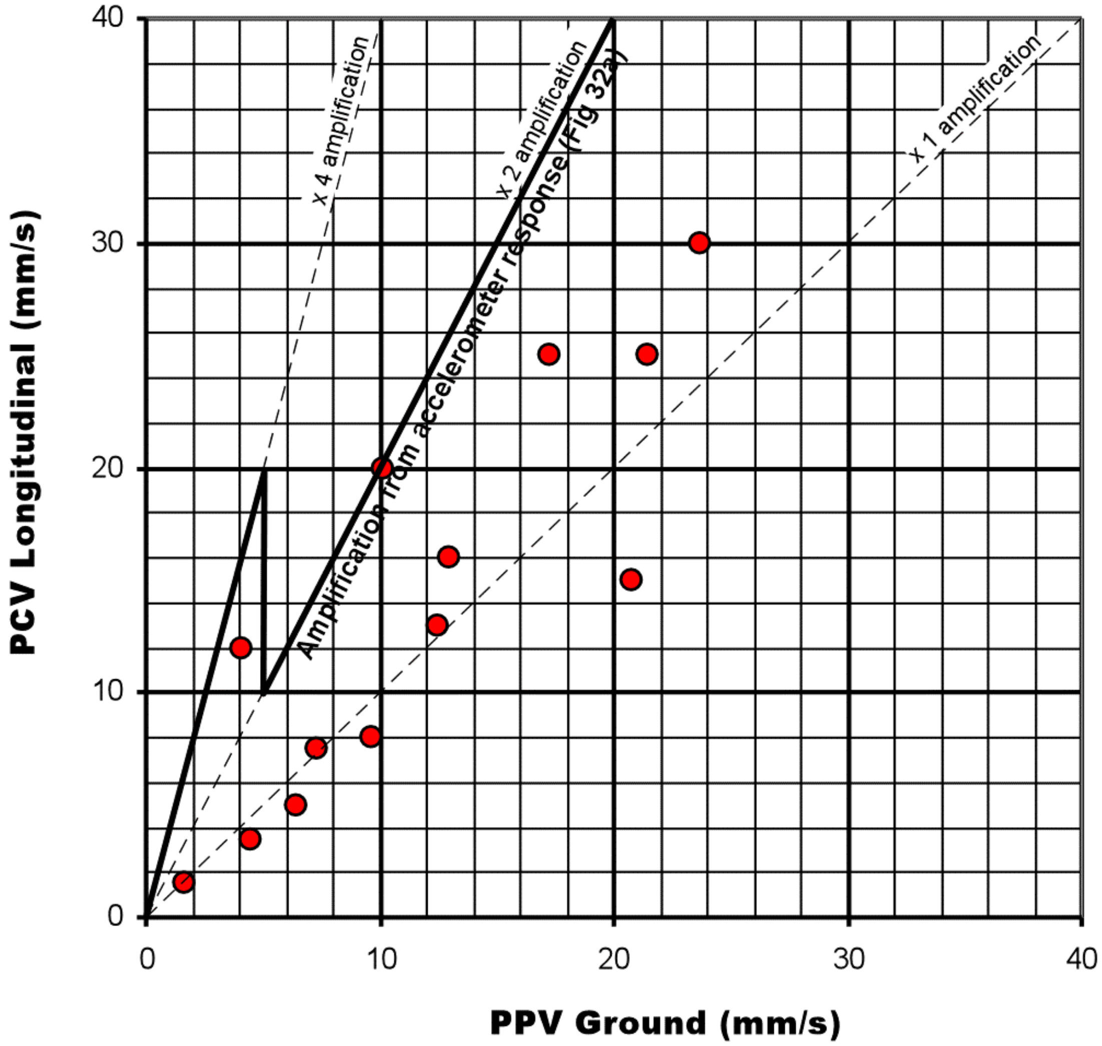


Figure 37 – PPV at eave level compared to PPV on ground

The direct horizontal and vertical direct strain measurements were vectorised and compared to the ground PPV (see **Figure 38**). Except for one outlier, the data envelope forms a narrow band.

As a check on the predictive model developed in Section 6, the measured strains were compared to strain calculated by the following methods:

When the PPV at eave level and on the ground are compared, these amplification factors for PPV are slightly different than those determined from the peak component accelerometer analysis. This coarser analysis suggests that the mean amplification factor up to 4.0 is applicable at 5 mm/s but reduces to 3.0 to about 25 mm/s and further reducing to 1.0 at 100 mm/s. This can be explained by the motion of the eave level geophone, which includes a partial out-of-plane motion component as well as the in-plane motion. The accelerometers, which were placed on the corners, were not subject to the same out-of-plane motion.

The ground PPV was compared to measured strain (refer **Figure 38**) and, except for one outlier, the data envelope formed a narrow band. As a check on the predictive model developed in Section 6, the measured strains were compared to strain calculated by two methods.

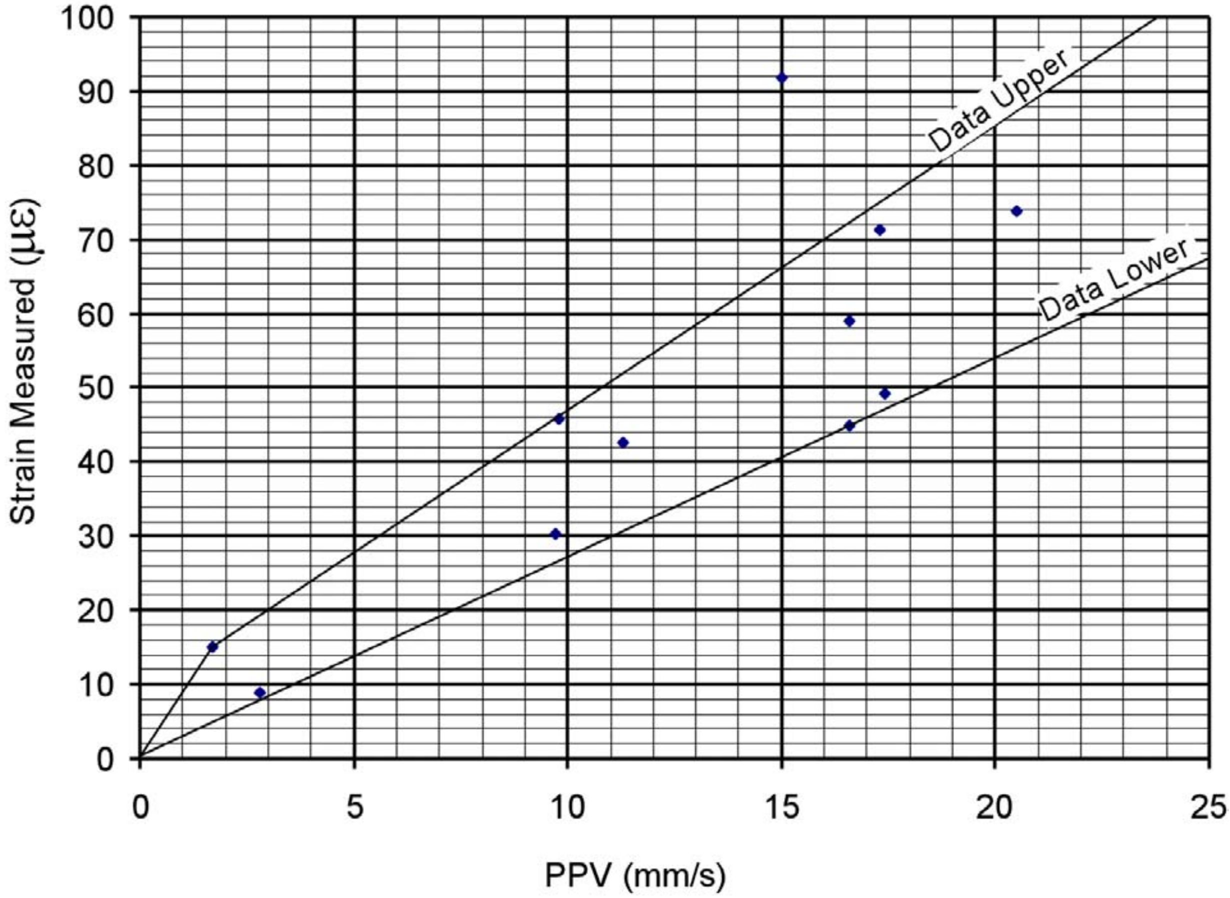


Figure 38 – PPV ms measured strain – Test House No. 1

$$\text{Method 1:} \quad \in \; = \frac{\text{PPV at eaves}}{2\,\pi\,\text{f}} \;\text{x}\; \frac{0.5}{2400}$$

The frequency used was the dominant frequency measured at the eaves.

$$\text{Method 2 (developed in Section 6.2):} \quad \in_{max} \; = \frac{\text{ground PPV x amplification factor}}{2\;\pi\;\text{x}\;6} \;\text{x}\; \frac{0.5}{2400}$$

$$\in_{min} \; = \frac{\text{ground PPV}}{2\;\pi\;\text{x}\;14} \;\text{x}\; \frac{0.5}{2400}$$

For ε_{max}, the amplification factor was based on the stepped function from Section 6, ie. 4.0 for up to 5 mm/s and 2.0 for >5 mm/s and 1.0 for >100 m/s. The resonance frequency was assumed to be 6 Hz. For ε_{min} the amplification factor is 1 and the frequency is 14 Hz.

The wall height was considered to be 2400 mm since the wall was articulated by a crack at the damp-proof course. The expression Sinφ Cosφ was conservatively considered to be 0.5 (corresponding to a maximum at 45^{o}) and would reduce as the wall length increases.

The calculated and measured strains are shown in **Table 16** and plotted in **Figure 39**. The measured strains are less than Method 1 estimates and much less than Method 2 ε_{max} estimates. The measurements are slightly above Method 2 ε_{min} estimates. The Method 1 calculations, based on peak eave PPV levels, include out-of-plane and vertical motion components and would, therefore, be expected to be conservative.

Table 16 – Strain Measurements Test House No. 1 – Rix's Creek

Date	Ground PPV (mm/s)	Freq. (Hz)	ε_H (με)	ε_V (με)	Vect. ε (με)	PPV Eaves (mm/s)	Freq. (Hz)	Amp.	(1) ε calc. (με)
23/12/99	7.3	5.8	37.8*	4.7	-	7.7	**	1.05	23.4
21/01/00	13.2	est.	112.0*	13.8	-	24.5	**	1.86	79.6
08/02/00	12.5	11.0	130.8*	24.8	-	32.3	**	2.6	105.0
15/02/00	21.5	12.0	175.3*	56.9	-	60.0	**	2.8	195.0
18/02/00	20.8	15.0	282.4*	47.8	-	70.3	**	3.4	228.5
27/03/00	4.45	10.0	23.7*	4.5	-	8.1	9.3	1.8	28.9
11/04/00	4.5	10.0	46.5*	10.4	-	12.2	10.4	2.7	38.9
13/04/00	6.3	12.0	34.1*	15.9	-	18.7	**	3.0	60.8
13/04/00	7.3	12.0	89.6*	7.3	-	18.8	11.5	2.6	54.2
04/05/00	17.4	10.1	35.9	33.8	49.3	37.5	8.1	2.2	153.5
09/05/00	11.3	13.8	37.5	20.4	42.7	25.7	13.1	2.27	65.0
12/05/00	1.7	9.7	13.0	7.5	15.0	6.2	12.9	3.6	15.8
12/05/00	15.0	7.0	87.3	28.7	91.8	40.8	10.9	2.7	124.1
22/05/00	20.5	10.5	55.5	48.9	73.9	37.4	10.5	1.82	118.1
28/07/00	4.9	13.0	18.2	-	-	11.9	**	2.43	38.6
07/08/00	16.6	11.3	43.7	10.2	44.9	30.3	9.4	1.83	106.9
07/08/00	2.8	10.3	8.2	3.3	8.8	5.7	9.5	2.0	19.5
08/08/00	9.7	5.4	29.3	7.8	30.3	23.2	4.5	2.4	171.0
11/08/00	71.2	17.0	167.1	46.0	173.7	96.3	**	1.35	31.3
17/08/00	17.3	11.5	64.3	30.7	71.2	50.5	9.3	2.9	180.0
02/11/00	9.8	11.4	44.0	12.6	45.7	17.3	11.1	1.76	51.7
06/11/00	16.6	20.8	55.8	19.4	59.0	27.5	9.3	1.65	98.0
07/12/00	148.4	14.0	252.3	240.5	348.5	220.2	11.2	1.48	651.0
20/12/00	222.0	24.0	171.5	102.4	194.7	191.5	12.5	0.86	508.0

* *measured across an existing crack*   ** *Av. 10.2*

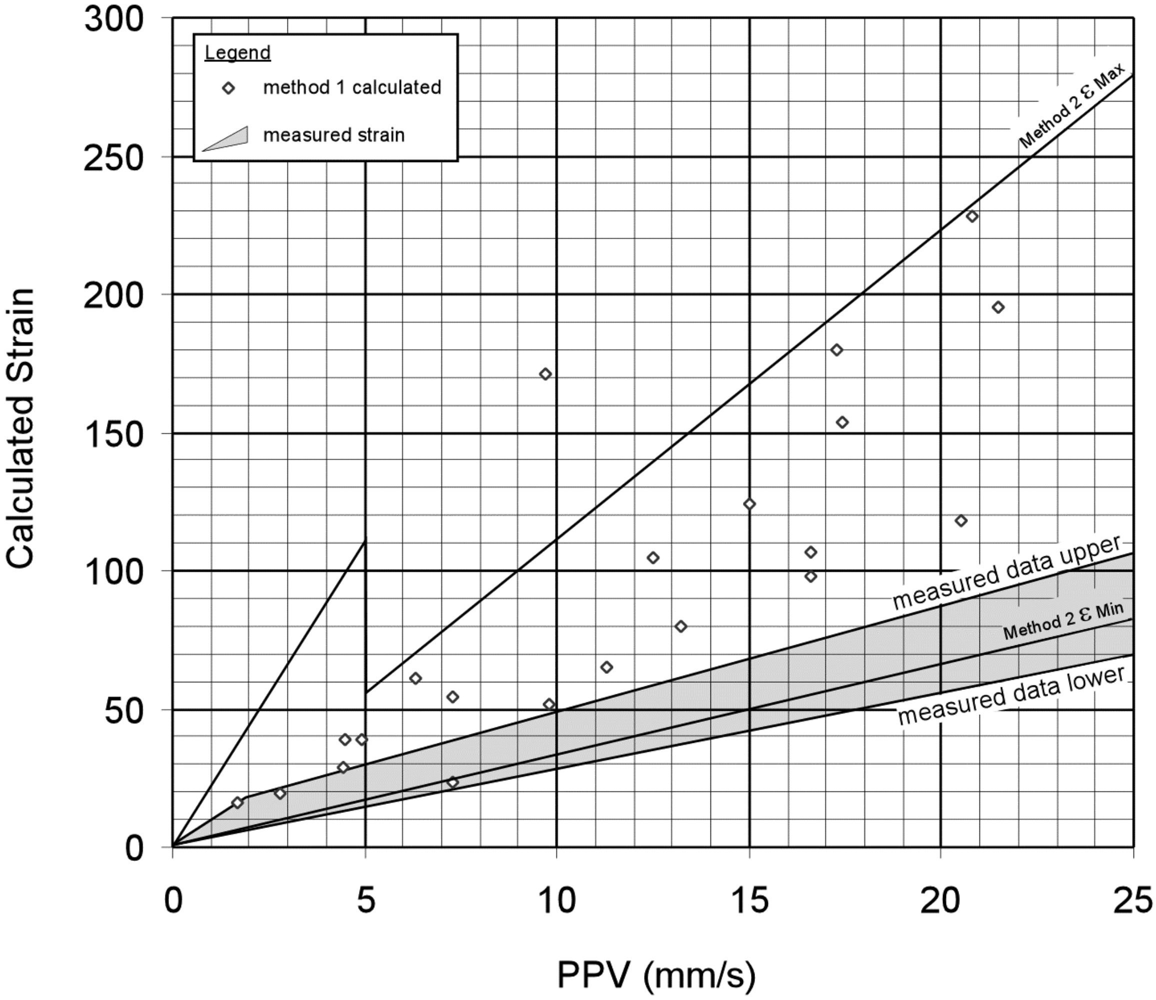


Figure 39 – PPV versus measured and calculated strain – Test House No. 1

8.2 Direct Strain Analysis – Test House No. 2 – Wybong Road

Direct strain measurements were taken of a number of blasts at Test House No. 2 - Wybong Road. The recorded data is listed in **Table 17**, together with the strains calculated by the two methods outlined previously.

Table 17 – Strain Measurements Test House No. 2 – Wybong Road

Date	Ground PPV (mm/s)	Freq. (Hz)	ε_H (με)		ε_V (με)	Vect. ε (με)	Eave PPV (mm/s)	Freq. (Hz)	Amp.	(1) ε calc. (με)
27/03/01	2.1	13.2	20.49		7.4	21.9	12.7	6.6	6.0	31.9
02/04/01	1.21	15.6	nr		nr	-	6.5		5.37	-
09/04/01	1.06	20.0	10.3	8.9 Hz	-	-	-	-	-	-
20/04/01	0.65	14.3	13.8	8.9 Hz	-	-	-	8.9	-	-
26/04/01	1.36	12.2	21.6		4.9	22.1	7.7	8.2	5.66	31.1
26/04/01	est 1.2	-	4.9		2.5	5.5	7.0	8.2	est 5.7	28.3
27.04/01	est 0.6	-	nr		3.2	-	3.1	est 6.9	-	14.9
30/04/01	1.31	11.9	24.6		3.7	24.8	7.7	6.9	5.87	37.0
01/05/01	2.37	-	47.5	10.1 Hz	6.6	48.0	7.3	6.4	3.08	37.8
09/05/01	0.62	8.5	8.04		12.9	15.2	est 3.1	8.4	est 5.0	12.2
05/06/01	est 0.9	-	8.0		1.7	8.2	4.7	6.8	est 5.0	22.9
29/06/01	2.96	6.0	41.1		7.6	41.8	11.4	6.7	3.85	56.4

The data represents the response of this house to peak ground vibration levels in the range 0.65 mm/s to 3.02 mm/s. The relationship between measured strain and peak particle velocity is shown in **Figure 40**, together with the strain calculated by Methods 1 and 2. For this two storey house, the measured strains are slightly less than the Method 1 estimates and are closer to the Method 2 ε_{max} estimates than Test House No. 1. At low vibration levels near 1 mm/s the measured strain is closer to Method 2 ε_{min} estimates.

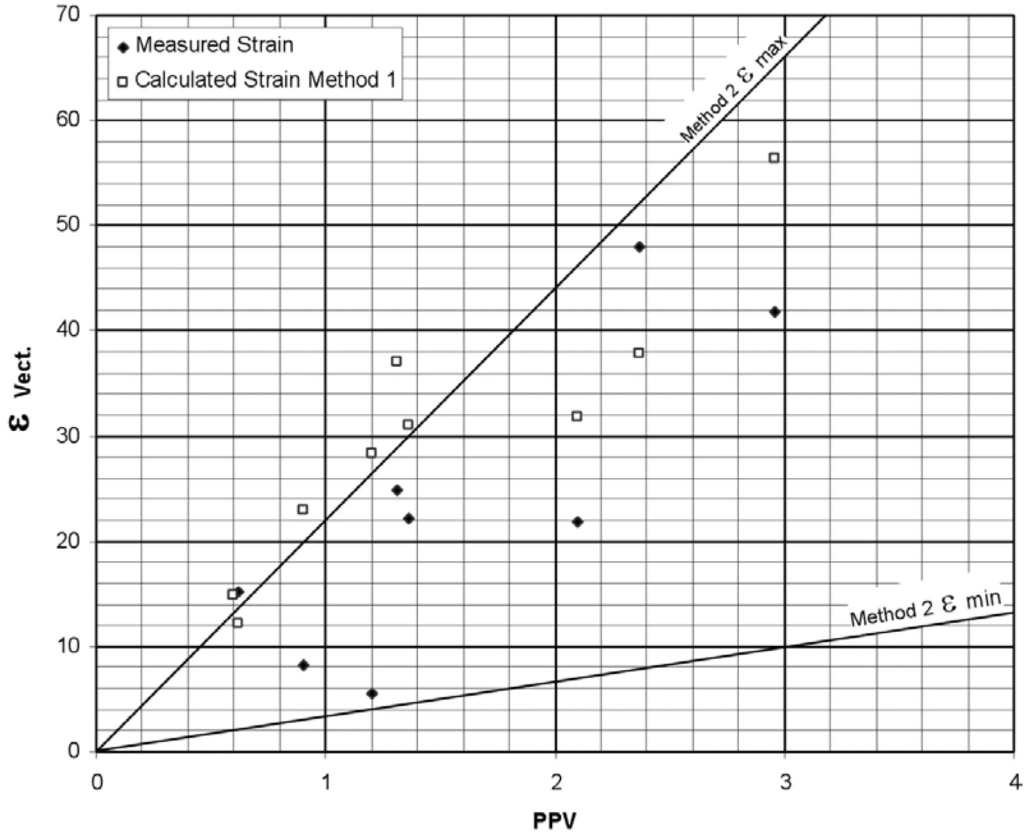


Figure 40 – PPV versus measured and calculated strain – Test House No. 2

8.3 Direct Strain Analysis – Test House No. 3 – Racecourse Road

A number of blasts were monitored at Test House No. 3 - Racecourse Road but, because of the low vibration levels experienced, only limited strain data was obtained. The measured strains were only slightly above background noise for the instrumentation. The recorded data is listed in **Table 18**, together with strains calculated by the two methods outlined previously – estimations made are listed and plotted in **Figure 41**.

Table 18 - Strain Measurements Test House No. 3 – Racecourse Road

Date	Ground PPV (mm/s)	Freq. (Hz)	ε_H (με)	ε_V (με)	Vect. ε (με)	PPV Eaves (mm/s)	Freq. (Hz)	Amp.	(1) ε calc. (με)
18/05/01	0.29	19.2	-	<2.0	-	-	*	est 1.2	1.03
22/05/01	1.08	est 12.5	1.5	1.8	2.3	-	14.0	est 1.2	3.06
31/08/01	0.96	9.4	n/a		-	-	*	est 1.2	3.44
05/10/01	0.9	7.0	n/a		-	-	*	est 1.2	3.22
22/10/01	0.49	7.7	<2.1		-	0.6	8.3	1.2	2.39
08/11/01	0.8	11.4	<4.0		-	0.9	14.7	1.1	2.03
06/12/01	0.29	14.7	<2.0		-	0.35	7.4	1.2	1.56
06/12/01	1.11	13.4	7.0	1.8	7.2	1.3	*	1.2	3.9

** Aug 11.1*

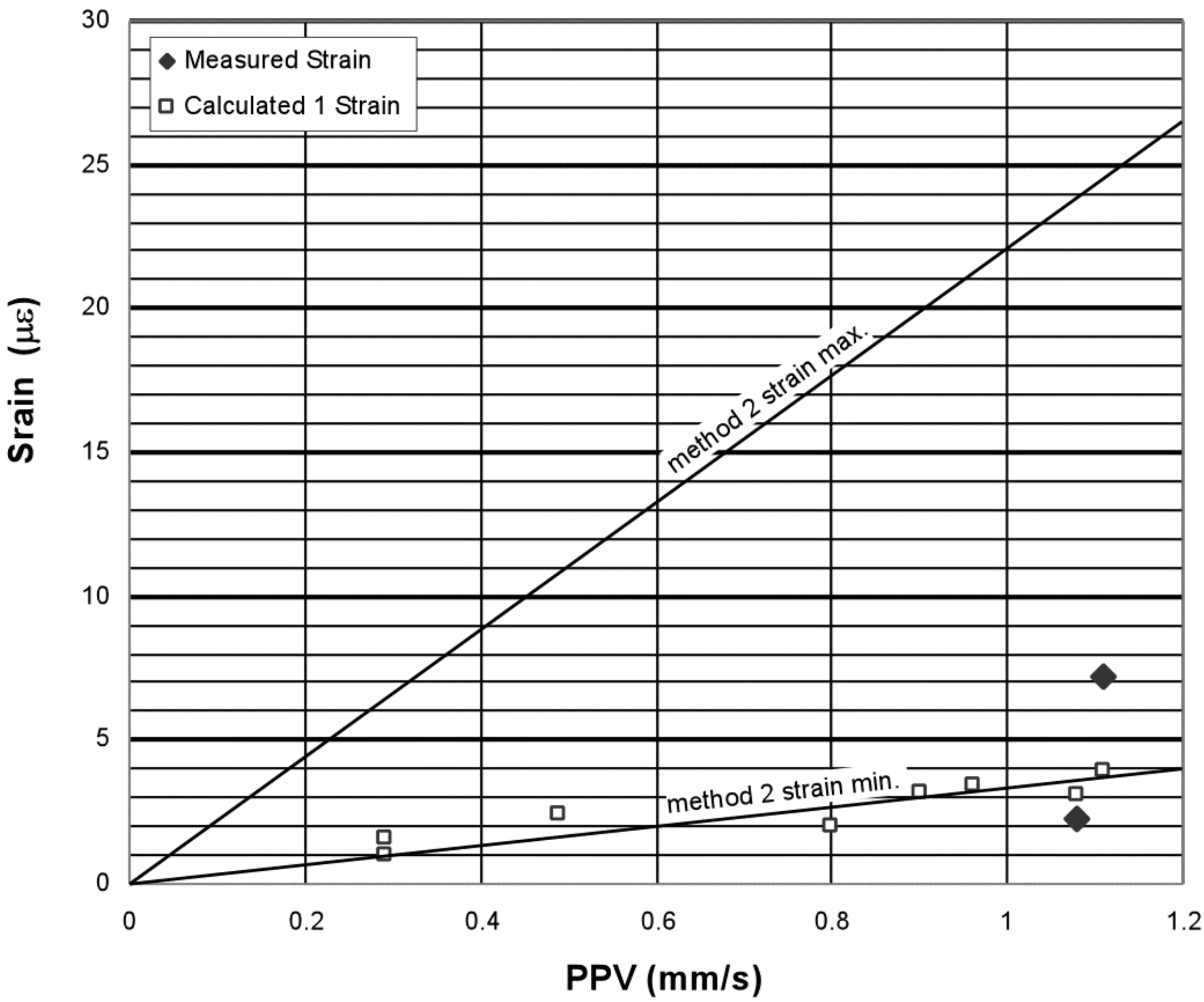


Figure 41 – PPV versus measured and calculated strain – Test House No. 3

The limited measured strain data is a similar order of magnitude to that determined by Method 1 calculation, which uses the eave level PPV measurement and gives a reasonable correlation to the recorded data. The Method 2 ε_{max} calculation, which assumes an amplification factor of 4.0 at low levels, gives a more conservative strain estimation. Again, for this single storey house, the Method 2 ε_{min} estimate is closer to the limited measured strain data.

9. EFFECT OF AIRBLAST ON STRUCTURES

Limited data on the effect of airblast on structures was collected during this research project. Test House No. 1 (Rix's Creek) was too close to the blasting for the air and ground vibration to cause separate responses in the house for many blasts. At Test House No. 3 (Racecourse Road), the airblast levels were too low for the instrumentation to register a measurable response. However, 5 blasts were identified at Rix's Creek where the airblast resulted in a measurable separate response. The comparison between measured responses due to ground vibration are listed in **Table 19**. The equivalent ground vibration resulting in the airblast response at the eaves was determined from **Figure 37** and is listed in **Table 19**.

Table 19 – Measured airblast response compared to ground vibration response

Date	PAV (dBL)	PPV Ground (mm/s)	PPV Ground Equivalent (mm/s)	PPV Eaves (mm/s)	
22/02/00	-	23.7	-	35.8	Ground response
	131.0	-	2.6	9.0	Air response
12/05/00	-	1.8	-	3.8	Ground response
	124.0	-	1.5	6.0	Air response
18/07/00	-	6.86	-	14.5	Ground response
	133.0	-	1.8	7.0	Air response
08/08/00	-	9.70	-	23.2	Ground response
	130.0	-	3.0	10.0	Air response
06/11/00	-	13.0	-	27.5	Ground response
	126.0	-	1.8	7.0	Air response

The response of structures to airblast was studied by the USBM and the results presented in RI 8485 (Siskind et al).

The equivalent racking responses measured in the USBM study are re-plotted in metric units in **Figure 42** and the data mean and envelope maximums are shown. The equivalent mid wall responses are also replotted in **Figure 43**, together with the five racking response measurements from **Table 19** of this study.

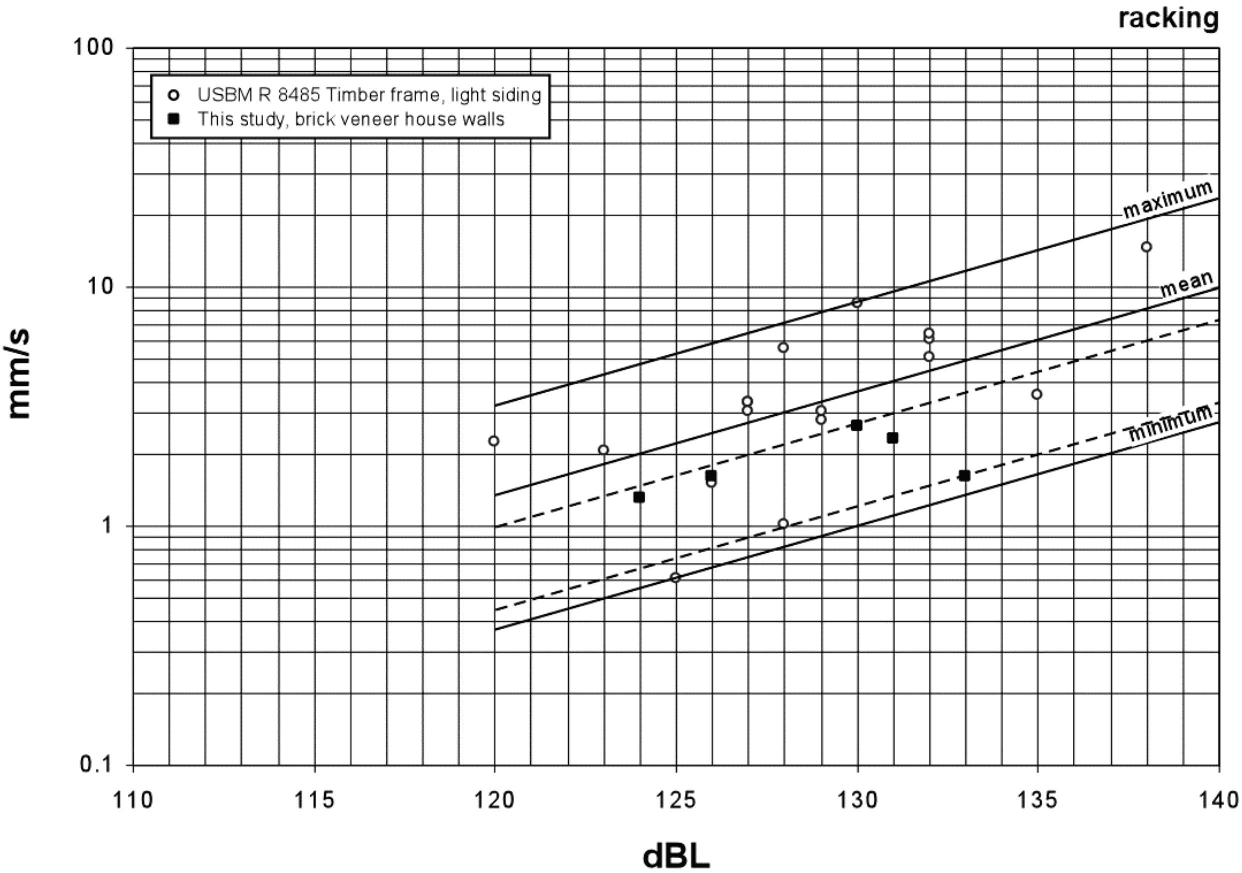


Figure 42 – Racking in-plane mid wall motion: airblast versus ground vibration (Siskind et al, 1984)

It can be seen that the relationship between ground vibration and the equivalent airblast structural response is complex. The USBM study was from mainly one and two storey timber framed houses with timber, aluminium, asbestos or asphalt cladding; lined with gypsum wallboard or lathe and plaster with partial or full basements. The limited data set from this study lies within but below the mean of the USBM study, indicating that brick veneer structures are less flexible and have less response to airblast than lightly clad timber framed structures.

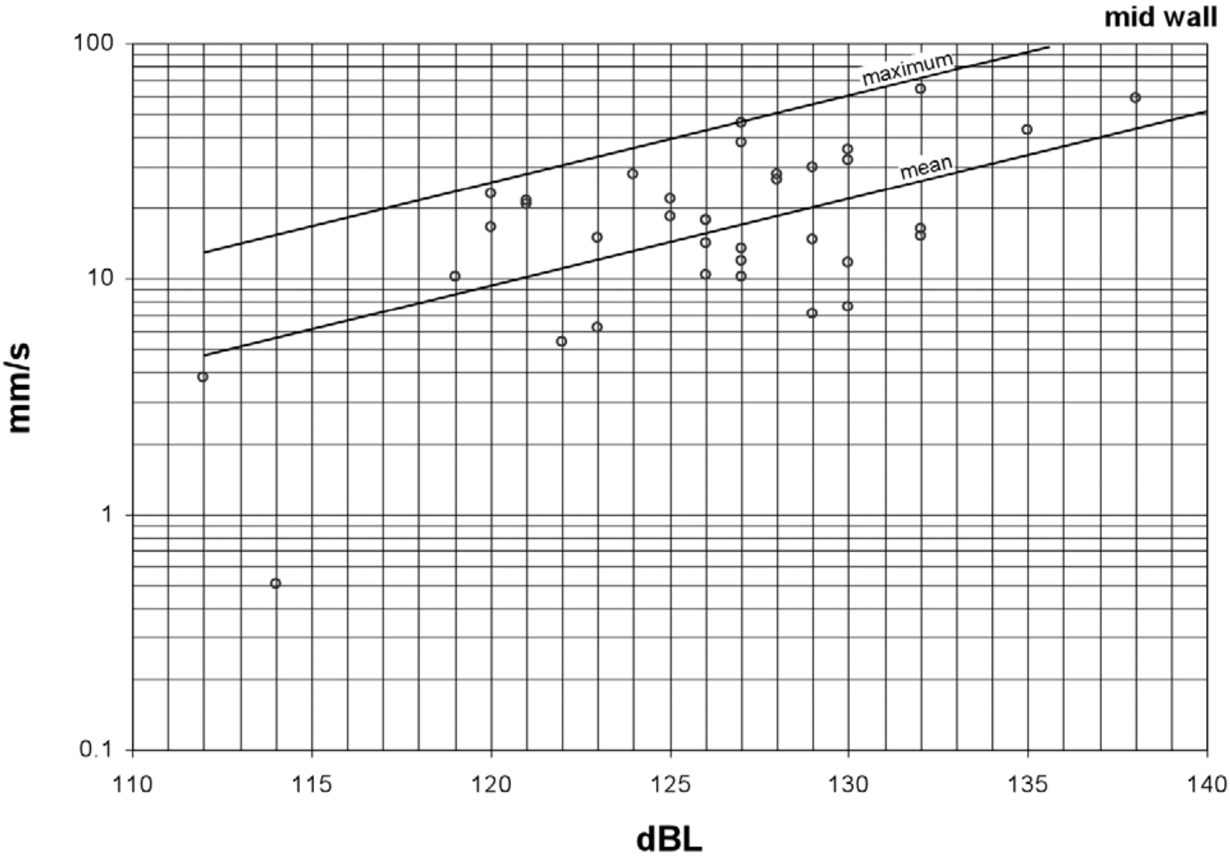


Figure 43 – Out-of-plane mid wall motion: airblast versus ground vibration (Siskind et al, 1984)

The most conservative airblast equivalent (worst case) for ground vibration levels are listed in **Table 20**.

Table 20 – Ground vibration and equivalent racking airblast response

Ground Vibration (mm/s)	Equivalent Airblast Response ACARP Study Brick Veneer Walls (dBL)	Equivalent Airblast Response USBM Study Timber Framed Light Clad (dBL)
1	119	110
2	126	115
5	135	124
10	142	131

The 133 dBL recommended airblast damage limit in AS2187.2-1993 has the approximate equivalent structural response to 10 mm/s in racking response terms from quarry blasts for lightly clad timber framed houses subject to vibration from quarry blasting.

Note: The equivalent airblast response figures from this ACARP study are for brick veneer walls and may not be applicable to other parts of the structure of brick beneer houses. Pending further investigation, it is recommended that the equivalent airblast response figures from the USBM study be used for parts of the structure other than brick veneer walls.

10. A RATIONAL APPROACH TO THE ASSESSMENT OF BLAST DAMAGE

One of the important findings of this investigation is that blast damage can be assessed on a rational basis. The dynamic strains induced in a structure by ground vibration can be estimated with reasonable accuracy by consideration of the racking response of the structure. These can then be compared to the static failure or cracking stains of the construction materials, which is a valid comparison if the dynamic strains are comparatively low so that fatigue is not a consideration.

The induced strains can also be compared to the strains induced by natural forces and events. The natural environmental strains may themselves exceed the cracking strains of the construction materials. If cracking has occurred, the possible contribution that vibration strains may have contributed to observed cracks may be determined and a cause weighting applied, if required.

10.1 Estimation of Dynamic Strains Induced in Buildings

The in-plane deflection places the maximum strain on the walls of a house. The maximum strain can be estimated from:

$$\text{Maximum strain } \in \; = \frac{\Delta}{L} \text{ Sin}\varphi \text{ Cos}\varphi \qquad [7]$$

$$\Delta = \frac{\text{the peak component ground velocity x amplification}}{2 \pi f} \qquad [8]$$

Sinφ Cosφ has a maximum value of 0.5 at 45^o.

The peak component particle ranges from (0.57 → 1) x PPV.

If the peak component particle velocity is not known, the peak ground particle velocity may be used for a conservative estimation. A wall height of 2400 mm is appropriate because most brick veneer constructions are articulated by a damp-proof course and the strain calculated for higher walls is proportionately less because it is inversely proportioned to height.

A most conservative estimate then is $\varepsilon = \frac{PPV(g)}{2 \pi f} \text{ x amplification x } \frac{0.5}{2400}$

The conservative amplification factor is 4.0 for ≤5 mm/s and 2.0 for 5mm/s to 100 mm/s. The least conservative amplification factor is 1.0 for all vibration levels.

This is a major simplification but, as this investigation has demonstrated, results in a conservative estimate of strains induced in a house from vibration.

A very conservative estimation of the peak strains for a wall 2400 mm high are given in **Table 21**, using the range of amplifications from this research and a peak component velocity equal to the peak ground (vector) particle velocity combined with a 6 Hz and 10 Hz frequency, which represents the range of frequencies found.

Table 21 – Range of calculated peak strains for a wall 2400 mm high

PPV (ground) (mm/s)	Amplification	Maximum Strain $\mu\varepsilon$ @ 6 Hz	Amplification	Minimum Strain $\mu\varepsilon$ @ 10 Hz
1.0	4	22	1	3.3
2.0	4	44	1	6.6
5.0	4	110	1	16.6
10.0	2	110	1	33.0
20.0	2	220	1	66.0
50.0	2	550	1	165.0
100.0	2	1100	1	330.0

Note 1: The theoretical maximum strains for PPV levels above 3 mm/s were well above strains measured in the investigation.

Note 2: It is emphasised that the strain levels given in the table are very conservative, and should be used only for a very preliminary strain assessment. If the PPV or the peak component velocity at eave height, the dominant frequency, and the wall shape are known, a more refined estimation may be made.

10.2 Failure Strains of Material

The failure strains at which commonly used building materials have been determined to fail are listed in **Table 22**.

Table 22 – Failure strains of material

Material	Failure Mode	Failure Strain (μmm/mm)
Gypsum Plasterboard	Tension/Shear	800 - 1100
Clay-brick Masonry	Shear	250 - 1000
Concrete Block Masonry	Shear	160 - 1000
Concrete (refer AS3600)	Shear	700 - 860
	Compression	875 - 1080

Note: The lower range of failure strains shown conform with standard laboratory testing methodology. Field experience obtained in this investigation showed that the failure strains were at or above the highest strains in the range.

10.3 Natural Strains due to Material Properties

The strains caused by natural forces and events are listed in **Table 23**. The strains produced by natural environmental loads can be in excess of crack strains. For example, it can be seen that the strains associated with concrete curing are in excess of the tensile failure strains, which is the explanation for the commonly observed shrinkage cracks in concrete paths and slabs and hollow block walls.

Table 23 – Natural strains due to material properties

Material	Failure Strain	Natural Strains (μmm/mm)			
		30°C Temperature Change	Material Moisture Change	Time Expansion	Time Shrinkage
Plasterboard	800 - 1100	485	576	-	-
Solid Plaster	200 - 300	330 - 485	-	-	-
Bricks (clay fired)	250 - 1000	150	25 – 166	600 - 1100	-
Concrete (slabs/blocks/pavers)	75 - 1000	390	200 - 600	-	200 - 1000
Timber (across grain)	-	-	3% - 15%	-	-

Note: The lower range of failure strains shown conform with standard laboratory testing methodology. Field experience obtained in this investigation showed that the failure strains were at or above the highest strains in the range.

The range peak of strains induced in a brick veneer house by exposure to ground vibration at the regulatory limits of 5 mm/s to 10 mm/s is 17 με to 110 με, which is about 1.7% to 10% of the failure strain of plasterboard or clay brick masonry.

10.4 Example of an Overall Assessment

The range peak of strains induced in a brick veneer house by exposure to ground vibration at the regulatory limits of 5 mm/s to 10 mm/s is 17 με to 110 με, which is about 1.7% to 10% of the failure strain of plasterboard or clay brick masonry.

The strains induced by natural environment loads, in the absence of foundation movement, such as temperature/moisture change and ceramic growth or concrete shrinkage, can be in excess of the failure strain of the common building materials. The 30°C temperature change strains listed may be excessive for plasterboard, but is conservative for materials used externally in unshaded locations.

The strains induced by even minor foundation settlements of 2.5 mm over a short span can also be in excess of the failure strains of common building materials and is a common cause of observed cracks. The strains induced by excessive foundation soil movement on buildings with poorly engineered footings can cause severe damage (by any classification) requiring extensive repair work.

11. COMPLAINT INVESTIGATION PROCEDURES

When investigating complaints relating to blast vibration damage, the following procedures should be conducted (not necessarily in the order listed):

- Record of complaint.
- Personal contact.
- Determine likely blast vibration exposure levels.

- Building inspection and record of damage:
  - May be informal initially.
  - A more detailed inspection by a qualified engineer or building surveyor may be required.
- Relate observed damage to possible causal mechanisms.
- Rational analysis to compare vibration induced strains to natural load strains and damage strains.
- Conclusion:
  - What were the most likely mechanisms that caused the observed damage from the evidence available?

11.1 Record of Complaint

Includes the time, date, complainant name, address and telephone number, the nature of the complaint, the date of the specific blast causing the damage or the timeframe of the crack/defect development.

11.2 Personal Contact

Most importantly, at the earliest opportunity, organise an appointment to inspect the damage concerned.

11.3 Determine Likely Blast Vibration Exposure Levels (for the complaint blast or over the complaint period)

- Extrapolation from vibration levels recorded at nearby monitoring stations.
- Estimate from the Mine Site Law (if determined) or General Site Law.
- Monitor the adjacent blast to the complaint blast at the complainant's house. Use the same loading and firing specifications for the second blast, if possible. The subjective judgement of the complainant may be requested to compare perceptions of the vibration from the two blasts.
- If necessary, build up a vibration history over a period of time by routine monitoring.

11.4 Determine Likely Vibration Exposure Levels

Plot complaint location on a map, estimate maximum ground and air vibration levels by extrapolation from within a nominal timeframe or for a specific blast. Determine the maximum ground and air vibration levels the house has been subjected to leading up to and including the complaint blast.

11.5 Inspect Damage and keep a Record of Observed Damage/Defects

This may be informal at first, but collect and record sufficient information to justify an opinion as to the mechanisms causing the cracks/defects.

Note particularly:

11.5.1 Externally

- Provision for taking roof/surface drainage away from the house – adequately maintained gutters and downpipes, paving slope, downpipe discharge.
- Garden beds against foundations.
- Large trees near the house. Trees planted during the life of the structure may change prevailing conditions. Trees may cause local lifting of paving, fences and walls, but may also lower the surface by removing water from the soil profile.
- Possible ponding of water near the house or water flowing beneath the house.
- Sagging or bulging weatherboards, window frames skew to the opening, tapered cracks in bricks, all indicating foundation movement.
- Crack location width and attitude (horizontal, vertical or angle). Identify pivot area of tapered cracks. Note multiple episodes of crack repair.

11.5.2 Room by Room

- Floor level and 'bounce' and a constant gap between floor and skirting board.
- Crack location, width and attitude (H, V or angle).
- Whether cracks are tapered or a constant width, determine relative movement directions and pivot points. Constant crack widths indicate a uniform shrinkage, tapered cracks indicate rotation and flexure.
- Location of vibration sources – air conditioners, automatic washing machines, etc.
- Look for characteristic cracking:
  - Plasterboard: at butt sheet joins (especially unreinforced joins), at cornices, nail heads and 45^{o} to door frames and windows.
  - Brickwork: at damp-proof course (if any), at window corners, above doors and at re-entrant corners. Note if control joints have been used.
  - Rendered plaster: cracks and 'druminess' perpendicular to subsidence direction, cracks at cornices and stress concentrations at corners of windows and doors, tap wall for 'druminess' indicating break of bond between brickwork and plaster lining.
  - Concrete cracks: at 1.5 metres to 2 metres spacing and at stress concentrations, note shrinkage joints and provision for expansion; crazing of surface. Ceramic tiles often reflect cracks in the slab beneath.
  - Concrete bricks and blocks frequently show uniform width vertical shrinkage cracks.
  - The modern fibre replacement for AC sheet is not stable when used in bathrooms without provision for expansion. Buckled fibre sheet substrate may buckle and crack and lift ceramic floor and wall tiles.

Often there is a pattern to the cracking that is consistent with subsidence or other movement of the structure (eg. walls tilting or foundation settlement).

11.6 Consider the Evidence

If possible, form an opinion from the evidence available at the time of inspection as to what is the most likely mechanism causing the cracking or defect complained of or the mechanisms that the cracks are consistent with. Opinions formed on site can be tested by searching for further supporting evidence. Initial conclusions and opinions should be based on a basic assessment technique that could be conducted by observant mine staff, especially staff with training and awareness.

A more formal inspection may be required by an experienced building surveyor or engineer because of ongoing or insistent owner concerns and may include some or all of the following:

- Production of a floor plan.
- Room by room map of cracks and defects, including location, length, width, attitude, etc and photographs, as required.
- Internal floor level survey or, less invasively, an external brick course level loop survey which will identify foundation settlement greater than 10 mm. Level survey of the house block to establish drainage lines, if the site is very flat.
- Geotechnical investigation of the foundation soil properties and foundation width and depth to quantify the adequacy of the footing system.
- Inspection of the ceiling cavity and below floor crawl space to check on the adequacy of floor and ceiling support.
- Relate observed damage and defects to possible casual mechanisms. Damage and defects have a cause. Use observations of cracks to identify structural movements necessary to form the cracks.

Rational analysis by comparison of the vibration induced strains with material damage or failure strains and strains from natural forces and events, if necessary. Proportion the possible additional dynamic loading induced by blast vibration as a contributing factor to observed damage, if required.

A recommended methodology for the investigation of complaints and assessment of damage has been attached as **Appendix A**.

12. SUMMARY OF KEY FINDINGS OF THE INVESTIGATION

1. The structure response of three brick veneer houses to blast induced vibration was measured by accelerometers, geophones and strain gauges. The measured accelerations and velocities w ere integrated or double integrated to determine displacements and amplification factors from ground level to eave level. The measurements permitted a procedure to be developed for determining the maximum strains induced in a structure from vibration.

2. The strains determined from worst case acceleration and velocity measurements proved to be conservative when compared to strains measured directly on the houses. The measured strains were within the range of strains determined by the methods developed in this investigation.

3. The strength properties of masonry veneer of the three test houses were determined and found to be variable but typical of domestic construction.

4. The first test house was subject to ground vibration levels sufficiently high to cause observable minor damage. At vibration levels from 1.5 mm/s to 20.5 mm/s no additional damage was recorded. At a vibration level of 71.2 mm/s, a section of plasterboard ceiling that had been incorrectly installed, sagged when, in the absence of adhesive, the nail heads pulled through the paster sheet and backing paper. When repaired, exposure to vibration levels up to 222 mm/s caused no further sagging of the ceiling.

5. At vibration levels in excess of 70 mm/s, there was minor cosmetic plasterboard damage, such as crack extensions, new hairline cracks and cracks around nail heads.

6. The crack width gauges showed permanent increases in crack widths of up to 0.3 mm from the largest blasts (190 mm/s and 222 mm/s).

7. No additional damage was observed to ceramic tiles in the hallway, laundry, bathroom and kitchen. No damage was observed in the concrete floor slabs of the garage or verandah. There was no additional masonry damage. There was no damage to glass windows or sliding doors. The only additional damage attributable to blasting was minor damage to internal plaster that could be easily repaired by filling and painting over.

8. The dynamic crack width measurements showed that with vibration levels up to 20.5 mm/s the maximum temporary width movement was 0.08 mm, which did not result in permanent displacement. At the environmental ground vibration limit of 10 mm/s, the maximum temporary width movement is about 0.04 mm.

9. The footings of two of the houses were sub-standard by modern requirements for the soil conditions found in the geotechnical investigations. Both houses had defects caused by footing movement before the commencement of blasting. The one house with footings regarded as adequate by the current Standard, showed no external brickwork cracks in the 5 years since construction and very little movement by level loop survey over a period of 10 months.

10. Reactive clay soil occurs throughout the Hunter Valley and houses with inadequately engineered footings have recorded damage ranging from slight to severe (AS2870-1996; Damage Categories 1–4) in areas without blast vibration.

11. A rational method has been developed for the estimation of the range of strains that the dynamic loading from blast vibration imposes on a structure without the need for detailed structural analysis. These estimated strains can be compared with the failure strain properties of the construction materials of the building and the strains imposed by natural environmental loads.

13. CONCLUSIONS

- Following an investigation in which the structure response of brick veneer test houses in the Muswellbrook and Singleton areas to blasting was measured, and the strength of their structural elements have been determined, analysis has shown that the stresses, due to blast vibration that are within currently enforced environmental limits, are well below damage levels.

- The vibration levels at which observable damage to houses occurred from blasting compares to the level determined from structural response and strength of materials considerations.

- The structural response effect of ‘natural factors', such as ground movement and rainfall, has been determined and compared to the strength of materials and found to be significant in the formation and propagation of cracks in buildings.

- The type of structural defects observed in the test houses have been observed in reference houses not exposed to blast vibration or mine subsidence.

- The results of this investigation regarding blast vibration levels, structure response and observed damage is consistent with authoritative overseas studies.

REFERENCES

Australian and New Zealand Environment Council (ANZEC) (1990): Technical Basis for Guidelines to Minimise Annoyance due to Blasting Overpressure and Ground Vibration.

Australian Standard AS2187.2-1993 Part 2: Use of Explosives.

Australian Standard AS2870 – 1996: Residential Slabs and Footings – Construction.

Australian Standard AS3600 – 1998: Masonry Structures.

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APPENDIX A

METHODOLOGY FOR THE INVESTIGATION OF BLAST VIBRATION DAMAGE CLAIMS

When investigating blast vibration damage claims it is important to assess:

- What level of stress resulted in the structure due to the blast vibration?
- What the level of stress can the structure withstand (ie. what was the strength of the structure)?
- What level of stress has resulted from non-blasting factors, such as shrinkage and reactive soils?

The procedures that should be carried out are summarised below:

- Record the complaint.
- Make personal contact.
- Determine the likely blast vibration exposure levels (both ground vibration and airblast overpressure).
- Inspect the building and record any defects and/or damage. In some cases a brief informal inspection may be sufficient, but in other cases a more detailed inspection by a qualified engineer or building surveyor may be required.
- Relate observed damage to possible causal mechanisms.
- Perform a rational analysis to compare vibration induced strains to natural load strains and damage strains.
- Reach a conclusion, based on a consideration of the most likely mechanisms that caused the observed damage from the evidence available, including an assessment of the stresses and strains involved.

Further details are given below:

A.1 Record of Complaint

Include the time, date, complainant name, address and telephone number, the nature of the complaint, the date of the specific blast causing the damage or the time frame of the crack/defect development.

A.2 Personal Contact

At the earliest opportunity, organise an appointment to inspect the damage concerned.

A.3 Determine Likely Blast Vibration Exposure Levels (for the complaint blast or over the complaint period)

Methods which may be used include:

- Extrapolation from vibration levels recorded at nearby monitoring stations.

- Estimate from the Mine Site Law Parameters (if determined) or General Site Law Parameters.

- Monitoring of the adjacent blast to the complaint blast at the complainant's house. Use the same loading and firing specifications for the second blast, if possible. The subjective judgement of the complainant may be requested to compare perceptions of the vibration from the two blasts.

- If necessary, build up a vibration history over a period of time by routine monitoring.

A.3 The determination of likely vibration exposure levels will require the complaint location and the location of the blast and measurement stations to be plotted on a map or scaled aerial photo

As well as considering the blast vibration levels for a specific blast, it may be necessary to consider the maximum ground and air vibration levels that the house has been subjected to leading up to and including the complaint blast.

A.4 Building Inspection and Recording of Observed Damage or Defects

This may be informal at first, but requires the observation and recording of sufficient information to justify an opinion as to the mechanisms causing the damage or defects.

Particular attention should be paid to:

External

- Provision for taking roof/surface drainage away from the house – adequately maintained gutters and downpipes, paving slope, downpipe discharge.

- Garden beds against foundations.

- Large trees near the house. Trees planted during the life of the structure may change prevailing conditions. Trees may cause local lifting of paving, fences and walls, but may also lower the surface by removing water from the soil profile.

- Possible ponding of water near the house or water flowing beneath the house.

- Sagging or bulging weatherboards, window frames skew to the opening, tapered cracks in bricks, all indicating foundation movement.

- Crack location width and attitude (horizontal, vertical or angle). Identify pivot area of tapered cracks. Note multiple episodes of crack repair.

Internal:

- Floor level and 'bounce' and a constant gap between floor and skirting board.

- Crack location, width and attitude (H, V or angle).

- Whether cracks are tapered or a constant width, determine relative movement directions and pivot points. Constant crack widths indicate a uniform shrinkage, tapered cracks indicate rotation and flexure.

- Location of vibration sources – air conditioners, automatic washing machines, etc.

- Look for characteristic cracking:
  - Plasterboard: at butt sheet joins (especially unreinforced joins), at cornices, nail heads and 45^{o} to door frames and windows.
  - Brickwork: at damp-proof course (if any), at window corners, above doors and at re-entrant corners. Note if control joints have been used.
  - Rendered plaster: cracks and 'druminess' perpendicular to subsidence direction, cracks at cornices and stress concentrations at corners of windows and doors, tap wall for 'druminess' indicating break of bond between brickwork and plaster lining.
  - Concrete cracks: at 1.5 metres to 2 metres spacing and at stress concentrations, note shrinkage joints and provision for expansion; crazing of surface. Ceramic tiles often reflect cracks in the slab beneath.
  - Concrete bricks and blocks frequently show uniform width vertical shrinkage cracks.
  - The modern fibre replacement for AC sheet is not stable when used in bathrooms without provision for expansion. Buckled fibre sheet substrate may buckle and crack and lift ceramic floor and wall tiles.

  Often there is a pattern to the cracking that is consistent with subsidence or other movement of the structure (eg. walls tilting or foundation settlement).

A.5 Consider the Evidence of the Building Inspection

If possible, form an opinion from the evidence available at the time of inspection as to what is the most likely mechanism causing the cracking or defect complained of or the mechanisms that the cracks are consistent with. Opinions formed on site can be tested by searching for further supporting evidence.

Initial conclusions and opinions should be based on a basic assessment technique that could be conducted by observant mine staff, especially staff with training in and an awareness of basic structural concepts.

A more formal inspection may be required by an experienced building surveyor or engineer because of ongoing or insistent owner concerns and may include some or all of the following:

- Production of a floor plan.

- Room by room map of cracks and defects, including location, length, width, attitude, etc and photographs, as required.

- Internal floor level survey or, less invasively, an external brick course level loop survey which will identify foundation settlement greater than 10 mm. Level survey of the house block to establish drainage lines, if the site is very flat.

- Geotechnical investigation of the foundation soil properties and foundation width and depth to quantify the adequacy of the footing system.

- Inspect the ceiling cavity and below floor crawl space to check on the adequacy of floor and ceiling support.

- Relate observed damage and defects to possible casual mechanisms. Damage and defects have a cause. Use observations of cracks to identify structural movements necessary to form the cracks.

A.6 Rational analysis to compare vibration induced strains to natural load strains and damage strains

The dynamic strains induced in a structure by ground vibration can be estimated by consideration of the racking response of the structure. These can then be compared to the static failure or cracking stains of the construction materials, which is a valid comparison if the dynamic strains are comparatively low so that fatigue is not a consideration.

The induced strains can also be compared to the strains induced by natural forces and events. The natural environmental strains may themselves exceed the cracking strains of the construction materials. If cracking has occurred, the possible contribution that vibration strains may have contributed to observed cracks may be determined and a cause weighting applied, if required.

A.7 Estimation of Dynamic Strains Induced in Buildings

The in-plane deflection places the maximum strain on the walls of a house. The maximum strain can be estimated from:

$$\text{Maximum strain } \in \; = \frac{\Delta}{L} \text{ Sin}\varphi \text{ Cos}\varphi$$

$$\Delta = \frac{\text{the peak component ground velocity x amplification}}{2 \pi f}$$

Sinφ Cosφ has a maximum value of 0.5 at 45°.

The peak component particle ranges from (0.57 → 1) x PPV.

If the peak component particle velocity is not known, the peak ground particle velocity may be used for a conservative estimation. A wall height of 2400 mm is appropriate because most brick veneer constructions are articulated by a damp-proof course and the strain calculated for higher walls is proportionately less because it is inversely proportioned to height.

A most conservative estimate then is $\varepsilon = \frac{PPV\ (g)}{2\ \pi\ f} \times \text{amplification} \times \frac{0.5}{2400}$

The conservative amplification factor is 4.0 for ≤5 mm/s and 2.0 for 5mm/s to 100 mm/s. The least conservative amplification factor is 1.0 for all vibration levels.

This is a major simplification but, as this investigation has demonstrated, results in a conservative estimate of strains induced in a house from vibration.

A very conservative estimation of the peak strains for a wall 2400 mm high are given in **Table 1**, using the range of amplifications from this research and a peak component velocity equal to the peak ground (vector) particle velocity combined with a 6 Hz and 10 Hz frequency, which represents the range of frequencies found.

Table 1 – Range of calculated peak strains for a wall 2400 mm high

PPV (ground) (mm/s)	Amplification	Maximum Strain με @ 6 Hz	Amplification	Minimum Strain με @ 10 Hz
1.0	4	22	1	3.3
2.0	4	44	1	6.6
5.0	4	110	1	16.6
10.0	2	110	1	33.0
20.0	2	220	1	66.0
50.0	2	550	1	165.0
100.0	2	1100	1	330.0

Note 1: The theoretical maximum strains for PPV levels above 3 mm/s were well above strains measured in the investigation.

Note 2: It is emphasised that the strain levels given in the table are very conservative, and should be used only for a very preliminary strain assessment. If the PPV or the peak component velocity at eave height, the dominant frequency, and the wall shape are known, a more refined estimation may be made.

The failure strains at which commonly used building materials have been observed to fail are listed in **Table 2**.

Table 2 – Failure strains of material

Material	Failure Mode	Failure Strain (μmm/mm)
Gypsum Plasterboard	Tension/Shear	1100
Clay-brick Masonry	Shear	1000
Concrete Block Masonry	Shear	1000
Concrete (refer AS3600)	Shear	860
	Compression	1080

10.3 Natural Strains due to Material Properties

The strains caused by natural forces and events are listed in **Table 3**. The strains produced by natural environmental loads can be in excess of crack strains. For example, it can be seen that the strains associated with concrete curing are in excess of the tensile failure strains, which is the explanation for the commonly observed shrinkage cracks in concrete paths and slabs and hollow block walls.

Table 3 – Natural strains due to material properties

Material	Failure Strain	Natural Strains (μmm/mm)			
		30°C Temperature Change	Material Moisture Change	Time Expansion	Time Shrinkage
Plasterboard	800 - 1100	485	576	-	-
Solid Plaster	200 - 300	330 - 485	-	-	-
Bricks (clay fired)	250 - 1000	150	25 – 166	600 - 1100	-
Concrete (slabs/blocks/pavers)	75 - 1000	390	200 - 600	-	200 - 1000
Timber (across grain)	-	-	3% - 15%	-	-

10.4 Example of an Overall Assessment

The range peak of strains induced in a brick veneer house by exposure to ground vibration at the regulatory limits of 5 mm/s to 10 mm/s is 17 με to 110 με, which is about 1.7% to 10% of the failure strain of plasterboard or clay brick masonry.

The strains induced by natural environment loads, in the absence of foundation movement, such as temperature/moisture change and ceramic growth or concrete shrinkage, can be in excess of the failure strain of the common building materials. The 30°C temperature change strains listed may be excessive for plasterboard, but is conservative for materials used externally in unshaded locations.

The strains induced by even minor foundation settlements of 2.5 mm over a short span can also be in excess of the failure strains of common building materials and is a common cause of observed cracks. The strains induced by excessive foundation soil movement on buildings with poorly engineered footings can cause severe damage (by any classification) requiring extensive repair work.

A.9 Effect of Airblast on Structures

Equivalent airblast figures are given in **Table 4**.

Table 4 – Ground vibration and equivalent racking airblast response

Ground Vibration (mm/s)	Equivalent Airblast Response ACARP Study Brick Veneer Walls (dBL)	Equivalent Airblast Response USBM Study Timber Framed Light Clad (dBL)
1	119	110
2	126	115
5	135	124
10	142	131

The 133 dBL recommended airblast damage limit in AS2187.2-1993 has the approximate equivalent structural response to 10 mm/s in racking response terms for lightly clad timber framed houses subject to vibration from quarry blasting.

Note: The equivalent airblast response figures from this ACARP study are for brick veneer walls and may not be applicable to other parts of the structure of brick veneer houses. Pending further investigation, it is recommended that the equivalent airblast response figures from the USBM study be used for parts of the structure other than brick veneer walls.

A.9 Consideration of All Evidence

Conclusions should be based on a consideration of all evidence, including the building inspection, and the determination of strain and stress levels.