Effects of Mine Blasting on Residential Structures

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Abstract: Blasting is common in the coal industry to remove rock overburden so that the exposed coal can be mechanically excavated. The ground vibrations and air blast produced by blasting are often felt by residents surrounding the mines. There has been a trend for regulatory authorities, especially those concerned with the environment, to impose low limits on blast vibration levels in response to community pressure, based on human perception and response to vibration. This paper reports the findings of an extensive study on a house which was located adjacent to a coal mine. The house was monitored for over 1 year and was subjected to ground peak particle velocity (PPV) ranging from 1.5 to 222 mm/s. The house was instrumented with accelerometers to measure its dynamic response due to blasting and it was also monitored for cracks before and after each blast. Based on this study, ground motion amplifications along the height of the structure have been established. A simplified methodology presented in this paper has been used to estimate the ground PPV at which cracking is likely.

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Introduction

Blasting is common in the coal industry to remove rock overburden so that the exposed coal can be mechanically excavated. Explosives used in open-cut coal mines are loaded into blast holes which have been drilled downwards through the overburden rock. The blast holes are then detonated in sequence and a portion of the energy released is converted to wave energy with compression (P) waves, shear (S) waves and surface Rayleigh (R) waves transmitted in all directions from the blast source. R waves receive the most energy and cause the most damage to structures since they travel along the surface of the ground, with particles moving in an elliptical path. These surface vibrations caused by the passage of the R-wave can be recorded in the two horizontal directions and one vertical direction in the form of acceleration, velocity and displacement time history traces. The frequency range of the vibrations is in the range of 2–40 Hz for a soil site of depth greater than 2 m and 10–100 Hz for rock sites (Dowding 1996). These complex time history recordings are often simplified to one value (based on the vector sum) such as ground peak particle acceleration, peak particle velocity (PPV), or peak particle displacement. The PPV is the most common measure for quantifying blast vibrations, as the velocity is approximately correlated to both building damage and annoyance levels to people.

The human body is an excellent detector of vibration but a poor measuring device. The human body perceives vibrations at velocity levels far below those needed to cause damage. Damage in buildings can be classified into two types: (1) cosmetic and (2) structural. Cosmetic damage refers to the formation of minor hairline cracks on drywall surfaces. In contrast, structural damage involves the more major cracking or failure of structural elements such as the formation of cracks through brickwork. The PPV threshold values associated with no structural damage are much greater than the PPV values needed to prevent cosmetic damage. Typically the human body can detect PPV on the order of 0.2 mm/s with clearly perceptible levels at 1.0 mm/s. The PPV needed to cause cosmetic building damage to ordinary structures varies among the different Standards worldwide but is typically in the range 5–50 mm/s (International Standard Organisation *ISO10137* 1992; British Standard *BS7385* 1993; German Standard *DIN4150* 1993). Environmental legislations in Australia typically specify ground vibration limits for residential zones of 5 mm/s for 95% of blasts and 10 mm/s for the remaining 5% of blasts (Australian and New Zealand Environmental Council 1990).

Major studies investigating the response of buildings to blast have been carried out in a number of countries. The U.S. Bureau of Mines studied blast produced ground vibrations from surface mining to assess its damage and annoyance potential during the 1970s and 1980s (Siskind et al. 1980; Stagg et al. 1984). The threshold level for cosmetic damage to residential construction was found to be in the range of 12–50 mm/s and quite frequency dependent. The report also noted that residents response and annoyance to blast vibrations was aggravated by secondary noises such as walls rattling and was at a level significantly below levels needed to cause damage. Blasting studies undertaken by the Univ. of Leeds (Blasting Research Group, Department of Mining and Mineral Engineering) during the 1990s noted that natural events such as temperature changes causing "freeze and thaw" actions were factors that contributed more significantly to cosmetic dam-

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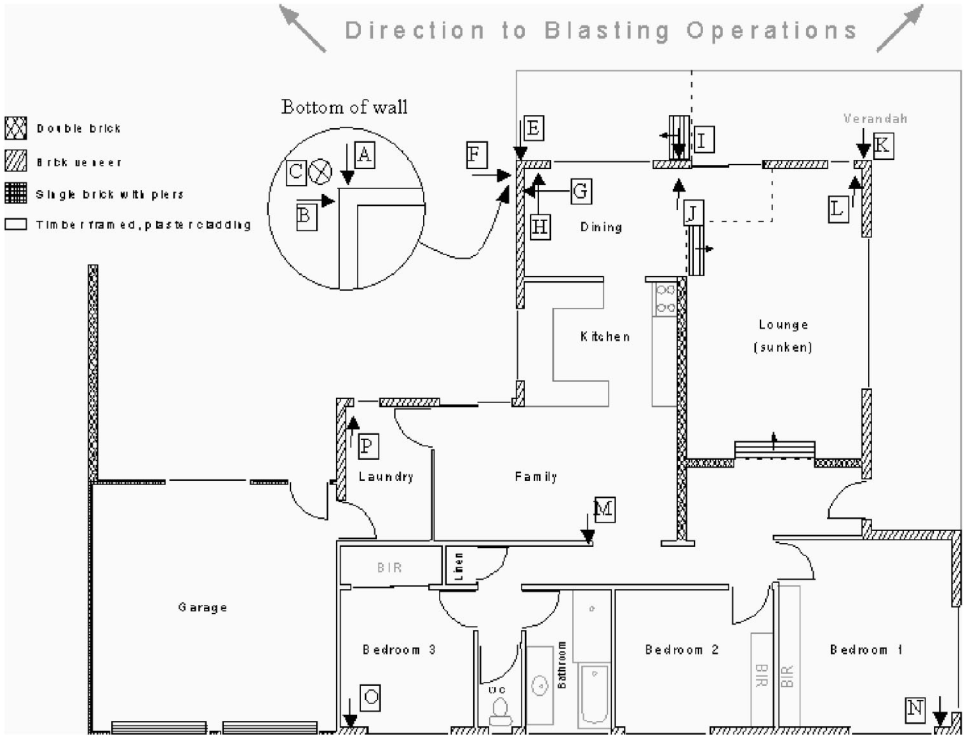

Fig. 1. Floor plan of Rix's Creek house (arrows labeled A–P indicate locations of accelerometers mounted on house to measure its dynamic response)

age of residential dwellings compared with blast induced vibrations (White et al. 1993). While these studies provide excellent resource material, the results cannot be directly translated for other countries due to the different residential construction practices, material types, building age, and environmental conditions. For example, plasterboard wall lining in the United States tends to be thicker than that used in Australia. Furthermore, the paper lining on the plasterboard tends to be of heavier weight in the United States compared to Australia. Hence, the failure stresses and strains of such manufactured materials in different countries may be different and local construction and material knowledge would be required.

This paper reports findings from an ongoing investigation on the effect of blast vibrations on Australian residential structures. As part of this investigation several houses were monitored in the Hunter Valley region in New South Wales (NSW) where a number of open-cut coal mines are operating close to townships. The houses monitored were selected to represent different construction types, distances from mines, and age. This paper focuses on results from field monitoring of a specific test house which was monitored from December 1999 to January 2001. The results from the other houses are consistent with, and reinforce, the findings presented in this paper.

Test House

Construction Form

The house is adjacent to the Rix's Creek open-cut mine located close to the township of Singleton in NSW (210 km north–west of Sydney). It is of conventional brick veneer construction, with a timber frame, 10 mm plasterboard lining for the walls and ceiling, tiled roof, and wooden floor boards. The house was constructed in the early 1970s and has a floor area of approximately 200 m^2. The brickwork is supported by strip footings while the wooden floor is supported by a series of floor joists, bearers, and masonry piers. A floor plan of the house is shown in Fig. 1 and a photo of the house showing its proximity from the mine is shown in Fig. 2.

The condition of the house at the beginning of the monitoring appeared reasonable with little evidence of deterioration from environmental effects and previous blasting. The plasterboard and brickwork had a number of cracks ranging in size from fine (<1 mm) to noticeable but easily filled (<5 mm), scattered throughout the house.

Fig. 2. Rix's creek house shown in background with section of overburden (foreground) ready for detonation

Masonry Properties

The brickwork was constructed from extruded fired clay bricks. Compressive and bond wrench tests were conducted on the brickwork by Newcastle Univ. The characteristic compressive strength was found to be 19.1 MPa. Bond wrench tests were performed in situ by removing selected bricks. The mean flexural tensile strength of the masonry was 0.25 MPa with a large coefficient of variation of 66% from 11 tests. This translated to an unrealistically low characteristic strength of 0.03 MPa. The bond strength is considered low but not necessarily atypical for domestic construction where the standard of workmanship is highly variable.

Foundations and Geotechnical Report

Soil and geotechnical investigations were carried out by Newcastle Univ. Based on the soil investigation it was found that the site can be classified as Class M, or moderately reactive in accordance with Australian Standard *AS2870* for residential slabs and footings (Australian Standard 1996).

The foundation of the structure was examined in an excavation which revealed a strip footing of between 370 and 400 mm deep which is less than the 500 mm depth required by *AS2870* for a masonry veneer dwelling on a Class M 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 *AS2870*: i.e., there is a possibility that damage exceeding Category 2 (very slight cracks, with crack width between 0.1 and 1.0 mm) may occur due to reactive clay foundation movements. In accordance with *AS2870*, cracks with widths between 0.1 and 1.0 mm are considered to be fine cracks which do not require repair.

Monitoring

Blasts

During the monitoring period (December 1999–January 2001) the test house experienced some 43 blasts with charge masses varying from 50 to 1,300 kg at distances between 50 and 1,000 m. The PPV, expressed in this paper as the peak vector sum, measured on the ground adjacent to the site varied between 1.5 and 222 mm/s. A summary of number of blasts classified according to their measured PPV is shown in Table 1.

Crack and Level Surveys

The crack lengths in all the rooms were marked and measured and, in addition, the width of some cracks were monitored using Demac gages. The lengths of all cracks were measured before and after each blast and any changes noted. Table 2 describes five typical cracks which were monitored and Fig. 3 shows photos for four of these cracks. Fig. 4 shows the changes in width for the five cracks (three cracks in the plasterboard and two in the brick veneer). The changes in crack width are overlaid on the measured PPV of each blast and also on the rainfall recorded during the monitoring.

Cracks 1 and 2 were the most active as shown in Fig. 4. However, opening and closing of cracks seemed to be more sensitive to rainfall rather than blasting activities. For example, the width of crack No. 1 did not appear to be affected by the numerous blasts with PPV less than 20 mm/s over the period April 1–August 10, 2000. A permanent offset of 0.1 mm was measured following the blast on August 11, 2000, with a peak ground velocity of 70 mm/s. In contrast, the crack closed from a width of 2.2 mm (March 5, 2000) to 1.5 mm (March 25, 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 December 7, 2000 (190 mm/s) caused the crack to close from 2.0 to 1.7 mm and the next blast (220 mm/s)

Table 1. Number of Blasts during Monitoring Period Classified According to Their Peak Particle Velocity

PPV (mm/s)	Number of blasts from December 1999 to January 2001
<5	9
5–10	8
10–20	12
20–30	4
30–50	6
50–100	2 (71 and 73 mm/s)
>100	2 (190 and 222 mm/s)

Table 2. Description of Typical Cracks Monitored in Test House and Their Widths Plotted in Fig. 4

Crack number	Location	Figure number	Description
1	Living room (internal)	—	Stepped crack in nonstructural brickwork
2	Kitchen (external)	3(a)	Vertical crack in veneer brickwork
3	Kitchen (internal)	3(b)	Vertical crack in structural brickwork
4	Kitchen (internal)	3(c)	45° crack in plasterboard
5	Laundry (internal)	3(d)	45° crack in plasterboard

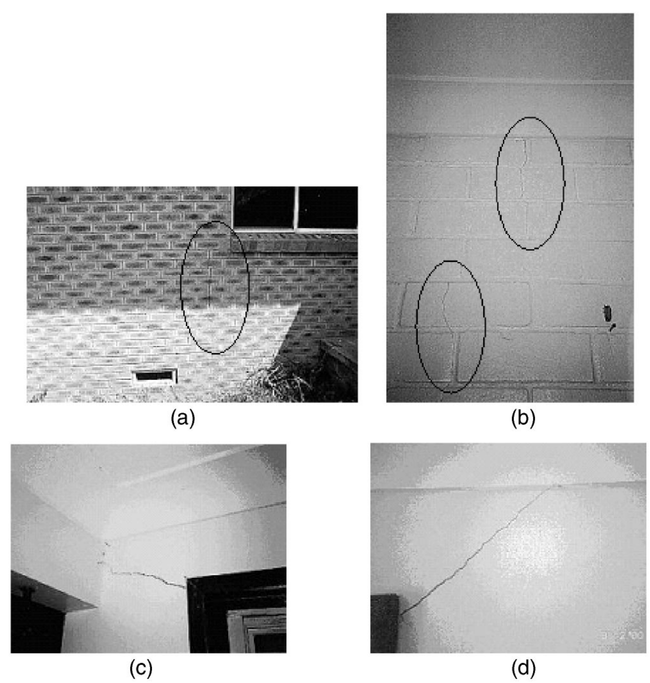

Fig. 3. Sample of cracks in brickwork and plasterboard which were monitored (crack width and length measured before and after each blast)

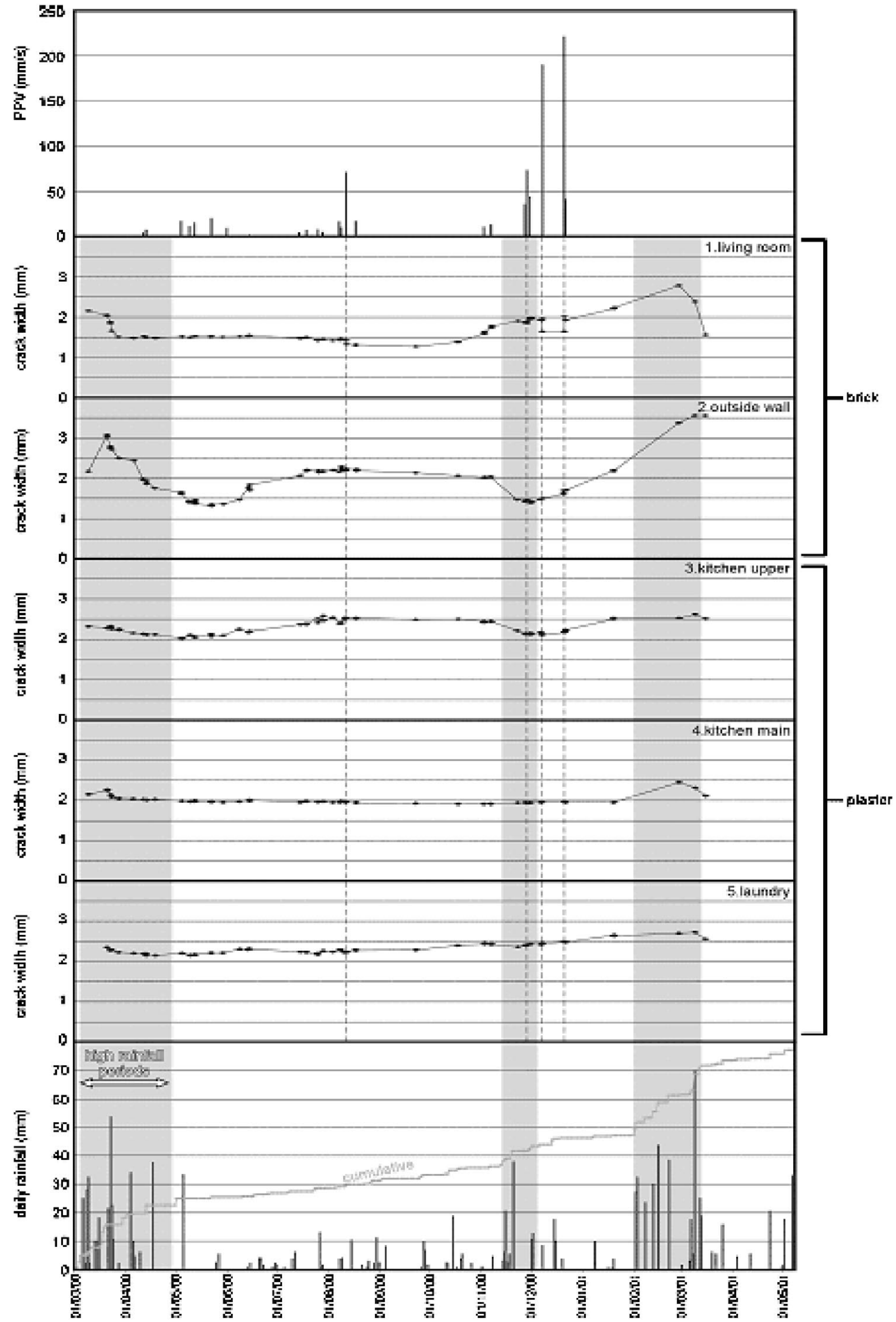

Fig. 4. Crack width movement compared to peak particle velocity and rainfall

caused the crack to reopen to 2.0 mm. It is obvious from Fig. 4 that the movements associated with the rainfall (shrinking and swelling of soil) have a profound impact on the opening and closing of cracks. The total crack length measured for the whole house also displayed the same pattern. There was no significant change in total length of cracks for blasts less than 70 mm/s, but there was significant increase of crack lengths in some locations associated with periods of high rainfall.

In addition to the crack survey, level surveys were conducted. The relative levels of the house foundations were measured twice some 5 months apart. The difference in levels between the two surveys indicated a maximum settlement of 12 mm and a maximum heave of 10 mm during the 5 months period as shown in Fig. 5. These measured movements are consistent with the predictions from geotechnical investigation.

Structural Monitoring

Fifteen accelerometers were used to measure vibrations in different locations in the house as shown in Fig. 1. Three accelerometers were located at ground level (A, B, C) to measure the two horizontal and one vertical components of acceleration, respec-

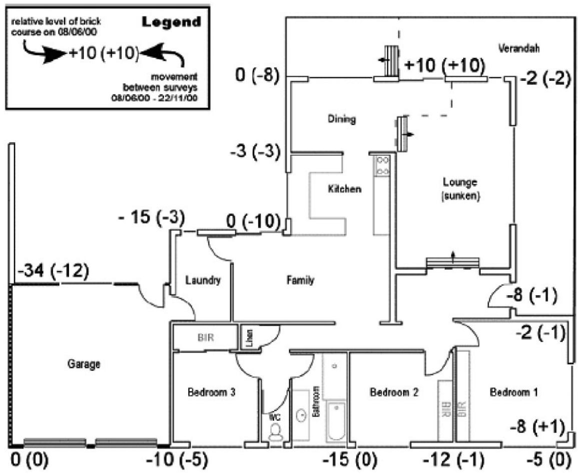

Fig. 5. Level loop survey: June 8 and November 22, 2000

tively. The remaining 12 accelerometers were orientated in the horizontal direction at ceiling level (approximately 2.4 m above floor level) with four on the external brick veneer walls (E, F, I, K) and eight on the internal plasterboard (G, H, J, L, P, M, N, O). A typical blast is presented in Fig. 6 which shows the record in both the time and frequency domains. Typical duration of blast records is approximately 4 s.

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) (also known as the global wall shear strain) which is defined by the in-plane horizontal displacement (Δ_1) of the wall at the ceiling level divided by the wall height (H) (refer to Fig. 7). The horizontal ceiling displacement can be estimated from the corresponding horizontal 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 Eq. (1)

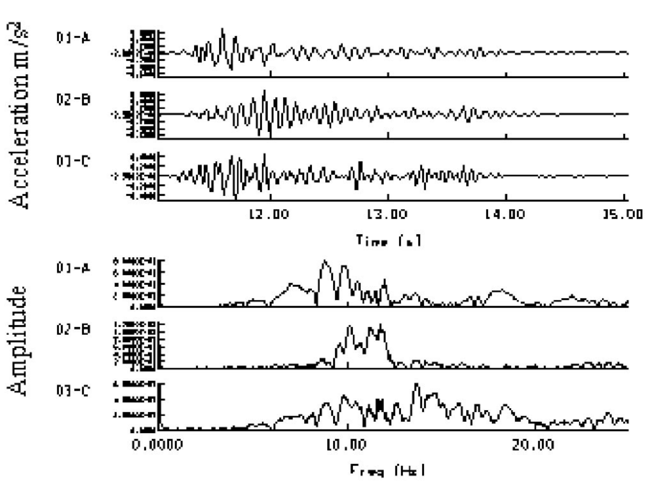

Fig. 6. Typical blast record at ground level in both time and frequency domains

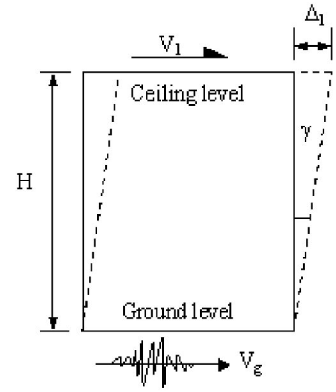

Fig. 7. In-plane shear deformation of wall due to ground vibration

$$\gamma = \frac{\Delta_1}{H} \tag{1a}$$

$$\Delta_1 = \frac{V_g}{2\pi f}\lambda \tag{1b}$$

$$\gamma = \frac{V_g}{2\pi f}\frac{\lambda}{H} \tag{1c}$$

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

$$\varepsilon = 0.5\gamma \tag{2}$$

The basis for selecting the amplification and frequency values are described in the following sections. The use of the ground PPV instead of V_g will provide conservative estimates of the structural response.

Amplification Effects

The ratios of the horizontal in-plane peak component velocity (V_1) at ceiling level to ground level were calculated to estimate the likely vibration amplification effects with height in the structure. The ratios were calculated for both the in-plane and out-of-plane directions and for both the frame and brick veneer walls. The in-plane measurements are of vital importance from damage level perspective, while the out-of-plane records are less structurally significant but do contribute to the overall vibration and noise perception of the occupants. This paper reports the in-plane results only.

The resulting amplification values varied significantly depending on the level of ground vibration, Fig. 8 plots 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 in Fig. 8, 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:

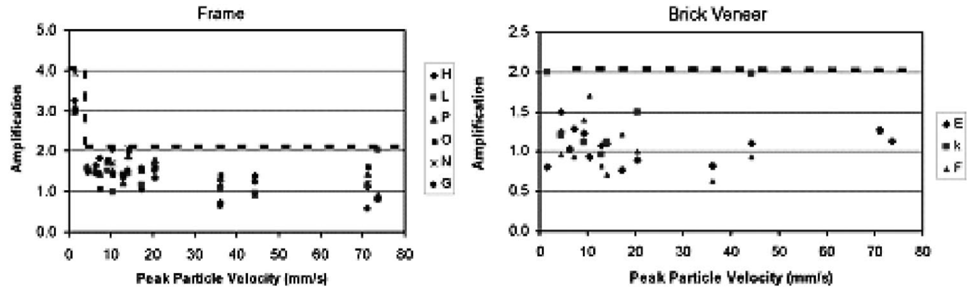

Fig. 8. Amplification of velocity in in-plane direction for frame and brick veneer

$$\lambda = 4.0 \quad \text{for PPV} \leqslant 5 \text{ mm/s} \tag{3a}$$

$$\lambda = 2.0 \quad \text{for } 5 < \text{PPV} \leqslant 100 \text{ mm/s} \tag{3b}$$

There is no obvious practical reason for this stepped function and further research is required to investigate if a smoother amplification function is appropriate between 3 and 10 mm/s. The amplification factors for the high vibration levels of 190 and 220 mm/s have not been shown in Fig. 8 for clarity but are less than 1.0.

It should be noted that the measured dominant frequency of all blasts reported in Fig. 8 was less than 30 Hz, with the majority being less than 15 Hz. Thus these blasts are considered to be among the most critical for residential structures which have a fundamental frequency of vibration in the range 5–15 Hz.

Dominant Frequency

For the measuring locations shown in Fig. 1, the acceleration records were integrated and double integrated to obtain the in-plane horizontal 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:

$$f = \frac{V_1}{2\pi\Delta_1} \tag{4}$$

It is noted that this is a major simplification, however, the method enables a realistic estimate of Δ_1 and hence the in-plane shear strain to be made. The dominant frequency tends to vary with V_g and hence PPV and was found to be in the range of 6–10 Hz. A lower bound frequency figure of 6 Hz can be used to estimate conservative values for the displacement and drift.

Quantification of Damage

The conservative values for the amplification (λ) and frequency (f) developed above have been used to estimate the horizontal ceiling displacements (Δ_1) and in-plane principal tensile strain (ε) for a single story house (ceiling height of 2.4 m) subject to different levels of ground vibration expressed in terms of PPV as shown in Table 3.

Most codes of practice around the world recommend drift ratio on the order of 1/300 (Reardon 1980) to 1/500 (*ISO4536*, ISO 1977) at the serviceability limit state to prevent damage from wind and earthquake loading. These drift ratios would correspond to blast vibrations on the order of 100 mm/s as shown in Table 3.

The principal tensile failure strain associated with plasterboard is on the order of 1,000 $\mu\varepsilon$ (Siskind 2000), which corresponds to ground vibration on the order of 100 mm/s. 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 on the order of 100–300 $\mu\varepsilon$). In contrast, a blast loading which induces racking displacements in a masonry wall would result in complex shear and tensile stresses and strains at the bricks and mortar interface. The shear strength at this interface is stronger than the corresponding tensile strength with a typical failure strain on the order of 1,000 $\mu\varepsilon$ (Stagg et al. 1984).

The strain levels presented in Table 3 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 and thermal movements. 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 on 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 for plasterboard assuming that the residual strains are on the order of 900 $\mu\varepsilon$.

In the test house, no new damage from blasting was observed for PPV less than 75 mm/s. This suggests that the residual strains in this house were relatively small and on the order of 200 $\mu\varepsilon$ or 20% of the plasterboard rupture strain.

It is recognized that for fatigue to be an issue, the dynamic strains need to be greater than some limiting threshold value. The dynamic strain associated with the normal PPV limits of blasting are generally small and less than the threshold value to cause fatigue cracking.

Table 3. Conservative Estimation of Displacements and In-Plane Principal Strains in Single Story House Subjected to Various Levels of Blast Vibrations

Ground peak particle velocity (mm/s)	Amplification λ	Frequency f (Hz)	Displacement Δ_1 (mm)	Drift γ	Strain $\mu\varepsilon$
5	4	6	0.5	1/4,500	111
10	2	6	0.5	1/4,500	111
25	2	6	1.3	1/1,800	276
50	2	6	2.7	1/900	553
75	2	6	4.0	1/600	829
100	2	6	5.3	1/450	1,105

Conclusions

This paper has described part of an ongoing investigation into the effects of blast vibrations on houses. The results from a typical single story brick veneer house, which was monitored for over 1 year, have been presented. During the monitoring period, the house was subjected to 43 blasts with ground PPV ranging from 1.5 to 222 mm/s. Crack surveys were performed before and after each blast. No additional damage due to blasting was observed for blasts with PPV below 75 mm/s.

Based on the measured dynamic response of the house, it was found that the amplification factor of the ground PPV at ceiling level was a maximum of four for the in-plane walls (racking motion) and PPV of 5 mm/s and less. For ground PPV between 5 and 100 mm/s, the amplification factor was found to be a maximum of 2.

Using a simple degree of freedom analogy, and adopting the obtained conservative amplification factors with conservative estimates of the natural frequency of the house, the maximum strains in the plasterboard in the house were estimated for different ground PPV values. These results suggest that the plasterboard is unlikely to crack at low ground PPV levels (less than 10 mm/s) unless the residual strains are extremely high (more than 90% of the failure strains).

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Notation

The following symbols are used in this paper:

f = dominant frequency of vibration of house;
H = wall height;
V_g = horizontal ground peak component velocity;
V_1 = in-plane peak horizontal velocity of wall at ceiling level;
γ = drift ratio (also known as global wall shear strain);
Δ_1 = in-plane horizontal displacement of wall at ceiling level;
ε = global principal tensile strain of wall; and
λ = amplification of velocity between ground and ceiling.

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