

LIDDELL COAL OPERATIONS

APPENDIX **N**

Chain Of Ponds Inn - Investigation
of Blast Vibration Vulnerability
(Bill Jordan and Associates, 2013)

APPENDICES



FORMER CHAIN OF PONDS INN BUILDINGS

INVESTIGATION OF BLAST VIBRATION VULNERABILITY



REPORT FOR LIDDELL COAL OPERATIONS

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Role	Name	Signature	Date

FORMER CHAIN OF PONDS INN BUILDINGS

INVESTIGATION OF

BLAST VIBRATION VULNERABILITY

1 FINDINGS

The three buildings of the former Chain of Ponds Inn at Ravensworth were instrumented to determine their reaction to ground vibrations caused by blasting at the nearby Liddell Coal Operations mine.

In this report are given details of the work done and proposals for mitigating the effects of greater ground vibration magnitudes. With implementation of the recommendations, I am confident that ground vibration peak particle velocity (PPV) levels up to 50 mm/s, with frequency control, will be acceptable at the site.

2 INTRODUCTION

2.1 Description

The remaining buildings from the former Chain of Ponds Inn, situated on the Old New England Highway at Ravensworth, date from the 1840s: the main part of the inn building (Building 'A') and the kitchen block/servants' quarters (Building 'B') are of ashlar stone construction; the stable block (Building 'C') is of brickwork, with stone quoins and lintels, and the later rear wing of Building 'A' is of brick construction.

The buildings have been dilapidated for many years and all have extensive termite damage to structural and joinery timbers; new roofing has been placed on all buildings to give protection from water damage, and termite control is now in place.

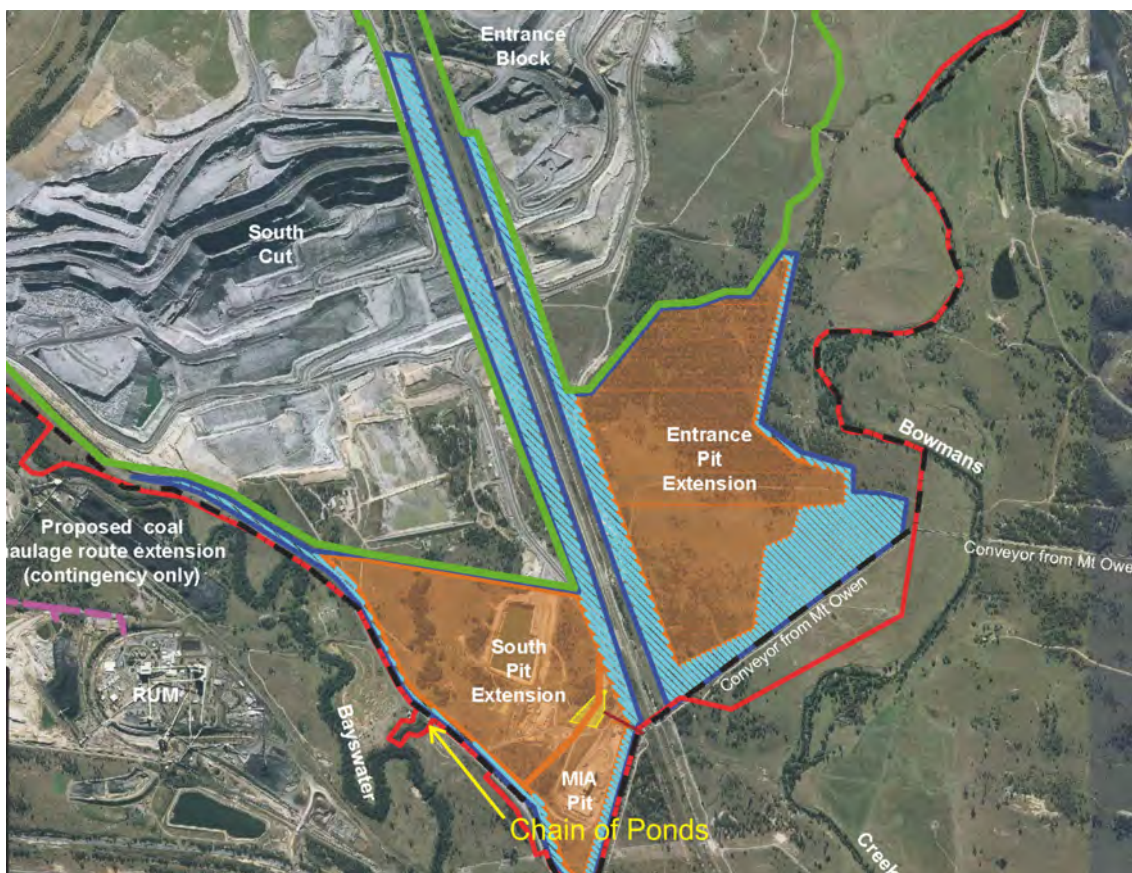


Figure 1: Aerial photo (from GSS Environmental) showing relationship of current and proposed mining with the Chain of Ponds buildings. c.f. Appendix A.

The location with respect to the mine is shown in Figure 1 and a close-up aerial view of the buildings, annotated to show the blast monitoring station, in Figure 2. A graphic giving better details of the proposed mining is in Appendix A.



Figure 2: Close-up aerial photo (from Google) of the buildings, annotated to indicate geophone location and building designations used in this and other reports.

2.2 The brief

GSS Environmental have been retained by Liddell Coal Operations to undertake an Environmental Impact Assessment to accompany an application to the NSW Department of Planning to allow for an extension of open cut mining at Liddell.

Bill Jordan & Associates have been providing advice on vibration vulnerability at Chain of Ponds for some years and, using earlier technology, work was done from 2006 which resulted in blast vibration levels at the property being raised from 2 mm/s PPV to 10 mm/s.

The new proposal, bringing mining much closer to the Chain of Ponds property, has required a reassessment of the earlier work. In recent years Bill Jordan & Associates has acquired very sensitive vibration measuring equipment and has developed processes for evaluating frequency-related structural behaviour when subject to ground vibration.

2.3 Previous work

The first phase of the earlier work was detailed in a number of reports from ourselves and sub-consultants, Robert Bird Group, in early 2007. That work involved the finite element modelling of Building 'A' with application of synthetic ground vibrations to the model using procedures developed for earthquake analysis. Subsequently in 2010, Building 'B' was modelled using a frame analysis method to determine its modes of vibration as a first stage to applying frequency-based criteria in future work.

Both the finite element analysis of Building 'A' and the subsequent work on Building 'B' indicated that fundamental vibration modes of both buildings were at frequencies of about 15 Hz.

3 THE ASSESSMENT

3.1 Work undertaken

3.1.1 MONITORING SET-UPS

Monitoring of the three buildings was undertaken on three days (14, 20 & 22/3/2013) for a total of five blast events. In addition, recordings of building vibrations were undertaken when they were being acted on by environmental actions, such as wind. The monitoring set-up locations are shown in Figure 3.

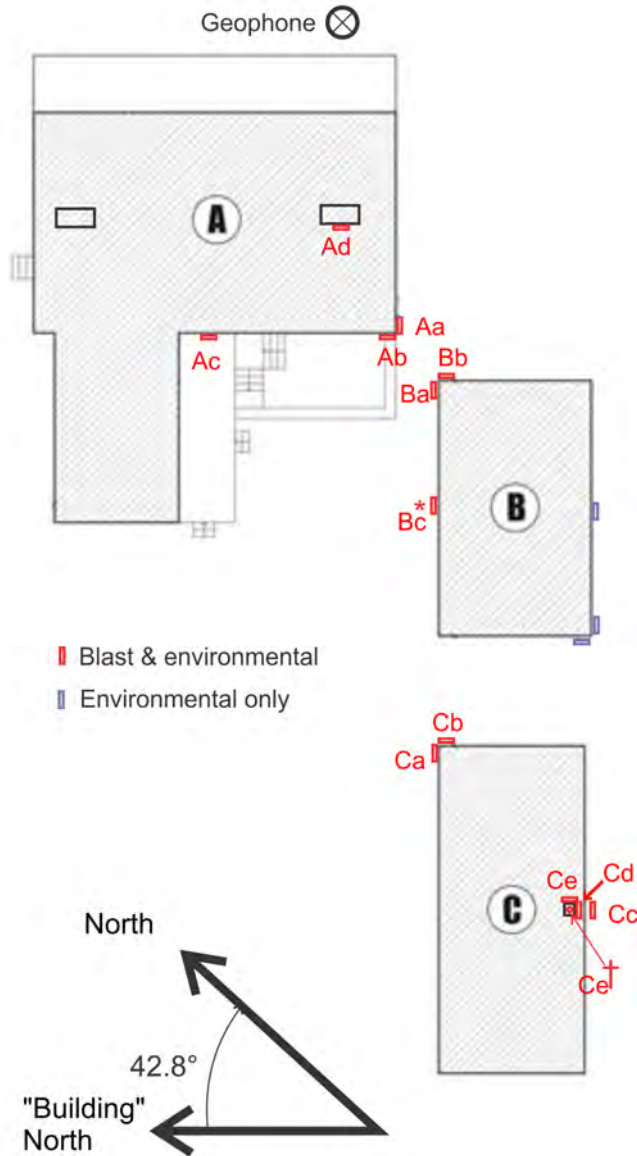


Figure 3: Locations of accelerometers used for building evaluations, together with geographic orientation of buildings.

All accelerometer positions measure horizontal movements except that marked †, which measured vertical motion at chimney top.

With the exception of position marked *, which was at mid-height on the wall, all positions were at the tops of walls or chimneys.

The code numbers ("Aa" etc.) are used in subsequent discussions.

Three Silicon Designs Model 2240-002 and three Model 2266-002 accelerometers were used for each monitoring set-up. The accelerometers are lightweight (10 g) so that their mass does not influence the reading, as can happen when heavy sensors are attached to building fabric. The accelerometers are also very sensitive so that in some situations useful data can be obtained from natural building movements without blast activation. The accelerometers were attached using a special wax formulation designed to be removable from sensitive heritage-significant surfaces without damage.

The accelerometers were connected to a Kelunji EchoPro seismic recorder which stored the measurements for later downloading and analysis. The recorder is connected to a GPS sensor which records position and time data; GPS time is accurate to less than a millisecond, so allowing a useful means of comparison with the ground wave monitor.

The vibrations were sampled at a frequency of 1000 Hz. The locations were chosen to best show the most damaging building movements with the least number of set-ups. In general, corners were chosen as the motion of the whole building swaying can be measured at these locations without influence from flexing of walls. Chimneys were chosen also as chimneys, being higher, can have a lower natural frequency than the building itself, depending on how well they are coupled to the walls. The vertical measurement was taken on the Building 'C' chimney as that chimney is only supported on deteriorating timber at first floor level.

Photographs of some of the set-ups are shown in Figures 4 to 9.



Figure 4: Accelerometers placed on the top south-western corner of Building 'A' to measure whole-of-building movement on two principal axes.



Figure 5: Single accelerometer placed at centre of rear wall of Building 'A' to gauge wall flexure. Note large crack separating stone wall from extension would have allowed independent movement.



Figure 6: Accelerometer placed at top of southern chimney of Building 'A'.



Figure 7: Accelerometers being attached to chimney of Building 'C'.



Figure 8: Attachment to top north-eastern corner of Building 'C'.



Figure 9: Indications of possible earlier out-of-plane movement of northern wall of Building 'B' suggested value of the mid-height monitoring of the wall.

3.1.2 RECORDINGS

Recordings were obtained for each of the blasts together with a significant movement due to wind action on a number of occasions. Representative raw recording graphs are shown in Figures 10 and 11.

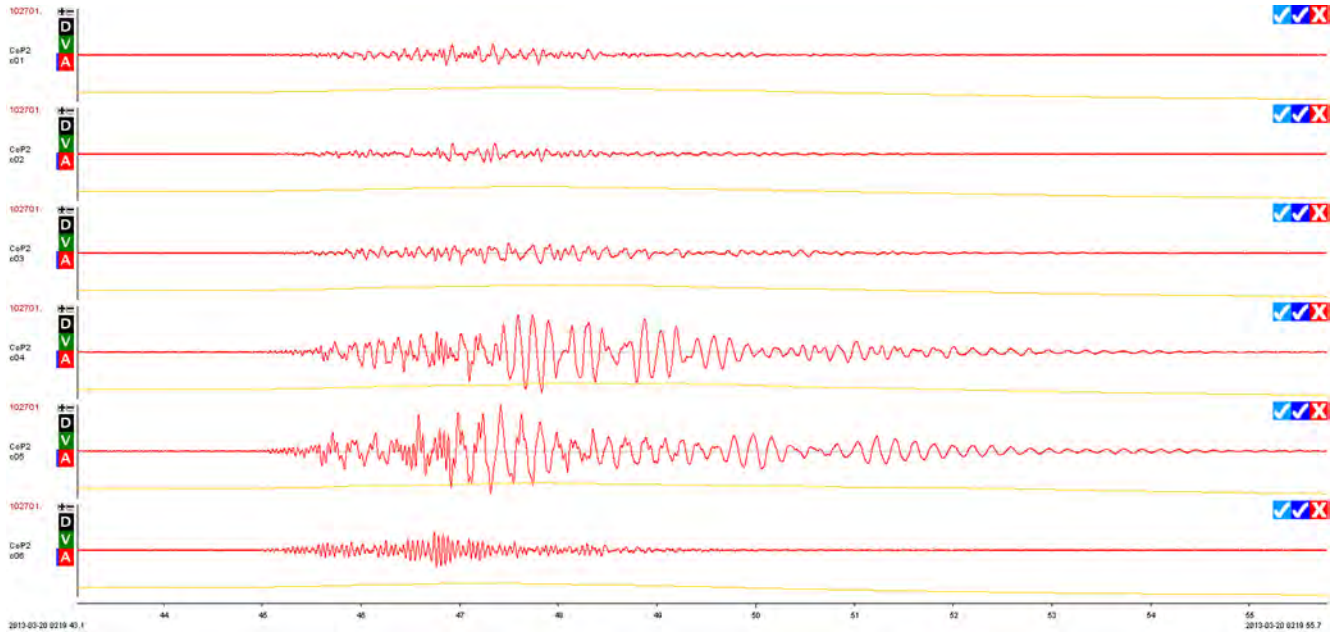


Figure 10: Vibration recording from blast event of 20/3/2013 on Building 'C'. Accelerometers 1 to 3 were on the building itself with accelerometers 4 to 6 on the chimney. These recordings, which are all to the same scale of acceleration units, show the marked difference between the chimney movement and the building as a whole. Recording channel 6, the lowest trace, is the vertical motion of the chimney.

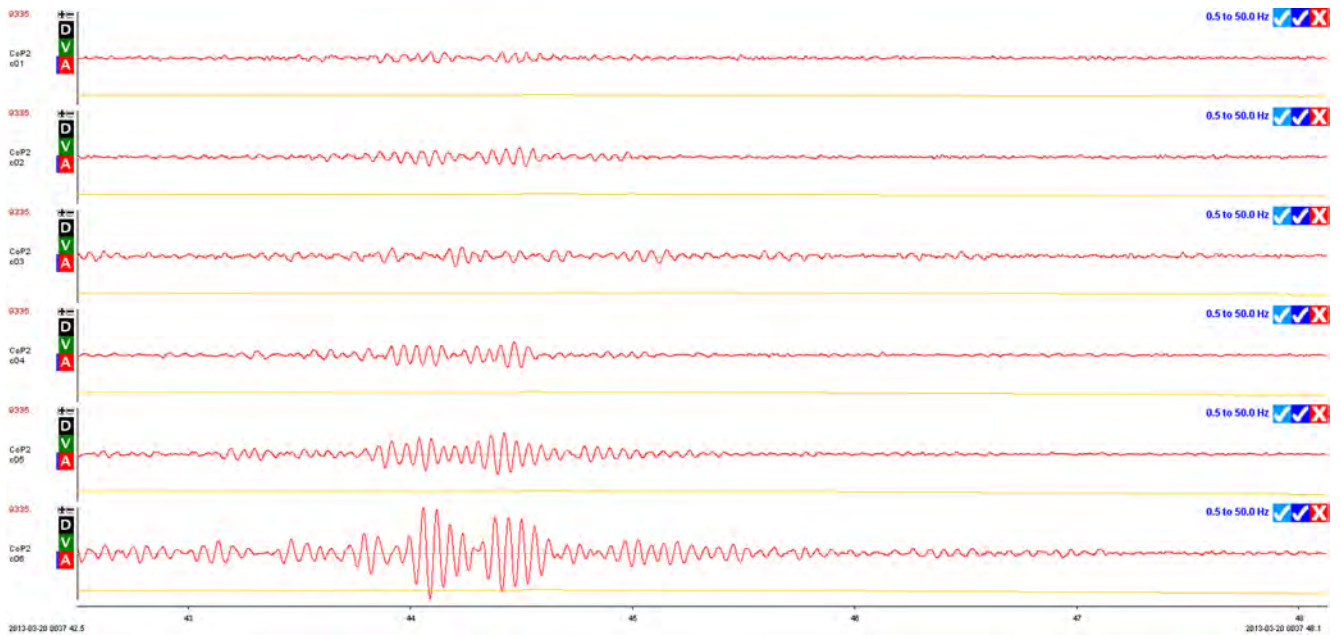


Figure 11: "Environmental" recording from Buildings 'B' and 'C' on 20/3/2013. Channels number from top to bottom with channels 1 to 3 being corner and wall centre on Building 'C' and channels 4 to 6 corner and wall centre on Building B. This recording was made about one hour before the blast and the channel 4 to channel 6 accelerometers were then moved to the chimney of Building C. The time scale on the horizontal axis is seconds.

The activator appears to have been a wind gust and provides a good indication of the natural frequencies of the buildings after very low (<0.5 Hz) and high (>50 Hz) frequencies have been filtered out.

3.2 Analysis

All the accelerometer recordings were analysed as follows:

- spectrograms showing the building vibration frequencies with respect to time were produced for each accelerometer using a specially written script in the Matlab program;

- the accelerometer waveforms were integrated to produce velocity waveforms and integrated again to produce displacement waveforms.

Spectrograms are derived by undertaking a “moving” Fourier Transform analysis of the waveform. Unlike a static Fourier Transform analysis, usually taken over the whole waveform recording, the spectrogram approach shows how the frequencies in the recording vary with time. The resulting colour plot varies in the order of the visible light spectrum from dark red for the highest magnitude to dark blue for zero magnitude.

The velocity data derived from the accelerometers is used to determine amplification factors, that is to show the ratio of the particle velocity experienced in the building compared to the ground. Displacement data can be compared to accepted limits for strain in various building materials.

Typical plots from this project are illustrated in Figures 12 to 14.

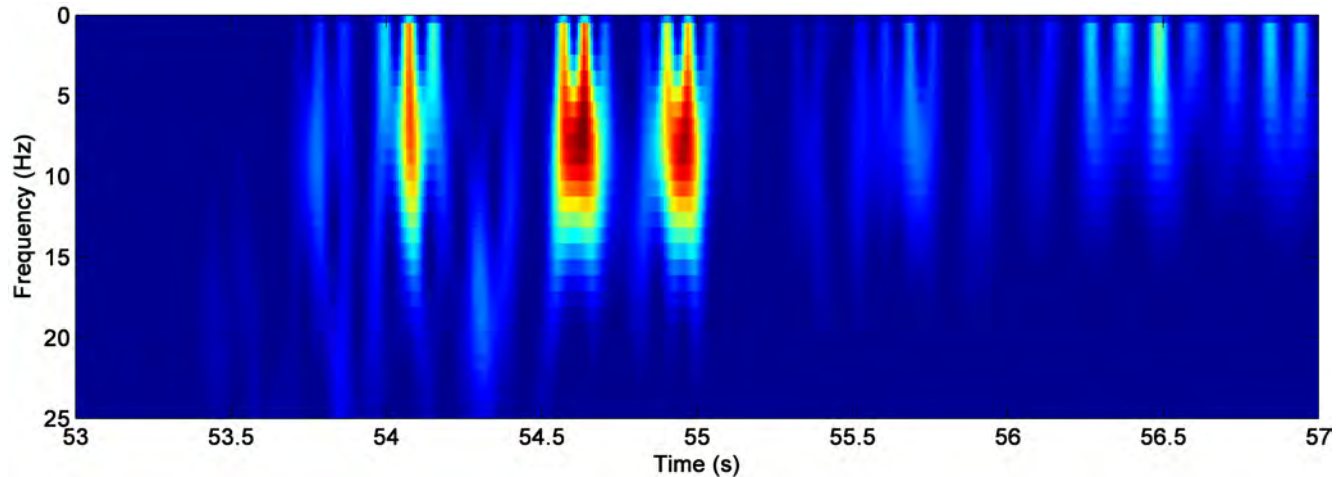


Figure 12: Spectrogram of chimney on Building 'A', (channel C05 accelerometer at position Ad) for blast 1 on 22/3/2013. The ground wave bears only passing relationship to this reaction (see full comparison in Appendix A), distinctly showing that the chimney has a natural frequency of about 8 to 9 Hz.

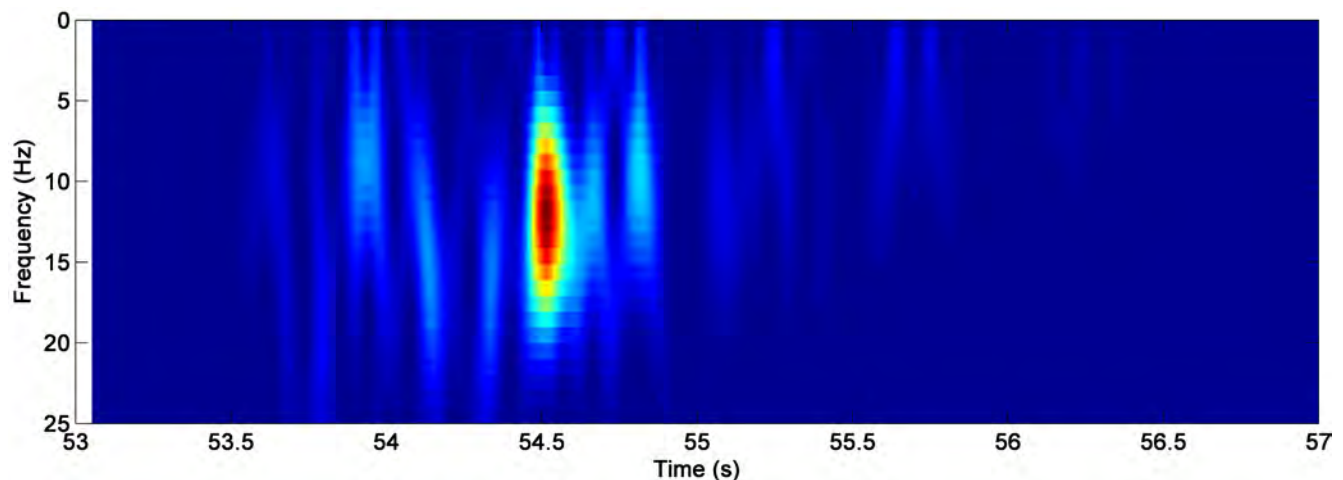


Figure 13: The ground wave spectrogram from the same event and oriented on the same axis shows how the chimney can react significantly to relatively low vibrations close to the resonant frequency. The principal energy burst in the ground wave is above 10 Hz, for which the chimney's reaction is no greater than seen for lower ground wave amplitudes nearer the resonant frequency.

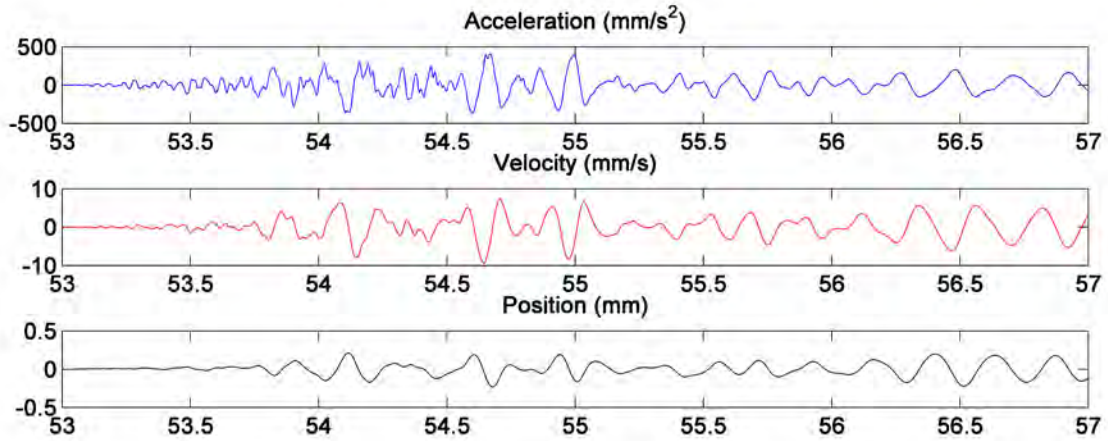


Figure 14: Plots of acceleration, velocity and displacement from accelerometer C05, position Ad, for the same event as figures 12 and 13.

4 BLAST RECORDS

4.1 Geophone recordings

There is an “Ecotech” geophone, together with a microphone, located adjacent to the front verandah of Building ‘A’. The recordings from this recorder were provided as spreadsheet .csv files which were transferred to the analysis software. Trigger times were also recorded, allowing a direct comparison of the ground motion and the induced building motion. The microphone results, recorded in pressure units of pascals, can be used directly to assess air pressure actions on the building or converted to the linear decibel scale (dBL) for comparison with consent condition limits. Both the geophone recorder and that used for the accelerometers use a GPS time signal, giving accuracy of better than 1 millisecond.

4.2 Analysis

A coordinate rotation on the values from the geophone was undertaken so that the axes were parallel to the building axes on which the accelerometers were oriented. The “north” building axis was measured as 42.8° east of true north, to which the geophone ‘L’ axis was oriented, and “building” ‘L’ axis and ‘T’ axis geophone values were produced by axis rotation formulae.

Spectrograms were produced for each channel of the ground monitor waveforms.

Except for comparison with a single set-up on the chimney of Building ‘C’, only the horizontal axis recordings were used for comparison. Whilst this is a simplification, as vertical ground motion can induce horizontal building reaction, little would be gained by the extra complication in attempting to understand three dimensional building motion.

5 BUILDING BEHAVIOUR

5.1 Natural frequencies

5.1.1 ASSESSMENT

Methods

Two methods can be used for assessing the natural frequencies of the buildings: structural modelling and analysis of the buildings vibrations. There are also formulae in such documents as the Australian Earthquake Actions Code (AS1170.4—2007) but these are of limited use for unreinforced masonry buildings which lie outside the scope of buildings for which the formulae were derived.

Structural modelling

As noted in section 2.3, Buildings 'A' and 'B' have been modelled as part of previous assessments of the buildings, with both buildings having fundamental modes of vibration of about 15 Hz. Whilst this work gave an approximate value for the resonant vibration frequencies, and was useful for checking those measured, assumptions made to make the modelling possible are likely to have introduced some error.

Measurements

As pointed out in the ACARP report¹, “racking” or swaying of buildings leads to the most damaging in-plane wall movements. To measure this motion it is best to place sensors at the tops of walls at corners so that the measurements are not complicated by wall flexure. This was the procedure followed for all buildings, but some wall-flexure movement was also monitored as the structural modelling had shown that these modes were relevant for Buildings 'A' and 'B' at similar frequencies.

Whilst other frequencies can be seen in the building for the blast events, these are mainly a direct reflection of the blast vibration and amplification factors are quite low. It is not uncommon to observe velocity amplification factors of 5 to 10 times for a building where resonance effects are significant.

It is sometimes possible to measure building frequencies when the building is activated by climatic forces, such as wind; long recordings without blast activation can also give useful results when the vibration record is subjected to Fourier analysis as a whole, or with a much longer window than is used for the spectrograms used for blast wave analysis.

As an illustration of the process adopted, the complete Fourier analyses for the wind gust acting on buildings 'B' and 'C' (as shown in figure 11) is shown in figures 15 and 16.

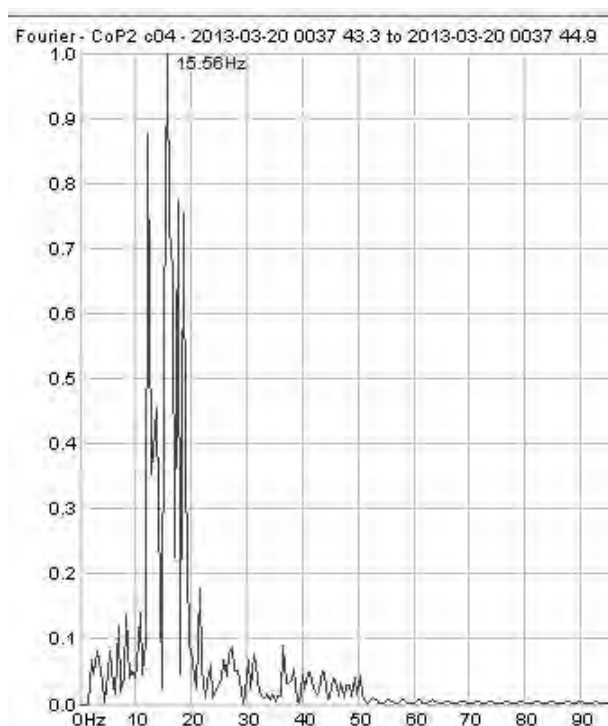


Figure 15: Site north-south Fourier analysis of Building 'B' from wind gust showing fundamental frequency of slightly more than 15 Hz. The earlier analysis showed a corresponding mode at 15 Hz.

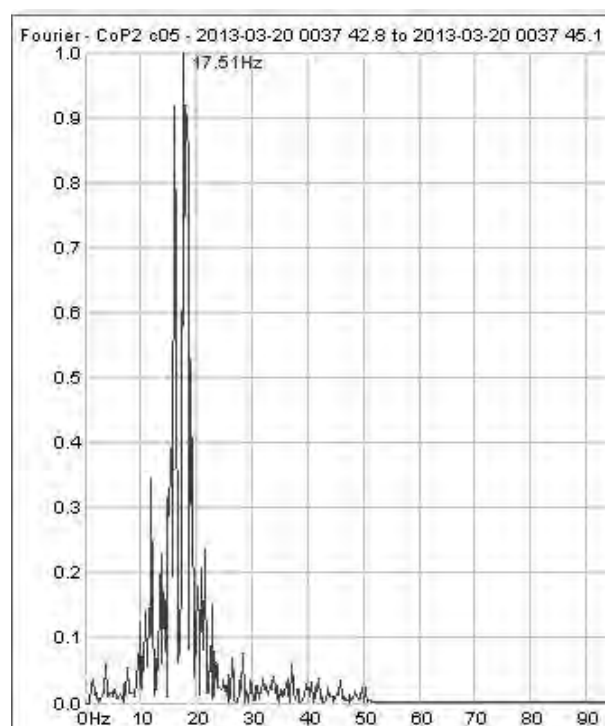


Figure 16: The east-west motion analysis for the same event gives a slightly higher fundamental frequency. A similar analysis of Building 'A' shows a fundamental frequency of 14.6 Hz which compares with the earlier FEA analysis value of 16.1 Hz

5.1.2 RESULTS

As a whole, the buildings show natural frequencies of approximately 12 Hz to 17 Hz. This is the frequency range which, if present in the ground wave, has the greatest potential to cause damage. The chimneys on Buildings 'A' and 'C' have natural frequencies in the range of 6 Hz to 10 Hz.

¹ Structure Response to Blast Vibration, Report C9040, Australian Coal Association Research Program, November 2002

Comparisons of the recorded and derived data for each blast event for representative events and accelerometers are shown in Appendix B.

5.2 Amplification and resonance effects

Velocity amplification, calculated by dividing the maximum recorded velocity in the building by the maximum recorded velocity in the ground wave, along parallel axes, is a common measure of building response². The amplification factor is a measure of the building or building element resonating with the ground wave: a maximum theoretical factor of 50 is possible, but factors greater than 20 are rarely found in practice with factors of less than 5 being most common.

From the analysis done for the site, a maximum amplification factor of 12.9 was found on the Building 'C' chimney for a blast event with PPV of 1.4 mm/s. Extrapolation of the displacement of the chimney for this event shows it to be quite vulnerable for PPV levels much lower than other parts of buildings on the site, indicating that either the critical resonating frequencies will need to be avoided by a considerable margin, the chimney will have to be restrained or a combination of both measures will need implementing.

5.3 Air blast effects

Air blast is measured by the microphone in pascals (N/m²). The linear decibel scale is related to the pressure by a logarithmic function:

$$dBL = 20 \log_{10} \left(\frac{p_{rms}}{20 \times 10^{-6}} \right)$$

where 'p' is in pascals, as recorded by the microphone; the quantity 20×10^{-6} is the pressure designated as the threshold of human hearing, 20 μ Pa. The dBL scale is not to be confused with the dBA scale which is weighted in accordance with the frequency sensitivity of human hearing.

A comparison of some relevant values is given in the following table:

dBL	Pa
100	2
120	20
130	63
135	112
140	200
142	250
150	632

Pressure values can also be compared with wind pressure values for which a building should be designed in accordance with Australian Standards, AS/NZS 1170.2 being the applicable standard for wind actions. For buildings such as those concerned and on that site, a serviceability design wind speed of 37 m/s is applicable. If the building were being designed as new, this is the lowest wind speed which would be considered for any of the various design criteria (ultimate, or Limit State, design wind speed is 45 m/s).

For walls there are different pressure coefficients applicable, depending on which way the wind is blowing, and these pressure coefficients can be either positive or negative: the highest pressure coefficient ($C_{p,e}$) applicable to this building would be 0.8 leading to a serviceability limit state pressure of 0.66 kPa (660 Pa or 150.4 dBL); the lowest $C_{p,e}$ would be -0.3, giving a suction pressure of 0.25 kPa (250 Pa). The pressure from even the lowest applicable design wind is therefore many times that likely to be generated by even the largest blast. For limit state design, a state which the building as a whole has to survive, the applicable pressure is 1.2 kPa (1200 Pa or 156 dBL).

As for ground motion, there is a frequency effect and wind pressures are applicable at much lower frequencies (< 1 Hz) than sounds from of air blast. However, only glass in buildings is normally considered vulnerable to sound pressures and none is exposed at the site at present.

² ACARP, *ibid*.

From the data gathered to date, there is no reason to believe that the building could be liable to damage at air blast levels below 150 dBL, a pressure which is still less than wind pressures likely to have been experienced by the building in the past: other blasting effects are likely to govern before this pressure is reached.

In this discussion it should be emphasised that the current consent condition air blast limits have been set primarily in terms of human perception and have little or no relevance to a property such as Chain of Ponds.

6 DISCUSSION OF FINDINGS

6.1 Scaling of results

In order to determine safe blasting limits, one method is to determine whether the building reaction will be in proportion to the ground wave vibrations as the vibration magnitudes increase. Many variables are involved, not just the ground wave magnitude, but direction, frequency content and blast duration, so an attempt has been made to determine a relationship using locations with data from four different blasts: the locations 'Aa' and 'Ac' have data from two blasts on each on 13/3/2013 and 22/3/2013. For each set of data, the velocity amplitude amplification factor was computed for both the PPV and the parallel component of the ground wave.

A rough trend was observed indicating that the velocity amplification ratio increased slightly at higher ground wave magnitudes. Continuing monitoring as blasting gets closer may resolve this relationship, but the data does suggest that a precautionary approach should be used in the initial design, with the likelihood of refinement to higher allowable levels as blasting gets closer and more data is obtained.

Comparison with the ACARP report data³ indicates that the scaling may be represented by a step function, but with a trend opposite to what was observed in this project. To quote from that report:

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

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

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.

No attempt seems to have been made in the ACARP study to assess the contribution of the blast design characteristics, and the form of the resulting ground wave, to the evaluation of this factor. Further data collected from this project may help resolve this difficulty.

6.2 Assessment of acceptable vibration levels

6.2.1 BASIS

Strain in building fabric is the parameter which best measures damage. Various building materials have different tolerances to strain with metals being able to tolerate large strains without damage and brittle materials (e.g. masonry and, more particularly render or plaster surfaces on the masonry) being able to tolerate much smaller strains before damage occurs.

The "general principles" section of the Structural Design Actions code, AS/NZS 1170.0:2002, tabulates suggested serviceability limit state criteria⁴ and gives a value of Height/600 for in-plane deflection at the top of a masonry wall under wind and earthquake actions: this value is a good starting reference for blast vibrations.

6.2.2 ASSESSMENT

Preliminary calculations indicate that PPV levels of up to 50 mm/s, perhaps more, will not incur damage from whole-of-building swaying motions.

For Building 'A', the applicable height is approximately 7 metres, giving a corresponding deflection limit of 12 mm. For the first blast event on 14 March 2013 (see Appendix B), a maximum displacement at the top of the wall of 0.27

³ ACARP C9040 Section 7.2.1

⁴ AS/NZS 1170.0:2002, Amendment No. 3 Table C1

mm was recorded for a PPV of 3.7 mm/s. If unity scaling were applied, at a PPV of 50 mm/s the displacement at the top of the wall would be $50/3.7 \times 0.27 = 3.6$ mm (< 12 mm). Scaling factors will be progressively assessed as PPVs increase, but this calculation indicates that the assessment is conservative.

Similar calculation indicate that walls not adequately restrained from out-of-plane movements could be affected by PPV levels as low as 20 mm/s.

Similarly, chimneys could be affected by PPV levels as low as 12 mm/s.

It should be noted that there are many unresolved and uncertain relationships between ground vibration characteristics and levels and building behaviour, due to the complex nature of the building structure which does not readily conform to the simple single-degree-of-freedom models used for many of the recommendations in various industry reports. Scaling factors are not readily determined, but more data is likely to give greater prediction confidence.

Continual monitoring of the buildings as blasting gets closer will be of most value in ensuring the best conservation outcome.

6.3 Comparison with blast vibration standards

Some of the blast vibration limit standards used elsewhere in the world are reproduced in the informative (i.e. not formally part of the Standard) Appendix J of AS 2187.2—2006, “Explosives—Storage and use, Part 2: Use of explosives”. Two frequency-dependent criteria, those from the British and USA standards, BS 7385-1 : 1990 and USBM RI 8507, are represented graphically. At, for example, 20 Hz, the British standard suggests a maximum PPV of 25 mm/s and the USBM standard 12.7 mm/s for similar sensitive structures. For different types of building, the ACARP study quoted above suggests a PPV limit of 100 mm/s. The lack of clear guidelines can be resolved by specific measurements.

6.4 Blast design aims

To control the extent of damage at Chain of Ponds, in conjunction with other blast design measures it would be desirable to avoid ground wave frequencies in the 12 Hz to 17 Hz range. Higher frequencies, around 30 Hz and above, are likely to excite other motions in individual parts of the building, such as areas of loose plaster. At current blast distances these frequencies are attenuated; as blasting comes closer the ground wave frequencies need to be monitored continually and the rise of significant ground movement at higher frequencies can be used as one of the “trigger” points (see below).

Whilst avoidance of frequencies at which the chimneys are excited would be desirable, it may not be practicable and support of the chimneys is considered a better option.

Ideally, and using a small frequency buffer, blast design would aim to produce ground motion frequencies below 11 Hz and above 18 Hz for damage to be minimised.

For the first blast monitored in the investigation, that at 13:21 on 14/3/2013, the Old New England Highway was closed and considerable fly rock was seen projecting towards the closed road. Fly rock has the potential to do more damage to the Chain of Ponds buildings than even very large ground vibrations. This is a limiting factor which cannot be covered in this report, but should be considered in blast design.

7 MITIGATION MEASURES

7.1 Further data gathering

7.1.1 FURTHER MONITORING

As noted in section 6.2, scaling of the results has proven inconclusive and published information is of little help, so a requirement for continuing monitoring as blasting gets closer is indicated. For each of the buildings monitoring should be carried out with increasing PPVs to help build up a more complete understanding of structural behaviour.

Whilst it would be possible to install permanent monitors, this would have the following drawbacks:

- permanent attachment of accelerometers to the heritage fabric may not be possible without damage, in all locations;
- individual monitoring visits during significant blast events would allow assessment of the fabric for damage at the time;
- the optimum locations of accelerometers may change as further data is gathered.

The existing monitoring equipment will record from six channels, and whilst it could be extended to 12 channels, cable lengths may become too long for reliability if the installation was spread out for all buildings at one blast event.

I propose, therefore, that at least two blast events be monitored at each predicted PPV trigger level. The proposed PPV trigger levels, which may change as data is obtained, are:

- 10 mm/s with and without frequency control;
- 15 mm/s with and without frequency control;
- 25 mm/s;
- 40 mm/s.

7.1.2 OTHER “TRIGGER” POINTS

The dilapidation survey carried out by EJE Heritage in March 2013 forms a useful basis for determining whether the blasting is having any affect on the buildings and will allow detection of non-structural and insignificant damage such as the dislodgement of loose plaster.

When monitoring is taking place as scheduled above, the dilapidation photographs should be compared with the existing conditions. In particular, pieces of wall or ceiling plaster found on the floor can readily show minor changes: if such are found, it will then be necessary to determine whether vibration or environmental effects are responsible.

Even if a monitoring exercise is not scheduled, it would be prudent to carry out a comparison after any large PPV exceedance or after a very strong wind event is experienced.

Not all subtle indications may be observed during the inspections after an event and update of the dilapidation report could be commissioned after the first 40 mm/s PPV events to provide a formal record using the same basis..

7.2 Design and installation of additional structural support

7.2.1 BASIS

Support of any badly termite-damaged timber structures, such as the front verandah, will be necessary under any scenario as they are liable to further damage from many causes, blasting-related or not. Chimneys should all be supported laterally over most of their heights and, in the case of the one in Building ‘C’, vertically. Chimney support will also remove low frequency control (< 10 Hz) from design requirements.

Some walls may need lateral support, by external props or internal ties, together with soldier and wale members.

7.2.2 DESIGN

The design of the support can be done progressively as further data is obtained, as mining gets closer and is producing larger ground vibration magnitudes. From the results to date, the triggers for design and installation can be based on predicted PPV levels as follows, and with suggestions of methods to be used, subject to continuing assessment:

12 mm/s

- support front verandah with scaffold falsework, all attachment to be with straps or ties, not fasteners;
- wrap chimney of Building ‘C’ (stables) in pallet-wrapping plastic and timber cribbing and strapping; support chimney with external counter-weighted scaffold tower with cantilevered support for chimney; install internal falsework beneath base of chimney.

25 mm/s

- support chimneys on Building 'A' — this may be possible using pre-tensioned internal ties, rather than external scaffolding, after further investigation and detailed design.

50 mm/s

- measures for this level tentative only, subject to further monitoring and design;
- support any vulnerable walls with propped or tied timbers.

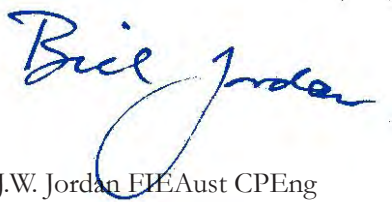
8 CONCLUSIONS AND RECOMMENDATIONS

Information available to date shows that the three buildings comprising the former Chain of Ponds Inn are unlikely to suffer significant damage from ground vibrations providing:

1. frequency control is implemented in association with ground wave magnitude control for blast design, with the aim of avoiding ground wave frequencies in the range of 11 Hz to 18 Hz with PPV levels up to 50 mm/s;
2. the buildings' behaviour to ground vibrations is progressively monitored as blasting comes closer, to allow adjustment to blast design;
3. certain elements of the buildings are secured to prevent damaging movements, subject to future structural design.

A programme of continuing monitoring of the buildings' behaviour when acted on by ground vibrations will allow "fine tuning" of the mitigation measures as the work proceeds.

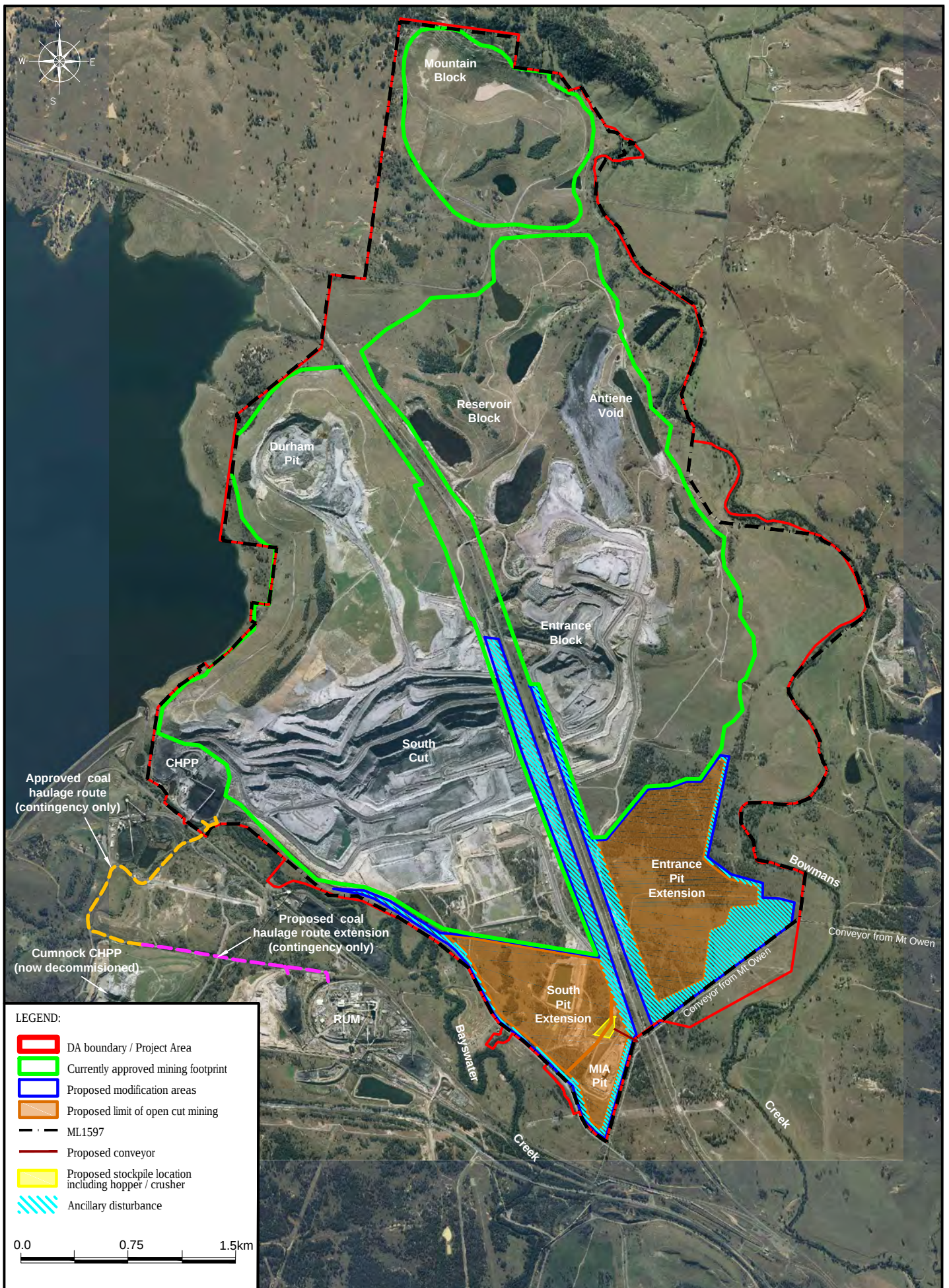
for and on behalf of Bill Jordan & Associates Pty Ltd



J.W. Jordan FIEAust CPEng

APPENDIX A

GSS Environmental project area drawing



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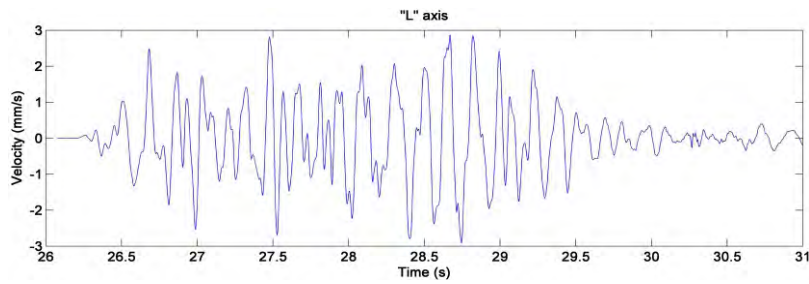
APPENDIX B

Representative vibration comparison illustrations

(refer to Figure 3 for locations)

Similar comparative diagrams are available for all blasts monitored. The ones chosen are for those monitoring positions for which the greatest number of events was recorded and which have been used to assess whether scaling factors can be analysed rationally.

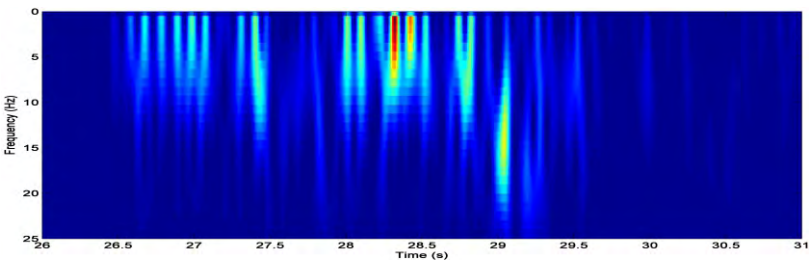
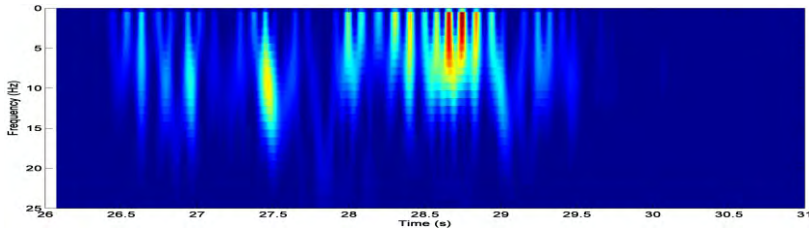
The absence of a clear trend in scaling factors is likely to be a function of the different blast characteristics. Continuing monitoring should help resolve some of the uncertainties.



Waveform & spectrogram of ground wave oriented to C01

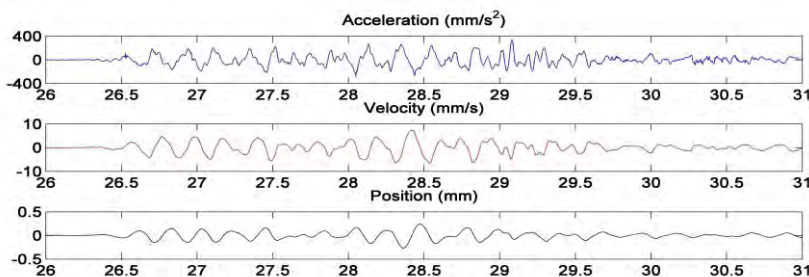
$v_{\max} = 2.87 \text{ mm/s}$

PPV = 3.71 mm/s



Channel C01 spectrogram.

Note the signal at 29 s is not related directly to the blast wave and gives an indication of the building's fundamental vibration mode.



Building acceleration, velocity & displacement waveforms.

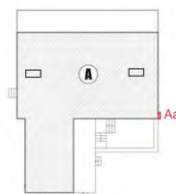
$a_{\max} = 338 \text{ mm/s}^2$

$v_{\max} = 7.34 \text{ mm/s}$

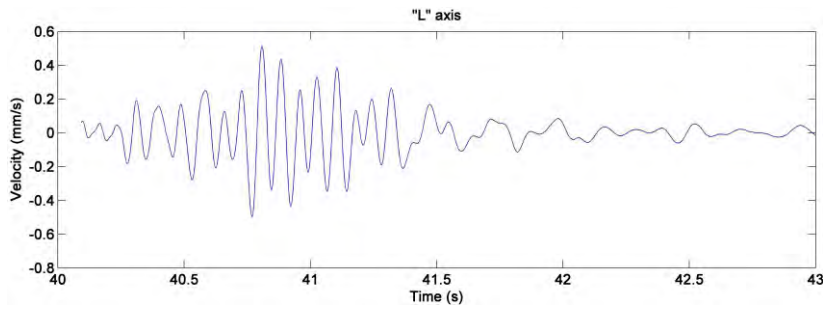
$s_{\max} = 0.27 \text{ mm}$

Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C01	2.87	7.34	2.56
Comments:	For this motion, a ground wave vibration of 50 mm/s unlikely to cause damage (see discussion)		

Ground & Building vibration relationship: position Aa
Axis $N_B - S_B$

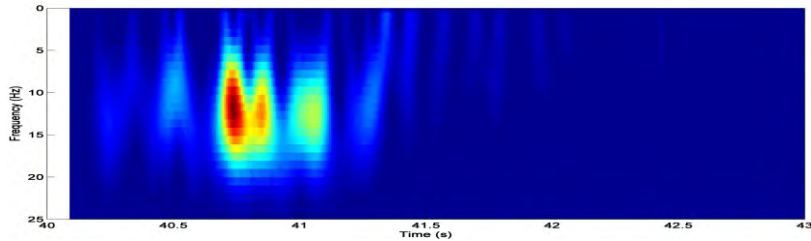


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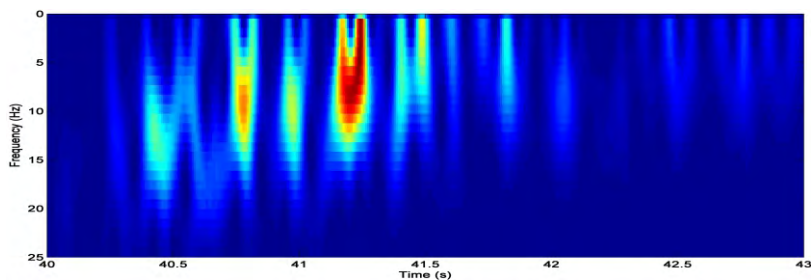


Waveform & spectrogram of ground wave oriented to C01

$v_{\max} = 0.513 \text{ mm/s}$
 $\text{PPV} = 0.541 \text{ mm/s}$

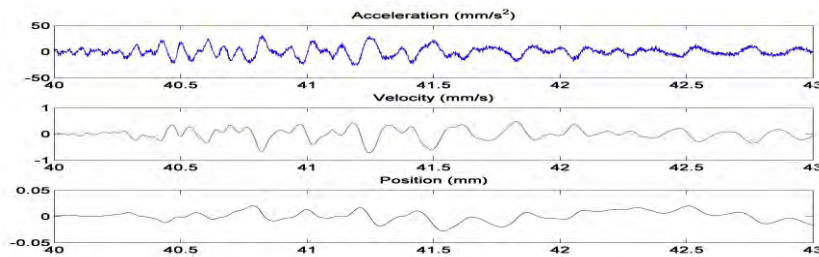


Channel C01 spectrogram



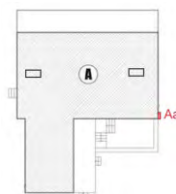
Building acceleration, velocity & displacement waveforms.

$a_{\max} = 32.2 \text{ mm/s}^2$
 $v_{\max} = 0.722 \text{ mm/s}$
 $s_{\max} = 0.028 \text{ mm}$

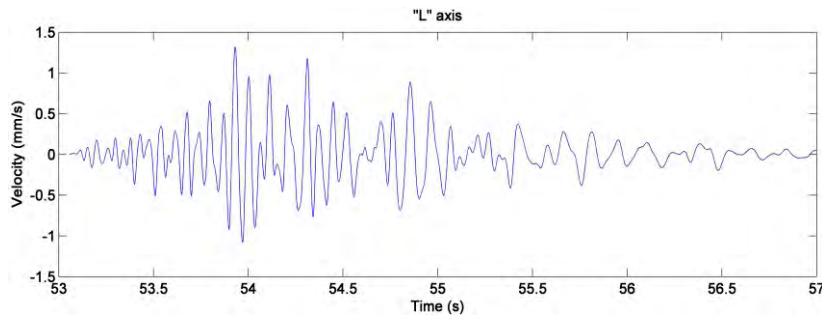


Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C01	0.513	0.722	1.41
Comments:	Comparing with Shot 1, amplification factor scaling is not linear		

Ground & Building vibration relationship: position Aa
 Axis $N_B - S_B$

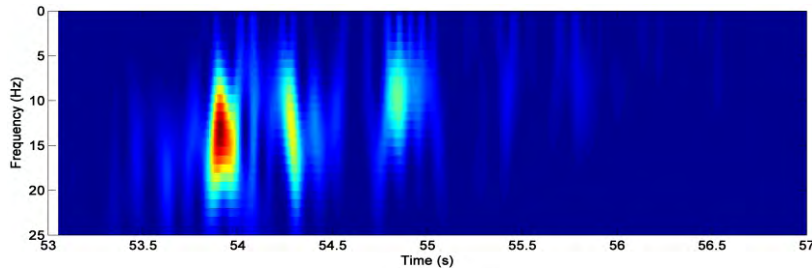


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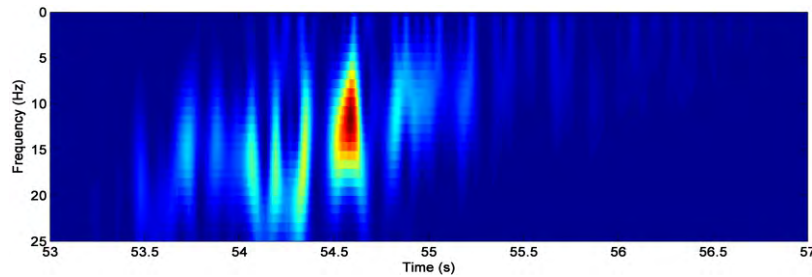


Waveform & spectrogram of ground wave oriented to C04

$v_{\max} = 1.25 \text{ mm/s}$
PPV = 1.91 mm/s

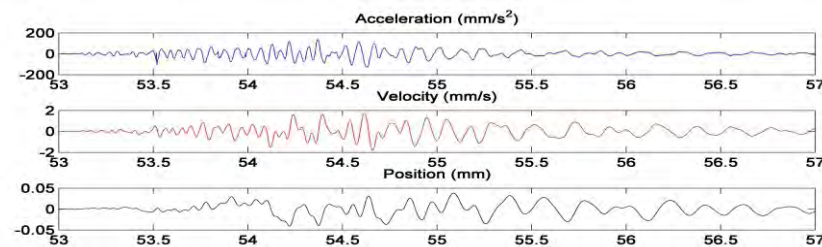


Channel C04 spectrogram



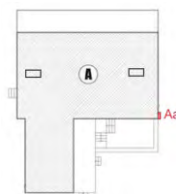
Building acceleration, velocity & displacement waveforms.

$a_{\max} = 137 \text{ mm/s}^2$
 $v_{\max} = 1.79 \text{ mm/s}$
 $s_{\max} = 0.04 \text{ mm}$

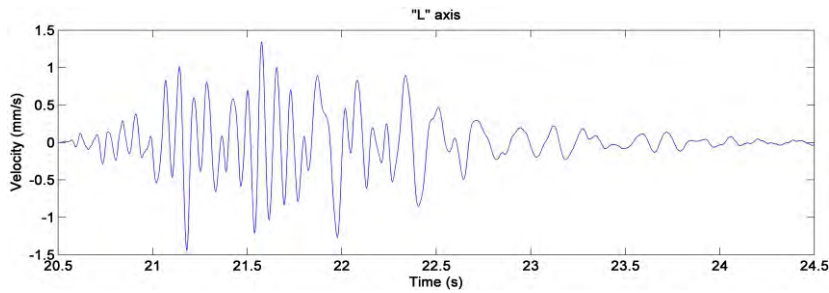


Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C04	1.25	1.79	1.43
Comments:			

Ground & Building vibration relationship: position Aa axis $N_B - S_B$

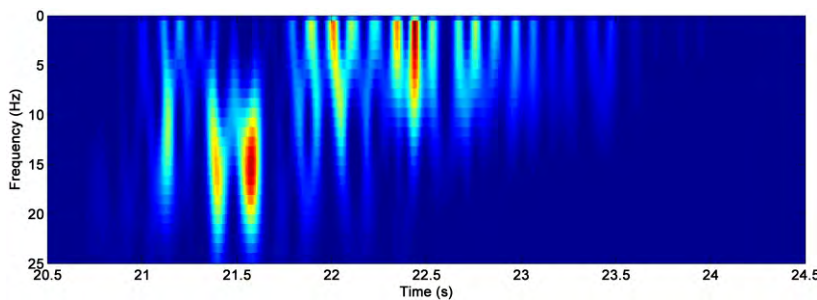
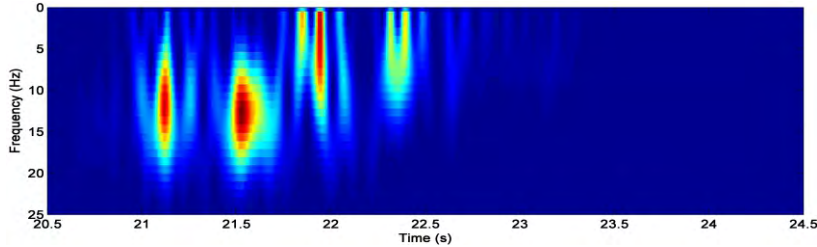


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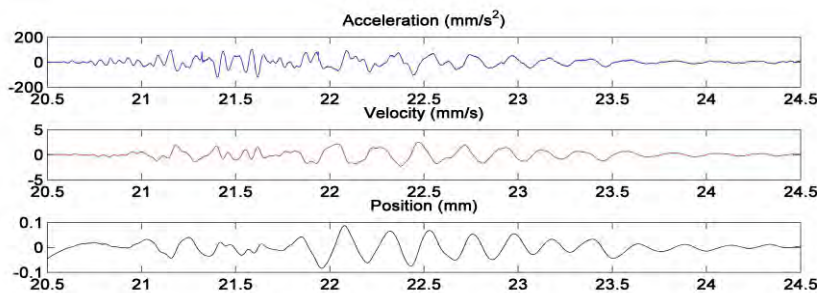


Waveform & spectrogram of ground wave oriented to C04

$v_{\max} = 1.34 \text{ mm/s}$
 $\text{PPV} = 1.45 \text{ mm/s}$



Channel C04 spectrogram

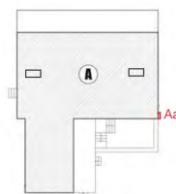


Building acceleration, velocity & displacement waveforms.

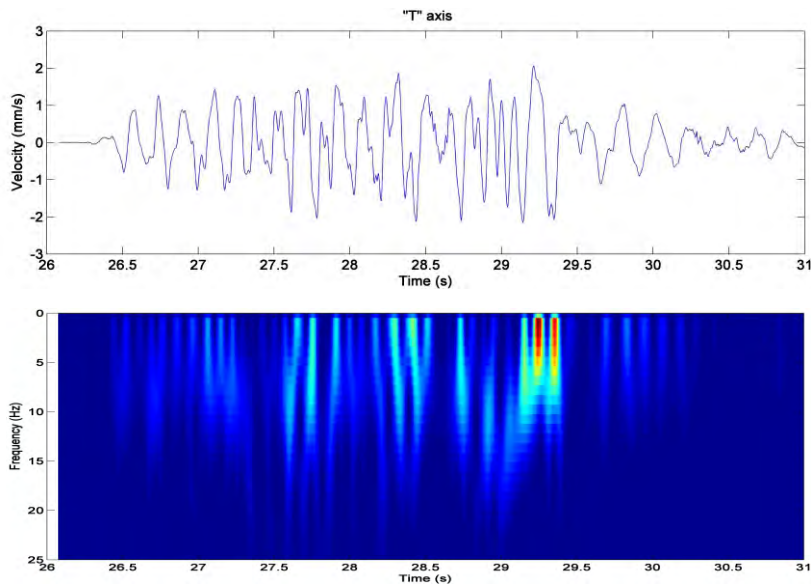
$a_{\max} = 105 \text{ mm/s}^2$
 $v_{\max} = 2.67 \text{ mm/s}$
 $s_{\max} = 0.087 \text{ mm}$

Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C04	1.34	2.67	1.99
Comments:			

Ground & Building vibration relationship: position Aa axis $N_B - S_B$

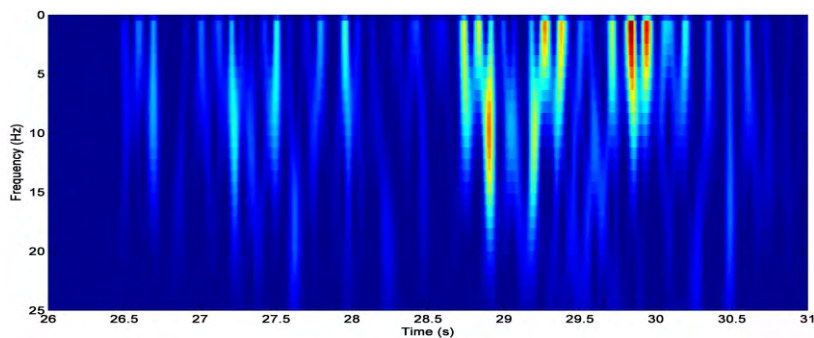


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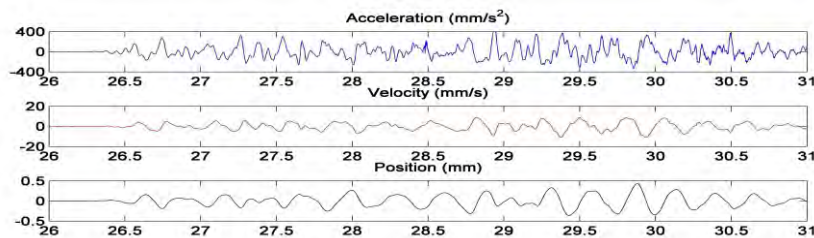


Waveform & spectrogram of ground wave oriented to C03

$v_{\max} = 2.06 \text{ mm/s}$
PPV = 3.71 mm/s



Channel C03 spectrogram



Building acceleration, velocity & displacement waveforms.

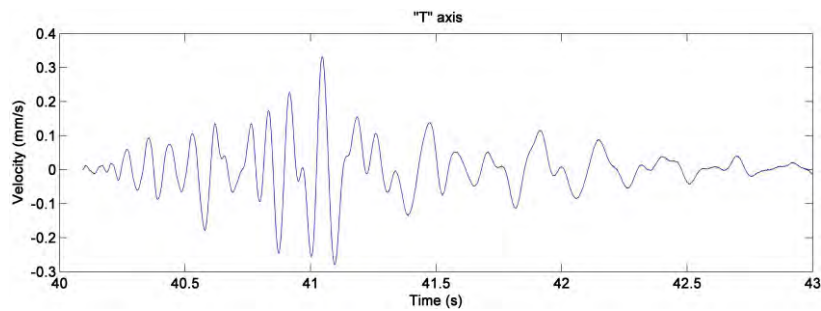
$a_{\max} = 483 \text{ mm/s}^2$
 $v_{\max} = 10.96 \text{ mm/s}$
 $s_{\max} = 0.43 \text{ mm}$

Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C03	2.06	10.96	5.32
Comments:	Centre of walls more vulnerable than corners: a maximum PPV on this axis of 30 mm/s is indicated.		

Ground & Building vibration relationship: position Ac
Axis $E_B - W_B$

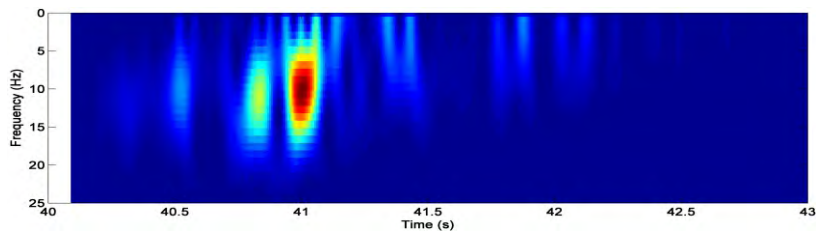


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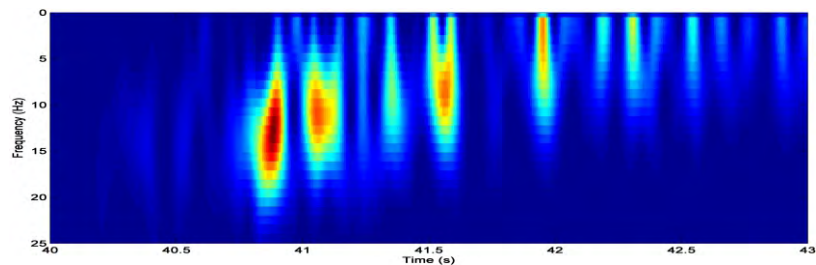


Waveform & spectrogram of ground wave oriented to C03

$v_{\max} = 0.333 \text{ mm/s}$
 $\text{PPV} = 0.541 \text{ mm/s}$

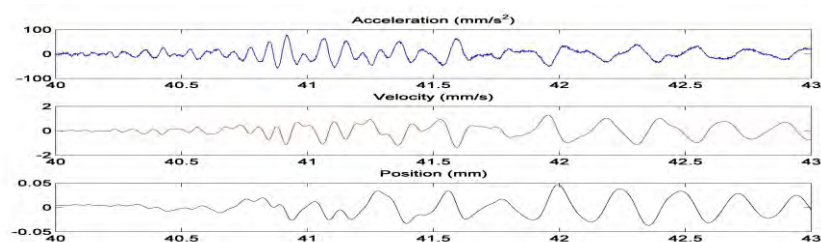


Channel C03 spectrogram



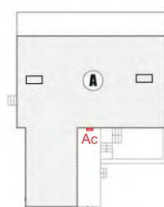
Building acceleration, velocity & displacement waveforms.

$a_{\max} = 80 \text{ mm/s}^2$
 $v_{\max} = 1.38 \text{ mm/s}$
 $s_{\max} = 0.047 \text{ mm}$

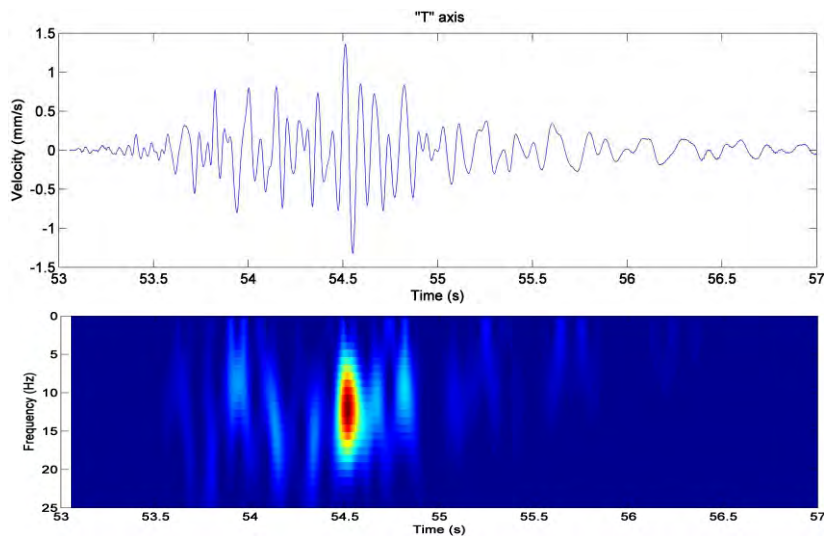


Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C03	0.333	1.38	4.14
Comments:			

Ground & Building vibration relationship: position Ac Axis $E_B - W_B$

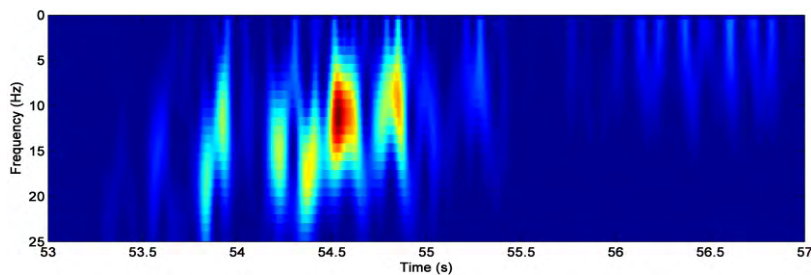


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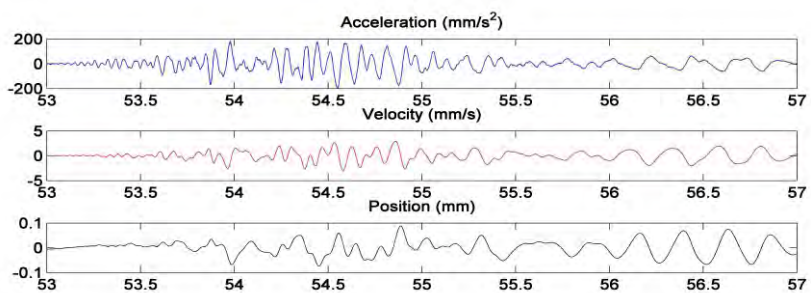


Waveform & spectrogram of ground wave oriented to C06

$v_{\max} = 1.32 \text{ mm/s}$
 $\text{PPV} = 1.91 \text{ mm/s}$



Channel C06 spectrogram

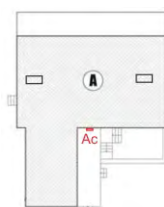


Building acceleration, velocity & displacement waveforms.

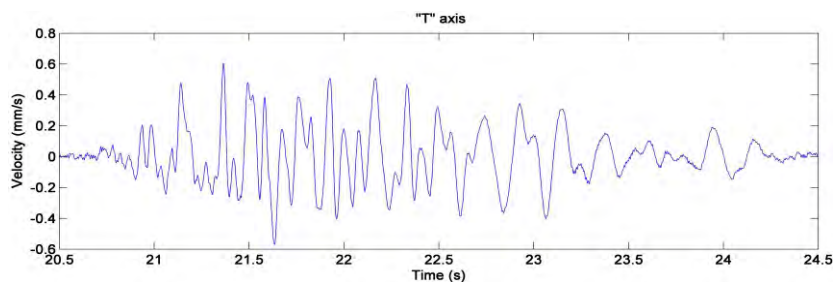
$a_{\max} = 181 \text{ mm/s}^2$
 $v_{\max} = 2.95 \text{ mm/s}$
 $s_{\max} = 0.088 \text{ mm}$

Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C06	1.32	2.95	2.23
Comments:	C06 on rear wall above hallway where motion is greater than that direction as measured at building corner. Amplification at 54 sec is ~ 3.5 due to ground wave being closer to resonant frequency at that time.		

Ground & Building vibration relationship: position Ac Axis $E_B - W_B$

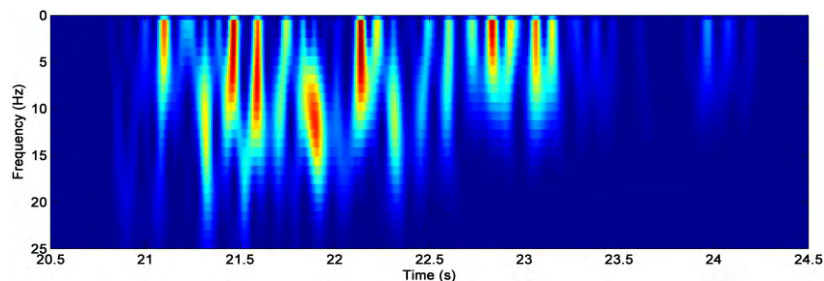


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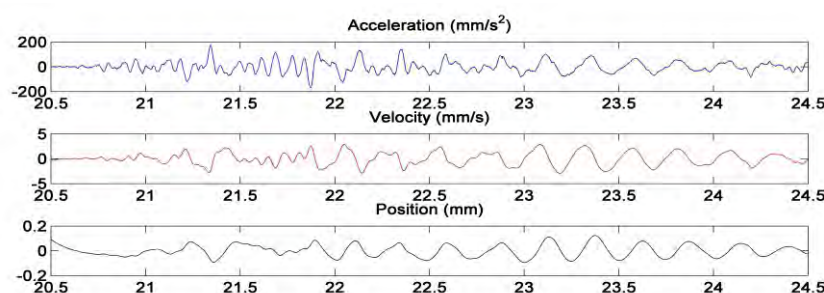
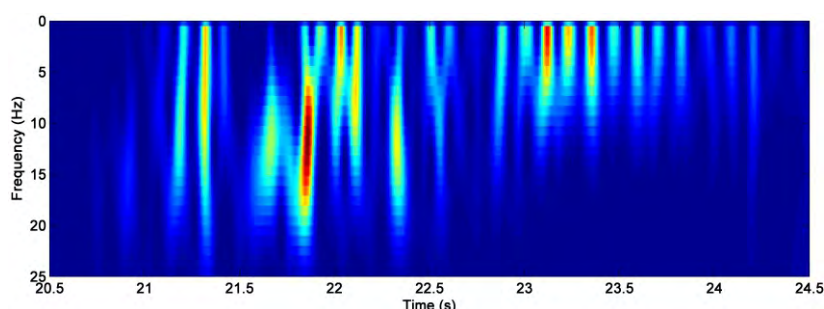


Waveform & spectrogram of ground wave oriented to C06

$v_{\max} = 0.60 \text{ mm/s}$;
PPV = 1.91 mm/s



Channel C06 spectrogram



Building acceleration, velocity & displacement waveforms.

$a_{\max} = 176 \text{ mm/s}^2$
 $v_{\max} = 3.12 \text{ mm/s}$
 $s_{\max} = 0.11 \text{ mm}$

Accelerometer No.	Max ground velocity, mm/s	Max. building velocity, mm/s	Velocity amplification factor
C06	0.60	3.12	5.2
Comments:			

Ground & Building vibration relationship: position Ac axis $E_B - W_B$



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