

1. Introduction

Blasting is common in the coal industry to remove rock overburden so that the exposed coal can be mechanically excavated. A portion of the blast 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. The frequency range of the vibrations is normally in the range of 2-40Hz for a soil site of depth greater than 2m and 10-100Hz for rock sites (Dowding, 1996). The resulting complex time history recordings are often simplified to one value (based on the vector sum) such as ground peak particle acceleration (PPA), velocity (PPV), or displacement (PPD). The PPV is the most common measure for quantifying blast vibrations, as the velocity is approximately correlated to infrastructure and building damage as well as annoyance levels to people.

There are national and international guidelines and standards which specify the maximum allowable blasting limits for structures. For example, Australian Standard AS 2187.2 (1993) recommends a maximum PPV of 10 mm/s for houses and 25 mm/s for commercial and industrial buildings or structures of reinforced concrete or steel construction. These limits consider the dynamic response of the structure to blasting and human comfort levels.

However, unlike structures, there is little guidance related the allowable maximum blast limits and response of infrastructure such as pipelines, transmission towers, electricity substations and telecommunication facilities. Thus, blasting limits vary greatly as they are normally specified by each asset owner/operator. In general, the limits are specified at very low levels and in many cases they are the same as for houses or buildings which are based on human comfort rather than damage threshold. Clearly, using limits for houses and buildings to infrastructure is not applicable due to the fact that the dynamic response is different and human perception is not involved.

As a result, the authors are conducting research to develop rational methods for determining the dynamic response of infrastructure to blasting and to establish the damage threshold of relevant elements. This paper focuses on elevated water pipelines and in particular presents results from field testing.

2. Response of Pipelines

Under blast loading buried pipelines usually deform in a compatible manner with the surrounding ground. The degree to which the pipeline follows the ground deformation depends on the relative flexibility of the pipeline and the surrounding medium. Pipes that are relatively flexible relative to the ground will deform with the ground. For less flexible pipelines, strains in the pipe will be less than the surrounding medium. The blast induced strains on a buried pipe can be summarised as follows (Dowding 1985) and illustrated in Figure 1.

- Axial strains, which occur through compressive / longitudinal wave propagation parallel to the longitudinal axis.
- Bending strains, which occur through shear wave propagation parallel to the longitudinal axis.
- Hoop strains, which occur when compressive or shear waves propagate perpendicularly to the pipe's longitudinal axis and cause changes in the circumferential stress.

For elevated pipelines, the interaction between the ground and the pipeline is further complicated by the presence of the supports and the interaction between the supports and the pipeline. In this case, the supports may filter the ground vibrations and the pipe may exhibit some dynamic response between supports. High local strains may also be experienced at the supports.

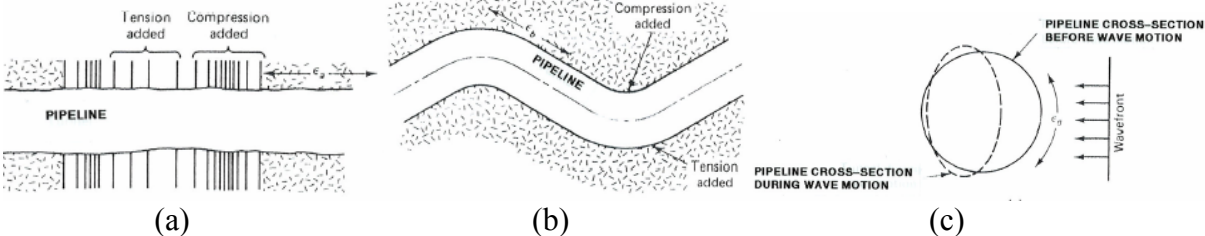


Figure 1: Modes of restrained deformation for buried pipes: (a) axial strain; (b) bending strain; and (c) hoop strain. (after Dowding 1985).

3. Case Study

This paper focuses on a case study representing a typical mild steel-cement lined (MSCL) pipe with flexible lead joints supported by concrete saddles as shown in Figure 2. This mains water pipe is 914mm in diameter with a wall thickness of 8mm and is part of a number of water pipelines which traverse an open cut coal mine. The pipelines occupy premium land that due to the highly stringent blasting requirements imposed by the pipelines operator cannot be mined. However, an agreement has been reached between the operators of the pipelines and coal mine to determine using rational methods safe blasting limits for the pipeline. This agreement is part of an expansion plan of the coal mine which also involves re-routing of the pipelines.



Figure 2: A photo of the pipeline showing the lead joints and concrete supports.

Prior to this study, the blasting limits specified were: (i) blasting is not to be carried out within 100m of the pipelines; and (ii) vibration with a peak particle velocity of 10 mm/s is not to be exceeded on more than 5% of the total number of blasts over a period of 12 months, measured on the pipelines. It was recognised that these limits may be too conservative, and more realistic values needed to be established. Consequently, an analytical investigation along with an experimental field studies were conducted. This paper focuses on the experimental results obtained.

4. Experimental Setup and Blasts

A plan view of the pipe showing the locations of saddles and joints is presented in Figure 1. The pipe was instrumented using 16 accelerometers and 7 displacement transducers. The accelerometers were located on the pipe to measure the vertical and transverse vibration of the pipe at saddle locations as well as at mid-spans between saddles as shown in in Figure 1. It should be noted that accelerometers A, B and C were located on the concrete saddle rather than the pipe. The displacement transducers were primarily used to measure relative movements across the lead joints.

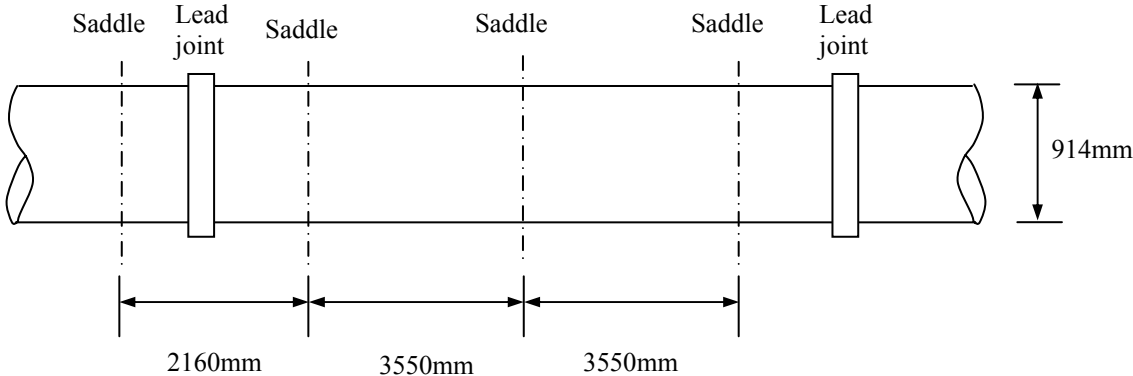


Figure 1: Plan view of a typical pipe segment.

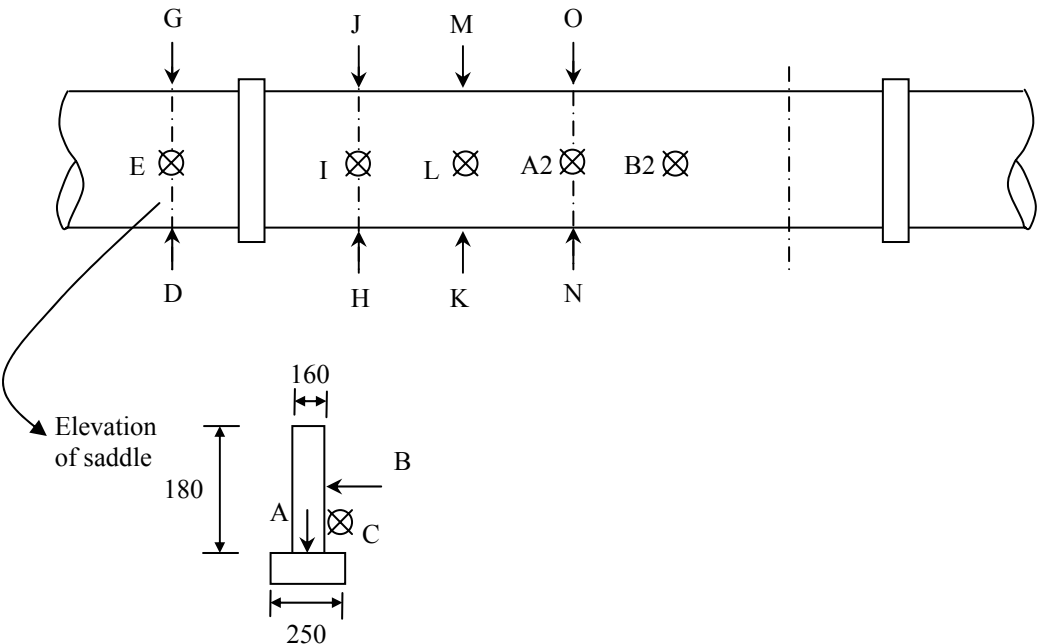


Figure 2: Locations of Accelerometers.

In addition to the accelerometers and displacement transducers, geophones and strain meters were also used, however they are not shown in Figures 2 for clarity.

A series of five single hole blasts were fired with charge masses designed to produce blasts with different levels at different distances from the pipe as outlined below:

- Blast #1: 100m from the pipe producing ground PPV of 10 mm/s
- Blast #2: 70m from the pipe producing ground PPV of 30 mm/s

Blast #3: 50m from the pipe producing ground PPV of 50 mm/s
Blast #4: 50m from the pipe producing ground PPV of 50 mm/s
Blast #5: 30m from the pipe producing ground PPV of 85 mm/s

A final blast (#6) was also conducted which consisted of five duplicate holes as Blasts #1 to #5 with an 800ms delay between them and fired as one blast. This blast produced a ground PPV of 90 mm/s. In addition, the vibration from a distant production blast was recorded. After the blast series was concluded, the water pumps were turned off and the vibration and strain from the resulting water hammer was measured for comparison with the strain due to blasting. The following section provides a summary of the results from these blasts. However, Blasts #1 and #2 are discussed in this paper as they produced very low levels of vibration and response.

5. Results

Blasts #3 and #4 produced very similar accelerations as measured both on the ground and the pipe. The maximum vertical vibration was higher than the longitudinal and transverse directions for both blasts. The maximum acceleration on the pipe recorded from both blasts was 16m/s^2 . This corresponds to a maximum velocity of about 92 mm/s. It should be noted that the pipe seemed to exhibit two distinct natural frequencies at about 15Hz and 35Hz as shown in Figure 3. These two modes responded in the vertical direction at both the saddle and mid-span locations (eg. locations L and A2 on Figure 2).

In the transverse direction, two modes were also been observed, although, the higher mode (about 35Hz) was much more dominant (Figures 4). The resulting velocity in the transverse direction was smaller than in the vertical direction for the same level of acceleration due to the dominance of the higher frequency mode. For Blast #3, the maximum velocity is about 65 mm/s in the transverse direction.

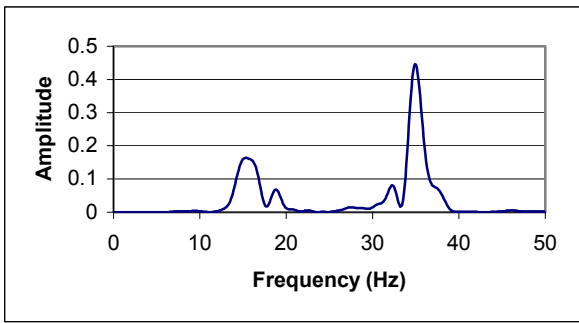


Figure 3: Frequency plot of accelerometer L from Blast #3.

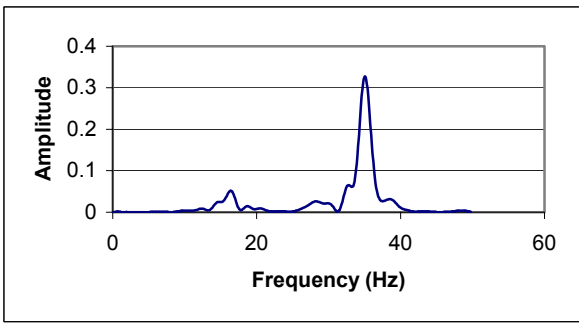


Figure 4: Frequency plot of accelerometer N (transverse at saddle) from Blast #3.

For both Blasts #3 and #4, the maximum measured displacement at the lead joint was very small (0.07 mm).

Blasts #5 and #6 produced similar levels of accelerations. Again, both blasts produced higher accelerations in the vertical direction than in the longitudinal or transverse directions. These two blasts produced similar vibration frequencies to those obtained for Blasts #3 and #4.

For both of these blasts the maximum vertical acceleration recorded was 17.3m/s^2 (at location A2). This corresponds to a vertical velocity of about 92 mm/s. In general, Blast #6 produced higher acceleration response for the majority of locations compared to all other blasts. For both blasts the maximum measured displacement at the lead joints was 0.4 mm.

6. Analysis

6.1 Estimation of maximum pipe response

Based on the data collected from the various blasts, it has been found that an amplification factor of 2 would be considered appropriate in predicting the pipe response for ground PPV greater than 10 mm/s. Hence, for the following analyses, it is assumed that the maximum PPV on the pipe would be 200 mm/s based on a maximum ground vibration of 100 mm/s and an amplification factor of 2. Alternatively, the maximum response of the pipe can be estimated using basic dynamic theory for a single degree of freedom system (SDOF) as follows:

$$\text{Velocity} = \text{Acceleration} / (2*\pi*\text{natural frequency}).$$

Based on the accelerometers measurements, the maximum acceleration measured on the pipe was 17.3m/s^2 . While the pipe is exhibiting two clear frequencies (15Hz and 35Hz) in many locations, it can be conservatively assumed that the pipe is acting as a single degree of freedom system responding at a natural frequency of 15Hz, hence:

$$\text{Maximum Velocity} = 17.3*10^3/(2*\pi*15) = 184 \text{ mm/s}.$$

Similarly, maximum displacement of 2mm can be calculated using a SDOF approximation.

6.2 Estimation of bending strain and stress

A formula for predicting the strain due to bending for a buried pipe with a soil shear wave velocity C_s is proposed by Dowding (1985) as follows:

$$\varepsilon_b = \frac{V_{max} 2\pi f R}{C_s^2}$$

where:

V_{max}	= peak particle velocity due to shear wave
f	= principal frequency
R	= pipe radius to outer fibre
C_s	= propagation velocity of shear wave

In this case, $V_{max} = 200$ mm/s (for the elevated pipe, the velocity on the pipe rather than on the ground has been adopted). Assuming conservative estimates of $f = 15\text{Hz}$, $R = 457\text{mm}$ and $C_s = 500,000$ mm/s, the resulting bending strain (ε_b) would be 35 microstrain.

Given the elastic stress-strain relationship for steel with Young's modulus of 200GPa, the maximum stress can be estimated (assuming that the pipe deforms with the ground) to be 7MPa.

The bending strain can be estimated using another method, based on the measured response. The pipe response can be approximated to have a sinusoidal deflection between the lead joints (points of discontinuity) over a distance of 9m as illustrated in Figure 5.

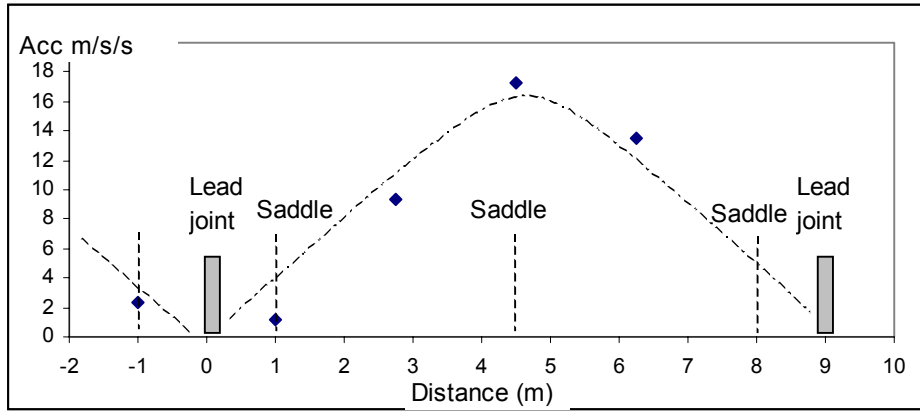


Figure 5: Snap shot of profile of vertical acceleration corresponding to measured peak response.

For a beam deflection profile as shown in Figure 6;

$$\text{Max. curvature} = \phi = y'' = \Delta_{max} (2\pi/4L)^2$$

$$\text{Bending strain } \epsilon_b = \Delta_{max} (2\pi/4L)^2 * R$$

$$\epsilon_b = \Delta_{max} \pi^2 / (4L^2 * R)$$

where L is half the span.

Hence, if

$$L = 4.5\text{m}, \Delta_{max} = 2\text{mm} \text{ \& } R = 457\text{mm}$$

then, $\epsilon_b = 112$ microstrain and

$$\text{bending stress} = 200 * 10^3 * 112 * 10^{-6} = 22\text{MPa}.$$

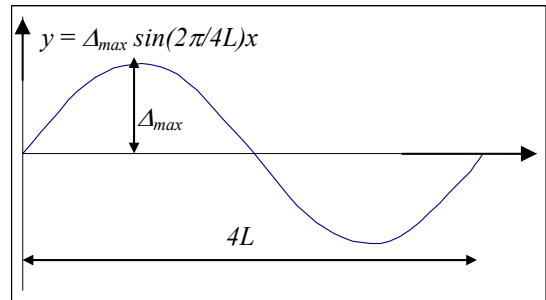


Figure 6: Sinusoidal beam deflection

The resulting bending stress using this approximation (22MPa) is higher than that produced by Dowding's prediction (7MPa), but it is of the same order of magnitude and is less than 10% of the yield strength of the steel pipe.

6.3 Estimation of axial strain and stress

A formula for predicting the axial strain is proposed by Dowding (1985) as follows:

$$\epsilon_A = \frac{V_{max}}{C_1}$$

where V_{max} = peak particle velocity due to compression wave
 C_1 = propagation velocity of shear wave

Based on the acceleration measurements on the saddle, the longitudinal acceleration was significantly smaller than both vertical and transverse directions. This is expected for an elevated pipe as it is able to move relative to the saddles and hence reduce the stress. However, to be conservative, the amplified PPV of 200 mm/s is used herein for predicting the axial strain for the elevated pipe.

Thus, if $V_{max} = 200$ mm/s and $C_1 = 2,000,000$ mm/s, the resulting axial strain would be $\epsilon_A = 100$ microstrain with a corresponding axial stress of 20 MPa.

For an elevated pipe, it is not expected that the pipe will develop hoop strains due to blasting since the pipe is not in direct contact with the ground and therefore not exposed to ground deformations that would cause ovaling of the cross section.

7. Discussion

For all the blasts there was no sign of leaks or permanent deformation to the pipe. Further, the direct measured strains on the pipe were relatively small, with the maximum value being about 20 microstrain corresponding to a stress of 4 MPa.

Clearly, blast induced stresses as estimated in Section 6, would be additional to the water pressure induced hoop and axial stresses on the pipe. Further, the concrete saddles introduce additional bending and localised stresses. Based on the location of the pipe, the geometry and configurations of the saddles, it is estimated that the operating conditions and saddles impose a maximum hoop stress of about 80MPa and a maximum axial stress in the pipe at the saddle supports of 100MPa.

The estimated maximum allowable stress on the pipe would be 150MPa based on grade 250 steel with a yield strength of 250MPa and ultimate strength of 400MPa and considering a conservative industry based safety factor of 0.6. Clearly, even for ground vibration of 100 mm/s or pipe vibration of 200 mm/s the additional stresses induced by the blasts would be significantly smaller than the maximum operating stresses. Further, the additive effects of all stresses and strains would be well below the maximum allowable limits for the pipe, joints and lining.

The results from this study are consistent with observations made from reconnaissance studies after major earthquake events. Based on such studies, it is recognised that buried pipelines are not normally damaged by earthquakes until vibrations reach a Mercalli Intensity of IX, which is equivalent to a PPV of around 360 mm/s. Earthquake vibrations are generally similar in form to blast vibrations, however, they are more demanding due to the larger ground displacements and significantly longer duration (typically blast ground vibrations do not exceed 5 second duration).

8. Conclusions

This paper briefly described the effects of mine induced blast vibrations on elevated pipelines. Due to lack of standards and data, there is little guidance for blast engineers and infrastructure operators on the safe blasting vibration limits for pipelines.

As part of the case study presented in this paper, trial blasts with different magnitudes were used to determine the actual response of the pipe. Using accelerometers, geophones, strain meters and displacement transducers, the various modes of pipe response were measured.

Based on the presented field measurements and analytical predictions it has been found that the pipe could easily sustain a maximum ground PPV of 100 mm/s or maximum PPV on the pipe of 200 mm/s. These limits are an order of magnitude higher than the current limits (typically 5-20 mm/s) imposed by the operators and owners of pipeline infrastructure.

9. References

1. Australian Standard AS2817.2, (1993) "AS 2187.2 Explosives - Storage, transport and use. Part 2: Use of explosives", Standards Australia, Sydney, Australia.
2. Dowding, C. H. (1985). "Blast Vibration Monitoring and Control", Prentice-Hall International Inc., USA.
3. Dowding, H. C. (1996). "Construction Vibrations", Prentice-Hall, USA, 1996.