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9.2.2 MONITORING AND CONTROL OF BLAST EFFECTS

CHARLES H. DOWDING

9.2.2.1 Introduction

Monitoring and control of blast effects near critical rock masses or constructed facilities depend upon two main considerations. First, shot designs must reduce the amount of explosives detonated at any instant and adjust the initiation sequence to reduce resulting ground and airborne disturbances. Second, the amount of explosives detonated per volume of rock and the shot pattern must be adjusted to ensure adequate fragmentation. Therefore, at the same time, the initiation sequence must be separated in time but not in space.

There is an optimum design which achieves both objectives of control of disturbances and production of adequate fragmentation. This optimum can be reached only through an understanding of the physics of rock mass and structural response to blast disturbance and the interaction between rock fragmentation and shot design. This segment summarizes the state of the art in vibration measurement and structural response to facilitate such an optimum blast design.

Furthermore, it is an effort to transfer advances in earthquake engineering and nuclear blast protective design to blast vibration monitoring and control while summarizing the most recent experimental observations of mining-induced ground motions and structural response. It is hoped that such a transfer and summary of the state of the art will mitigate several recent trends.

For one, there has been a general downward trend in regulatory limits on allowable blast-induced vibrations. In addition to new observations, this drift, in part, can be attributed to the tendency to take the limit of the last study and divide it in half "to be safe." Unfortunately, too many studies whose limits were divided were themselves only summaries of past work that had also divided past limits. The discussion here presents the background for original experiments conducted to determine safe blasting controls and therefore will allow the reader to set appropriate limits based upon the original past work within the framework of existing regulations.

Another trend is the misapplication of peak particle velocity limits that were determined for cosmetic cracking of residential structures. These limits have been applied to tunnel liners, radio towers, slabs-on-grade, and curing concrete. This segment draws attention to studies made to determine limits specifically for these and other cases. Where no studies exist, it presents methods based upon response spectra or ground strains that allow setting of appropriate criteria or limits.

Frequency of vibration and ground strain form the foundation for the presentation. The importance of frequency cannot be over estimated, as it is as critical as peak particle velocity in determining the response of aboveground structures. For below ground structures, frequency, in combination with propagation velocity, controls response. In both cases, cracking results from induced strains, where particle velocity is employed as an index of the strain level.

In addition, computerized monitoring instrumentation is described. Such computerization simultaneously increases monitoring efficiency as well as decreases costs, both original capital costs as well as those associated with record keeping. The latter labor-saving efficiency associated with automated record keeping continues to be undervalued by many mining operations.

9.2.2.2 Range of Blast Effects

Blast effects on surrounding earth materials and structures can be divided into the permanent and transient displacements. While the focus of this chapter is the transient displacements, effects of permanent displacements are presented as they are associated with significant transient effects at relatively small distances.

Permanent Degradation and Displacement of Adjacent Rock: Permanent effects, with the exception of fly rock, are only described within a few hundreds of feet (meters), and can be divided into degradation and displacement.

Degradation—Degradation is normally described by cracking intensity. Such blast-induced cracking has been observed experimentally to vary with hole diameter and rock type (Siskind and Fumanti, 1974; Holmberg and Persson, 1978). Small-hole-diameter construction blasting has induced cracking at distances of 3½ to 7 ft (1 to 2 m), and larger-hole-diameter mining blasts are capable of producing cracks at distances of 35 to 70 ft (10 to 20 m). Careful blast design can dramatically reduce these maximum distances.

Displacement—Displacement can be produced by either delayed gas pressures (those that accumulate during detonation) or by vibration-induced shaking. Delayed gas pressure have dislocated blocks as large as 1300 yd^3 (1000 m^3) during construction blasting (Dowding, 1985). Such movement is unusual, but is associated with isolated blocks, leakage of gas pressures along open joints, and poor shot design with large burdens. Vibratory or shaking-induced displacement is normally associated with unstable blocks in rock slopes and can occur wherever static factors of safety are low and ground motions produce permanent displacements that are greater than the first-order asperity wave length (Dowding and Gilbert, 1988). Gas pressure related displacement can occur out to several hundred feet (meters).

Fly Rock—Fly rock is a special case of permanent displacement of rock by explosive expulsion from the top of the blasthole. Rock has been observed to have been propelled as far as 330 to 3300 ft (100 to 1000 m) (Roth, 1979). Statistical studies have shown that the probability of these extreme events is quite low under normal circumstances, 1 in 10,000,000 at 2000 ft (600 m) (Lundborg, 1981). Since the probability increases with decreasing distance, blasting mats are required in any construction blasting in an urban environment to prevent all fly rock.

Soil Densification or Compaction—Another special case of permanent displacement is the vibratory densification of a nearby mass of loose, clean sand. The propensity for such densification is a function of the soil's density, mineralogy, and grain size distribution. Soils that are densifiable are loose sands, with less than 5% silt-size particles. These clean sands were densified out to distances of 70 ft (20 m) (Ivanov, 1967) after detonation of single, 11-lb (5-kg) charges within the loose sand mass itself. Soils that are either slightly cemented or contain more than 5% fines are a great deal less subject to vibratory densification from typical ground motions.

Transient Structural Response: Transient effects result from the vibratory nature of the ground and airborne disturbances that propagate outward from a blast. In this discussion, it is assumed that no permanent displacements are produced. Thus the only effects are those associated with the vibratory response of facilities in or on the rock or soil mass surrounding the blast. Transient means that the peak displacement is only temporary, lasts less than one-hundredth of a second, and the structure returns to its original position afterwards.

Transient structural effects can be arranged to reflect the expected distance from a blast. Beginning with the closest, transient effects are structural distortion, faulted or displaced cracks, falling objects, cosmetic cracking of wall coverings, excessive instrument and machinery response, human response, and micro disturbance.

The first four effects, those that relate to structural response, are normally grouped together for experimental observation as structural response, and do not normally occur when vibration levels are regulated to prevent cosmetic cracking.

Excessive structural response has been separated into three categories arranged below in the order of declining severity and increasing distance of occurrence (Northwood et al., 1963; Siskind et al., 1980b). Beginning with effects that occur closest to the blast, the categories are listed here.

1. MAJOR (Permanent Distortion). Resulting in serious weakening of the structure (e.g., large cracks or shifting of foundations or bearing walls, major settlement resulting in distortion or weakening of the superstructure, walls out of plumb).
2. MINOR (Displaced Cracks). Surficial, not affecting the strength of the structures (e.g., broken windows, loosened or fallen plaster), hairline cracks in masonry.
3. THRESHOLD (Cosmetic Cracking). Opening of old cracks and formation of new plaster cracks, dislodging of loose objects (e.g., loose bricks in chimneys).

These specific definitions of response should not be described collectively as "damage." To do so blurs the distinction between threshold or cosmetic cracking and major response or structural distress.

Regulation to Prevent Cosmetic Cracking of Residential Structures—Regulatory controls in North America are based on the occurrence of threshold cracking of plaster and gypsum wall board in residential structures (Siskind et al., 1980; Dowding, 1985). Observed cracking is cosmetic in nature and does not affect structural stability. These cosmetic cracks are hair-sized and are similar to cracks that occur during the natural aging of structures. In fact they are indistinguishable from those that result from natural aging. Control limits are based upon direct observations of test homes immediately before and immediately after blast events, to avoid confusion with the similar cracks that might occur from natural processes. These controls do not apply to engineered structures that are constructed of steel and concrete, buried structures, or adjacent rock.

Distinction of Blast-Induced Cracking from Natural Cracking: Control of blast-induced transient effects to prevent threshold or cosmetic cracking reduces blast-induced displacement or strains in structures to or below that caused by every day human activities and changes in the weather (Stagg et al., 1984; Dowding, 1988). These cosmetic cracks in many cases are smaller than cracks caused by other natural or occupant initiated processes that are active in all constructed facilities. Thus blast-induced threshold cracks can be scientifically observed only with visual inspection immediately before and after each blast. Observations made under less stringently controlled conditions have little scientific merit because of the high probability of environmentally produced cracks occurring between or before visual inspections.

Multiple Origins of Cracks—Several institutional references (Anon., 1977; Anon., 1956; Thoenen and Windes, 1942) present excellent summaries of the multiple origins of cracks. Basically, cracks are found to be caused by the following:

1. Differential thermal expansion.
2. Structural overloading.
3. Chemical changes in mortar, bricks, plaster, and stucco.
4. Shrinkage and swelling of wood.
5. Fatigue and aging of wall coverings.

Table 9.2.2.1. Comparison of Strain Levels Induced by Household Activities, Daily Environmental Changes, and Blasting

Loading Phenomena	Site[a]	Microstrain Induced by Phenomena, μin./in.	Corresponding Blast Level[b]	
			in./sec	mm/s
Daily environmental	K_1	149	1.2	30.0
changes	K_2	385	3.0	76.0
Household activities				
Walking	S_2	9.1	0.03	0.8
Heel drops	S_2	16.0	0.03	0.8
Jumping	S_2	37.3	0.28	7.1
Door slams	S_1	48.8	0.50	12.7
Pounding nails	$S_{1,2}$	88.7	0.88	22.4

Source: Stagg et al., 1984.
[a] K_1 and K_2 were placed across a tape joint between two sheets of gypsum wallboard.
[b] Blast equivalent based on envelope line of strain vs. ground vibration.

6. Differential foundation settlement.

Overtime, all of the causes listed are likely to crack walls, whether or not blasting occurs.

There are three important implications associated with the list above. Structures expand and contract preferentially along existing weaknesses (cracks). Seasonal expansion and contraction along these cracks will return patching and repainting to the original cracked state within several years. This persistent cracking is annoying to those owners who are unaware of the difficulty of patching existing cracks of any kind. Second, the distortion that caused the cracking also creates stress concentrations which may lower a wall-covering's resistance to vibration cracking; however, current regulatory limits already implicitly include these distortion effects as explained in 9.2.2.6. Third, these natural cracks continue to occur over time. Therefore, any postblast inspection at low vibration levels is likely to find new cracks from natural aging unless preblast inspection is conducted immediately before the blast.

Response of Structures to Everyday Activities: A comparison of strains produced by blast vibrations and everyday events with those needed to fail wall-covering materials gives perspective to the observation of cracking at low particle velocities. Table 9.2.2.1 compares strains from daily environmental changes (temperature and humidity) and household activities measured in the US Bureau of Mines test house (Stagg et al., 1984). The door was slammed adjacent to the wall on which the strains were measured.

It appears that in the course of daily life, an active family will produce strains in walls similar to those produced by blasting vibrations of 0.1 to 0.5 ips (2.5 to 12 mm/s). Most astonishing are the measurements in a wood-framed home of relatively enormous strains from daily changes in temperature and humidity. These alone are large enough to crack plaster.

Blast-induced Air Overpressures: Blast-induced air overpressures are the air pressure waves generated by explosions. The higher-frequency portion of the pressure wave is audible and is the sound that accompanies a blast; the lower-frequency portion is not audible, but excites structures and in turn causes a secondary and audible rattle within a structure.

Overpressure waves are of interest for three reasons. First, the audible portion produces direct noise. Second, the inaudible portion by itself or in combination with ground motion can produce structural motions that in turn produce noise. Third, they may crack windows; however, air-blast pressure alone would have to be unusually high for such cracking. Previous researchers (Kamperman and Nicholson, 1970; Borsky, 1965) have found that response noise within a structure (from blasting and sonic booms respectively) is the source of many complaints. It appears that structure and wall motions, which are induced by airblasts and sonic booms, rattle loose objects within the structure, which then startle the occupants.

Human Response: Humans are quite sensitive to motion and noise that accompany blast-induced ground and airborne disturbances. Therefore, human response is significant in the reporting of blast-induced cracking. Motion and noise from blasting can be startling and lead to a search for some physical manifestation of the startling phenomena. Many times, a previously unnoticed crack provides such confirmation of the event. Furthermore, if a person is worried and observes a crack that was not noticed before, the crack's perceived significance increases over one noticed in the absence of any startling activity. These concerns are real and in the mind of the observer are sincere.

In typical mining situations, significant blast-induced inaudible air overpressure and audible noise immediately follows the ground motion and intensifies human response. Both the ground and airborne disturbances excite walls, rattle dishes, and together tend to produce more noise inside a structure than outside. Thus both the audible noise as well as the wall rattle produced by inaudible pressures contribute to human response. To complicate matters even more, inaudible air overpressures can vibrate walls to produce audible noise at large distances, which are inaccurately reported by occupants as ground motions.

9.2.2.3 Character of Blast Excitation and Structural Response

As shown in Fig. 9.2.2.1, both the ground and airborne disturbances (upper-four time histories) produce structure response (lower-four time histories). Because of the importance of frequency, the full wave form or time history should be recorded. When a critical location is known, blast response is best described by the strain at that location. Alternatively, particle velocity (that shown in Fig. 9.2.2.1) can be measured outside the structure of concern, as many recent cracking studies have correlated cracking with excitation particle velocity measured in the ground.

Ground Motion: *Ground motion* can be described by three mutually perpendicular components labeled L (longitudinal), T (transverse), and V (vertical) (Fig. 9.2.2.1). The L and T directions are oriented in the horizontal plane with L directed along the line between the blast and recording transducer. When a study focuses upon structural response, axes can be labeled H1, H2, and V, with H1 and H2 oriented parallel to the structure's principal axis.

Variation of peak motions in each component (L, V, and T in Fig. 9.2.2.1) has led to difficulty in determining which is more important. Horizontal motions seem to control the horizontal response of walls, and superstructures and vertical motions seem to control the vertical response of floors. In an absolute sense, the peak ground motion is actually the maximum vector sum of the three components, which usually occurs at the largest peak of the three components, the dashed line in Fig. 9.2.2.1. This true maximum vector sum is not the FALSE maximum vector sum calculated with the maxima for each component (dots in Fig. 9.2.2.1) no matter their time of occurrence. The FALSE maximum vector sum may be as much as 40% greater than the

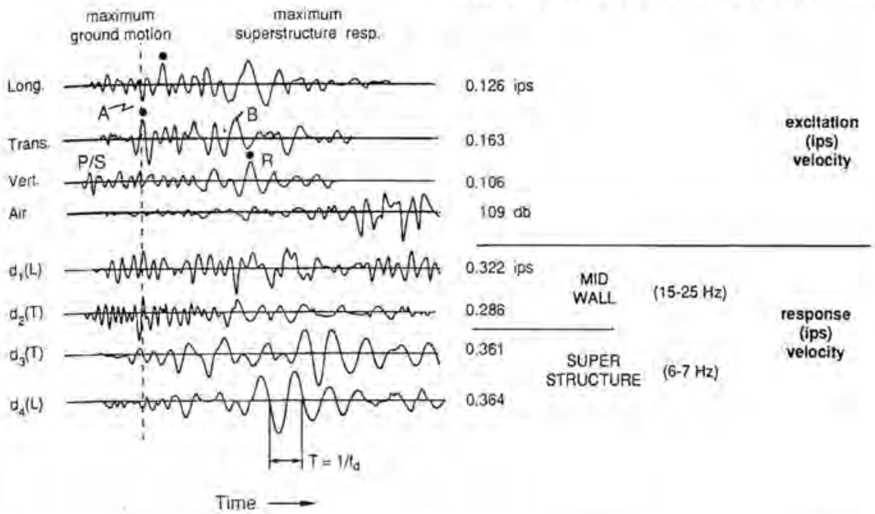


Fig. 9.2.2.1. Comparison of blast excitation by ground and airborne disturbances and residential structure response of walls and superstructure. Measurements were made some 2000 ft (600 m) from a typical surface coal mining blast. (After Dowding, 1988). Conversion factor: 1 ips = 25.4 mm/s.

TRUE maximum vector sum, which is normally 5 to 10% greater than the maximum, single component peak.

In general, experimental observations of threshold or cosmetic cracking, which form the basis of blasting controls in North America, have been correlated with the maximum single component regardless of direction. Therefore, use of the FALSE maximum vector sum, for control, provides a large, unaccounted for, factor of safety.

Two wave types are produced by blasting, *body* and *surface,* and are illustrated by the ground motion in Fig. 9.2.2.1 measured some 2000 ft (600 m) from a typical surface coal mining blast. Body waves travel through earth materials, whereas surface waves travel along surfaces and interfaces of earth materials. The most important surface wave is the Rayleigh, denoted R on the vertical trace in Fig. 9.2.2.1. Body waves can be further subdivided into compressive (compression/tension) or soundlike waves, and distortional or shear waves, denoted as P/S on the vertical trace in Fig. 9.2.2.1. Explosions produce predominantly body waves at small distances. These body waves propagate outward in a spherical manner until they intersect a boundary such as another rock layer, soil, or the ground surface. At this intersection, shear and surface waves are produced. Rayleigh surface waves become important at larger transmission distances as illustrated in the vertical trace by the relatively larger "R" amplitude compared to the "P/S" amplitude.

Sinusoidal Approximation—Typical blast vibrations, no matter the wave type, can be approximated as sinusoidally varying in either time or distance along the radial or longitudinal line as shown by the time variations in Figs. 9.2.2.2a and b. This approximation is useful because it makes calculations for strain and acceleration from particle velocity much simpler than that for an irregular pulse. Ground motion from a blast is similar to the motion of cork caused by a passing water wave. Displacement of the cork from its at rest position is similar to the displacement u of a particle in the ground from its at rest position. Similarly, the cork's velocity $\dot{u}$ as it bobs up and down is analogous to that of a particle in the ground, hence the term *particle velocity.*

The water wave that excites the cork can be described by its wave length γ, the distance between wave crests; the wave speed or propagation velocity c at which it travels outward from the stone's impact; and the frequency f or the number of times the cork bobs up and down in one second. Frequency f is equal to $1/T$ or the reciprocal of the period or time it takes the cork to complete one cycle of motion. Frequency is measured in cycles per second or hertz, H_3. Propagation velocity c should not be

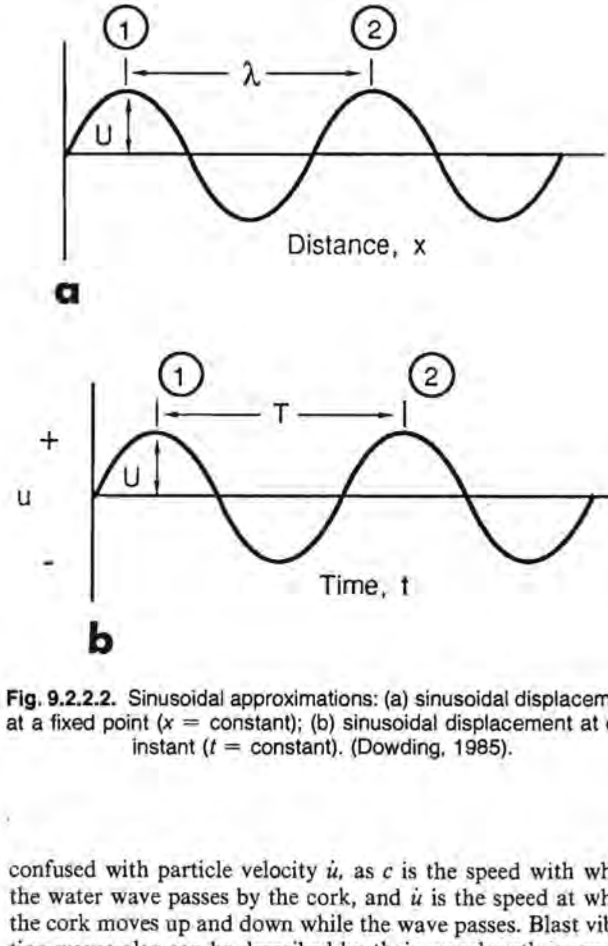


Fig. 9.2.2.2. Sinusoidal approximations: (a) sinusoidal displacement at a fixed point (x = constant); (b) sinusoidal displacement at one instant (t = constant). (Dowding, 1985).

confused with particle velocity $\dot{u}$, as c is the speed with which the water wave passes by the cork, and $\dot{u}$ is the speed at which the cork moves up and down while the wave passes. Blast vibration waves also can be described by their wavelength, propagation velocity, and frequency in the same fashion as the water wave.

Kinemetric Relationships of Ground Motion—The general form for the sinusoidal approximation is best understood by beginning with the equation for sinusoidal displacement u:

$$u = U \sin(2\pi f t) \tag{9.2.2.1}$$

where U is maximum displacement, f is frequency, and t is time.

The relationship between the maximum particle displacement u_{max}, particle velocity $\dot{u}_{max}$, and acceleration $\ddot{u}_{max}$, is also greatly simplified by the sinusoidal approximation and is found through differentiation with respect to time of Eq. 9.2.2.1, as shown, whenever the sin/cos function maximizes at 1:

$$\begin{aligned} u_{max} &= U \\ \dot{u}_{max} &= U\,2\pi f = 2\pi f u_{max} \\ \ddot{u}_{max} &= U\,4\pi^2 f^2 = 2\,\pi f \dot{u}_{max} \end{aligned} \tag{9.2.2.2}$$

Usually, acceleration is normalized (divided) by gravitational acceleration, 386.4 in./sec^2 (9814 mm/s^2). Therefore, an acceleration of 79 in./sec^2 (2000 mm/s^2) is

$$\frac{2000}{9814} = \frac{79}{386} = 0.2 \text{ g}$$

or two-tenths that of gravity.

Kinemetric relations between particle displacement, velocity, and acceleration for complex wave forms are exactly related through integration or differentiation of any of the wave forms. For instance, an acceleration time history can be integrated once for a velocity time history, which in turn can be integrated for a displacement time history. Even though a particle velocity record can be differentiated to find acceleration, it is not recommended, as the procedure is sensitive to small changes in the slope of the velocity time history. Further discussion of the inaccuracies of differentiation and integration can be found in Dowding (1985) and in texts devoted to interpretation of time histories (e.g., Hudson, 1979).

Transient Nature of Blast Motions—Great care should be taken not to confuse the effects of steady-state, single-frequency motions with those of transient, blast motions. Most vibration studies conducted by personnel trained in mechanical and electrical engineering and geophysics implicitly assume that the motions are continuous (last many cycles) and steady state (have constant frequency and amplitude). As can be seen in Fig. 9.2.2.1, blast-induced motions last only one or two cycles at a relatively constant amplitude and frequency. Such conditions are not similar enough to steady state motions to allow specific application of steady state approximations such as resonance.

Estimation of Dominant Frequency—Adoption of frequency-based vibration criteria has made the estimation and calculation of the dominant frequency an important concern. Dominant frequency can be estimated through visual inspection of the time history or calculated with Fourier frequency spectra or, alternatively, response spectra.

The accuracy or difficulty of visually estimating the dominant frequency depends upon the complexity of the time history. The easiest type of time history record for frequency estimation is one with a single dominant pulse like that shown in the inset in Fig. 9.2.2.3. This dominant frequency can be determined through the hand measurement of the time of the two zero crossings on either side of the peak. The difference between these times is one-half of the period, which is the inverse of twice the frequency of the dominant peak as shown in the figure.

As shown in Fig. 9.2.2.3, the relatively large explosions produced by surface coal mining, when measured at typically distant structures, tend to produce vibrations with lower principal frequencies than those of construction blasts. Construction blasts involve smaller explosions, but the typically small distances between a structure and a blast as well as rock-to-rock transmission paths tend to produce the highest frequencies. Such high-frequency motions associated with construction blasts have less potential for cracking adjacent structures (Dowding, 1985).

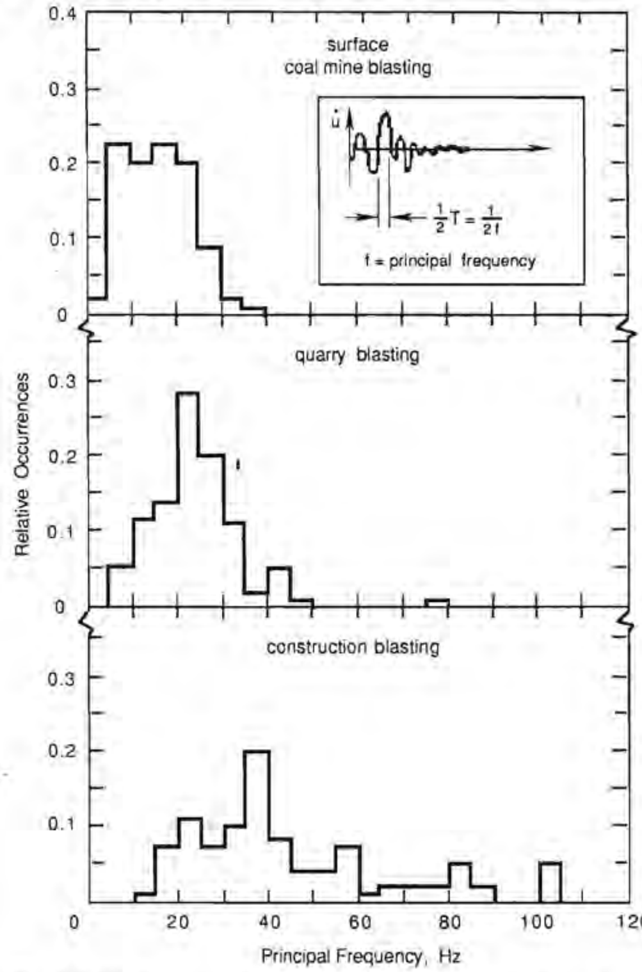


Fig. 9.2.2.3. Dominant frequency histograms at nearest structures categorized by industry. Dominant frequency is defined in the inset. (After Siskind et al., 1980b.)

The most difficult type of record to interpret is that which contains nearly equal peaks at two dominant frequencies such as that in Fig. 9.2.2.1. The two dominant frequencies are the initial 15- to 20-Hz portion (peak A) and the later 5- to 10-Hz portion (peak B). As can be seen in the figure, the initial portion produces the highest wall response while the second produces the greatest superstructure response. For the best frequency correlation of both types of response, both frequencies should be calculated.

The best computational approach to determining the dominant frequency involves the response spectrum. The response spectrum is preferred over the Fourier frequency spectrum because it can be related to structural strains (Dowding, 1985). A compromise approach is to calculate the dominant frequency associated with each peak by the zero crossing approach described above.

Since many time histories do not contain as broad a range of dominant frequencies as that in Fig. 9.2.2.1, most approaches require only the calculation of the frequency associated with the maximum particle velocity for blasts that produce small particle velocities. The more complex frequency analyses are employed only when peak particle velocities approach control limits.

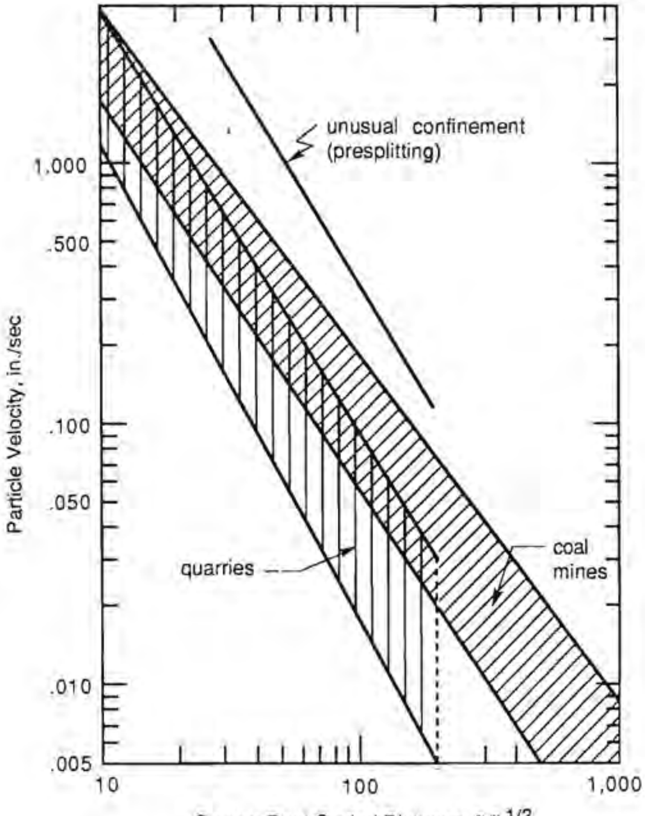


Fig. 9.2.2.4. Attenuation relationships showing scatter from geological and blast design effects as well as high expected velocities from confined shots, such as presplitting. (After Siskind et al., 1980b). Conversion factors: 1 ips = 25.4 mm/s, 1 ft/lb$^{1/2}$ = 0.8197 m/kg$^{1/2}$.

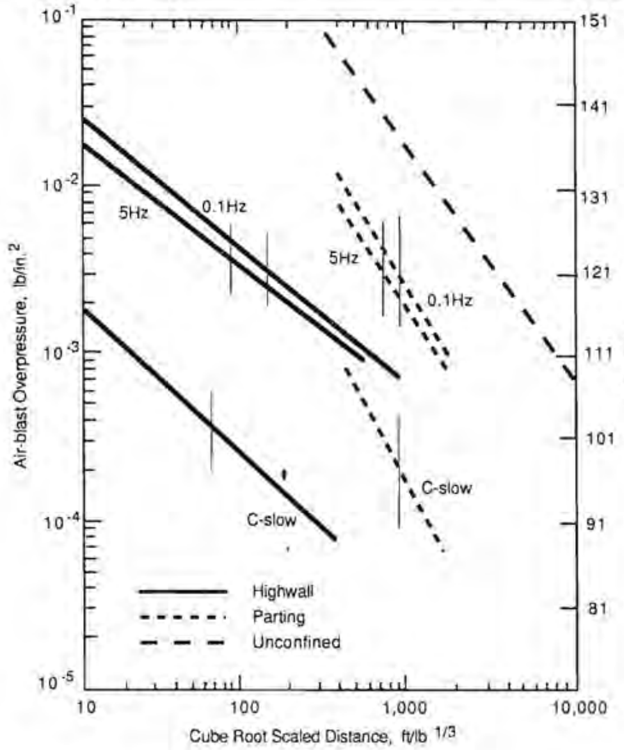


Fig. 9.2.2.5. Attenuation relationships for air overpressures produced by confined (highwall) and partially confined (parting) surface coal mining blasts, as well as unconfined blasts. (Siskind et al., 1980a). Conversion factors: 1 lb/in.^{2} = 6.894 kPa, 1 ft/lb$^{1/3}$ = 0.8759 m/kg$^{1/3}$.

Propagation Effects—Ground motions always decreased with increasing distance. Effects of constructive and destructive interference and geology are included within the scatter of data about the mean trend of the decay in amplitude with distance. While this scatter is large, the associated decay with distance is observed in all blast-vibration studies. Typical examples of this decay are shown in Fig. 9.2.2.4 where maximum particle velocity is plotted as a function of square-root scaled distance from the blast.

Square-root scaling, or plotting peak particle velocity as a function of the distance R divided by the *square root* of the charge weight, $R/W^{1/2}$, is more traditional than the cube-root scaling, which incorporates energy considerations (Hendron, 1977). Both square- or cube-root scaling can be employed to compare field data and to predict the attenuation or decay of peak particle velocity; however, square-root scaling is more popular.

Several square-root attenuation relationships employed in the United States are shown in Fig. 9.2.2.4. They are banded to reflect scatter, which is typical of blasting operations. Curve P should be used for presplitting, cratering, and beginning new bench levels. It is also the basis for Office of Surface Mining (OSM) regulations for conservative shot design when monitoring instruments are not employed.

Dominant frequencies also tend to decline with increasing distance and with increasing importance of surface waves. At larger distances typical for mining, higher frequency body waves begin to have relatively lower peak amplitudes than the lower frequency surface waves, as shown in Fig. 9.2.2.1. Since lower frequencies can elicit greater structural response (Medearis, 1976) as shown in 9.2.2.5, OSM scaled-distance limits decline with increasing absolute distance.

Blast-Induced Air Overpressures: Just as with ground motions, blast-induced air overpressure waves can be described with time histories as shown in Fig. 9.2.2.1. The higher frequency portion of the pressure wave is audible sound. While the lower frequency portion is not audible, it excites structures, which in turn causes a secondary and audible rattle within the structure. The air-blast excitation of the walls is shown by comparing the last one-quarter of the time histories of air blast and wall response in Fig. 9.2.2.1. Unlike ground motions, air overpressures can be described completely with only one transducer, since at any one point air pressure is equal in all three orthogonal directions.

Propagation Effects—Propagation of blast-induced air overpressures has been studied by numerous investigators and is generally reported with cube-root rather than square-root scaled distances. Peak pressures are reported in terms of decibels (dB), which are defined as

$$dB = 20 \log_{10}\left(\frac{P}{P_o}\right) \tag{9.3.2.3}$$

where P is the measured peak sound pressure and P_o is a reference pressure of 2.9×10^{-9} psi (20×10^{-6} (P_a)).

Fig. 9.2.2.5 summarizes the effect of two important instrumentation and shot variables. First, the effect of the weighting scales is dramatically evident. C weighting greatly reduces the recorded peak pressure at any scaled distance. This does not mean the peak is reduced by changing instruments, but rather that the C weighting system does not respond to the low-frequency pressure pulses. These low-frequency pressure peaks excite structures and occupants whether or not they are sensed by the measurement instruments. The other (5- and 0.1-Hz) labels denote the lower-frequency bounds of the recording capabilities of the "linear" systems.

Second, the effect of venting caused by inadequate stemming can be observed in Fig. 9.2.2.5 from the higher average pressures produced by the parting shots at any scaled distance. Parting shots are detonated in thin rock layers between coal strata in surface mines. Consequently, there is less hole height available for stemming, and these shots many times eject the stemming and thereby produce abnormally high air overpressures. The unconfined relationship should be used for demolition of structures after modification for effects of weather and ground reflection.

Various effects of the wind have been reported and should be added to the average relations presented in Fig. 9.2.2.5. Wiss and Linehan's (1978) study of air overpressures produced by surface coal mining showed that in moderate winds the typical 7.7-dB reduction for each doubling of distance is reduced by

$$7.7 - 1.6\ V_{\mathrm{mph}} \cos\theta \text{ dB} \qquad (9.2.2.4)$$

where V_{mph} is wind velocity in miles per hour and θ is the angle between the line connecting the blast and transducer and the wind direction.

An air-temperature inversion causes the sound pressure wave to be refracted back to the ground and at times to be amplified at small, 16-acre (65 km^2) sized locations. Such an inversion occurs when the normal decrease in temperature with altitude is reversed because of the presence of a warmer upper layer. Schomer et al. (1976) has shown that for propagation distances of 2 to 40 miles (3 to 60 km), inversions produce zones of intensification of up to three times the average, attenuated or low air overpressures *at those distances*, with an average increase of 1.8 times (5.1 dB). At distances less than 2 miles (3 km), where high air overpressures are likely to occur, his measurements show no inversion effects.

Structural Strains vs. Particle Velocity: While particle velocity is the traditional measurement of choice, structural strains control cracking. They should be measured directly from relative displacements on structures or within rock masses when critical locations are known, and can be obtained with a variety of strain and relative displacement gages (Stagg et al., 1984). Unfortunately, these critical locations may be either unknown or too many in number to economically measure. Therefore, some means of estimation is necessary.

Ground motion and air overpressure time histories can be employed to calculate the relative displacement of structural components with a knowledge of the responding structure's dynamic response characteristics (Dowding, 1985). These relative displacements can in turn be employed to calculate strains. The accuracy of these estimates is limited by the degree to which the structure behaves as a single degree of freedom system and the accuracy of the estimate of the dynamic response characteristics.

Appropriate Measurement of Particle Velocity: While any of the three kinemetric descriptors (displacement, velocity, or acceleration) could be employed to describe ground motion, particle velocity is the most preferable. It has the best correlation with scientific observation of blast-induced cracking, which forms the basis of vibration control. Furthermore, it can be integrated to calculate displacement. If acceleration is desired, it should be measured directly to avoid differentiation of the particle velocity time history.

The location for measurement varies throughout the world. In North America, the excitation or ground motion is measured on the ground adjacent to the structure of interest. In Europe, the excitation motion is measured on the structure's foundation. The difference stems from historical precedent and location of transducers during scientific observation of cracking rather than difference in philosophy. In North America, many times it is impossible to place transducers on adjacent property owned by a party not involved in the project. Furthermore, if it is desired to describe the excitation motions, then they should be measured outside of and not on the structure. If it is desired to measure structural response motions then they should be measured on the most responsive structural members, which are not the basement or foundation walls because of the restraint provided by the ground.

Time histories of the three components of motion should be measured because of the importance of excitation frequency. Recording of peak motions will not yield information about the dominant frequency and time history details that control structural response. Peak motions and dominant frequency can be employed to describe low-level, non-critical motions. Therefore, machines employed to monitor critical motions (type I in 9.2.2.4) should be capable of recording time histories of selected critical motions. Machines that record only peak motions (type II in 9.2.2.4) can be employed with those that record time histories to provide redundant measurement where frequency content does not vary widely.

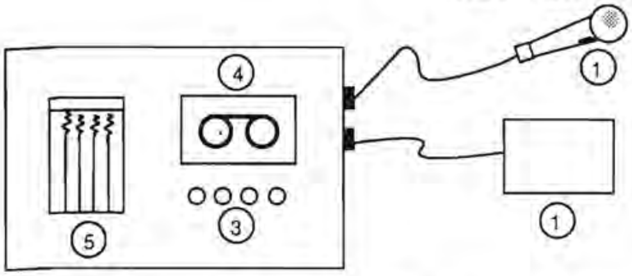
9.2.2.4 Measurement Instruments and Their Deployment

This segment describes characteristics of instruments that measure the ground motions (acceleration, velocity, displacement) and air blast (air overpressure). Since there are many excellent sources for information on instruments, the principal characteristics of available systems will be summarized rather than exhaustively reviewed. The most complete single reference for detailed instrumentation information that is updated periodically is the *Shock and Vibration Handbook* (Harris and Crede, 1976. Specific information on blast vibration monitors is contained in specific publications by the US Bureau of Mines and the Office of Surface Mining (i.e., Rosenthal and Morlock, 1987).

An idealized, field-portable blast monitoring system operating on a 12-V battery is illustrated in Fig. 9.2.2.6. It consists of transducers (1) that convert physical motion or pressure to an electrical current, which is transmitted through cables (2) to an amplifying system (3); and a magnetic tape, paper or computer digital, recorder (4) that preserves the relative time variation of the original signal for eventual permanent, hard-copy reproduction by a pen recorder or light-beam galvanometric recorder or dot matrix printer (5). As one can imagine, there is an almost endless variety of configurations of these five basic components. However, the best involve micro processors (computers) for data acquisition, storage and reproduction.

Transducers: Transducers are one of the weaker links in the measurement system because they must translate kinemetric motions or pressures to electrical signals. The remaining components transform electrical signals or light beams and are not restricted by mechanical displacement. The main characteristics of transducers that affect their performance are sensitivity and frequency response.

Sensitivity of an instrument is the ratio of its electrical output to its kinemetric displacement, velocity, and acceleration or overpressure for energy-converting transducers (i.e., do not require



1. Velocity (3 orthogonal) and sound pressure transducers
2. Cables
3. Amplifier
4. Recorder (tape, disk, or memory)
5. Light beam oscilloscope or dot matrix printer

Fig. 9.2.2.6. Idealized, field portable blast monitoring system that shows the schematic relationship of the five principal components. (Dowding, 1985).

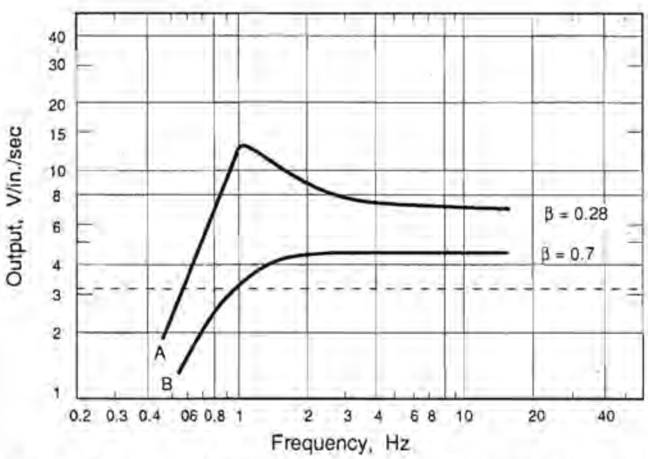


Fig. 9.2.2.7. Example response spectra of a velocity transducer with differing percentages of damping. With 70% of critical damping, this system is ± 3 dB (± 30%) down 1 Hz. (Dowding, 1985.) Conversion factor: 1 ips = 25.4 mm/s.

an energy source). Since allowable limits are specified in terms of ground particle velocity, all blast monitors come equipped with velocity gages.

Frequency response is the frequency range over which the electrical output is constant with a constant mechanical motion. This constancy is normally expressed in terms of decibels. For instance, linear within 3 dB between 5 and 200 Hz means that the transducer produces a voltage output that is constant within 30% between 5 and 200 Hz. Generally, it is better to look at the transducer's response spectra (such as those shown in Fig. 9.2.2.7) to determine the frequencies where this difference occurs. For example, the difference occurs at low frequencies for the velocity transducers in the figure. The importance of the frequency response of air overpressure transducers was discussed in 9.2.2.3.

Transducer Attachment—One of the most critical aspects of vibration monitoring is the mounting of the transducers in the field. The importance of mounting is a function of the particle acceleration of the wave train being monitored. The type of mounting on a horizontal surface is the least critical when the vertical maximum particle accelerations are less than 0.3 g. In this range, the possibilities of rocking the transducer or the transducer package are small, and the transducer may be placed upon a horizontal measurement surface without a device to supply a holding force. When the maximum particle accelerations fall between 0.3 and 1.0 g, the transducer or transducer package should be buried completely when the measurement surface consists of soil (Johnson, 1962). When the measurement surface consists of rock, asphalt, or concrete, the transducers should be fastened to the measurement surface with either double-sided tape, epoxy, or quick-setting cement (Hydrocal or other gypsum-based cements set within 15 to 30 min). If these methods are unsatisfactory, or accelerations exceed 1.0 g, only cement or bolts are sufficient to hold the transducer to a hard surface.

All transducers mounted on vertical surfaces should be bolted in place. Air overpressure transducers should be placed at least 3 ft (1 m) aboveground, pointed downward (to prevent rain damage) and fitted with a windscreen to reduce wind excitation-induced false events.

Digital, Tape, and Hard-Copy Recorders: Microprocessor (computer) or digital recording systems now dominate technical recording because of the ease of computer linkage. The signal is sampled at a certain rate, say, 1000 times/sec, and each sample is converted to a single magnitude. This magnitude and its associated time are then stored in computer memory. Digital recording has several advantages. It is very accurate, as variation in the speed of the tape, if it is used, has no effect, and records can be directly accessed by a computer. Details of the digitization process are discussed elsewhere (Dowding, 1985).

Of those blast-monitoring systems with tape recorders, most employ compact FM cassettes. Many of the systems involve separate record and reproduction modules to reduce the complexity of recording. Care should be exercised to determine the exact details of the system before purchasing, as tape recorder performance varies at low temperature.

A permanent record or "hard copy" of the vibration time history is usually made on photographic film, floppy disk or battery-powered memory chips or paper. Almost all present film-based recorders employ special field-developable, ultraviolet light sensitive paper in combination with light-beam galvanometers to record high frequency motions. The newest generation recorder employs dot matrix printers and/or floppy disks with microcomputers. Unfortunately, those monitors that print after a vibration event may not be recording another event while printing. If multiple shots are likely, this reset time should be determined. Furthermore, printer behavior in cold weather is variable and should also be investigated.

Most recorders can be bought as either single- or multichannel units. A four-channel unit is necessary in blast monitoring to record simultaneously the three components of the ground motion (L, V, and T) and the air blast. The present trend in vibration equipment is to include a signal-conditioning amplifier in the recorder to allow flexible amplification of the signals.

Frequency analysis of records requires a time history and thus some form of permanent record. Instruments recording only peak particle velocities will not allow a frequency analysis. Sending records through the mail for interpretation, results in a delay of five days, and sometimes up to a month. Systems with light sensitive paper or dot matrix printers allow immediate interpretation of frequency without additional costly equipment.

Calibration: It is obvious that the entire vibration measurement system should be calibrated, as it is futile to record data if they cannot be exploited because of a lack of reference. Manufacturers supply calibration curves with their instruments that are similar to the response spectra for transducers shown in Fig. 9.2.2.7. Recalibration or checking requires special vibrating platforms where frequency and displacement are controlled, and, in the field, a calibrating circuit to pulse the magnetic core of the geophone (Stagg and Engler, 1980).

Number of Instruments: While the obvious irreducible number of instruments for each blast is one, two would provide a more thorough documentation of the spatial distribution of effects. If only one instrument is employed, then it should be located at the nearest or most critical receiver. This single type I instrument should record time histories of the three axes of particle velocity as well as air overpressure. Since it must monitor continuously, it must trigger (begin recording) automatically, and be capable of monitoring even while printing or communicating results. When blasting will occur at more than one general location (i.e., involve nearest structures separated by hundreds of feet or meters), then two and four are the irreducible and optimum number of instruments, respectively. The third should be a spare to insure continuous coverage in case of failure.

The second and fourth instruments in the situations described here may provide a smaller level of service and will be termed type II. They must at least continuously record the peak particle velocity in one axis and may or may not measure air overpressure. The best axis is the vertical, since no horizontal direction decision is required and surface waves usually involve a significant vertical component regardless of the direction of the maximum horizontal component. These instruments should be located at a greater distance than the nearest structure to monitor a large area.

The third or spare instrument can be either type I or II. Where air overpressures will be problematic or frequencies critical, the spare should be type I. This spare instrument can also be employed to monitor sites where complaints develop. This public relations work is essential in North America where lawsuits arise even when all blast effects comply with regulatory guidelines.

This approach describes the least number of instruments. Applicable regulations and mining schedules may require a larger number.

Instrument Deployment During Test Blasts: When blasting projects begin, when geological conditions change radially, or when new initiation systems are introduced, test blasts should be conducted to minimize the number of instruments necessary to monitor production blasts. These tests are conducted to produce project-specific attenuation relations for both air overpressures and ground motion. Such relations vary from project to project because of changes in geology and blasting practices. Additionally, the test blasts allow the determination of the frequency content of motions at different scaled and absolute distances. Frequency is important in estimating structural response through response spectrum analyses.

The attenuation relation is not solely a site property. Although it is dependent upon geology, it is also heavily dependent upon the blast geometry and timing. For instance, with the same charge per delay, a blast with a larger burden will produce an attenuation relation with a similar slope or decay with distance, but with a larger intercept. Furthermore, differing initiation timing will produce changes in the time history, both length and frequency content.

During test blasts, a minimum of four instruments should be deployed to measure peak particle velocity at widely differing scaled distances for the same blast. Therefore, for any one blast design, parameters and initiation sequence are constant, and the resulting attenuation relationship shows only the effect of distance, direction, and/or geology. Seismographs and/or transducers should be placed along a single line with constant geology to determine best the attenuation relationship, or at all critical structures to determine the effects of direction and variable geology. Ideally, the linear orientation should be along a path with constant thickness of soil and not cross any large geologic discontinuities such as faults. If geology changes radically, then two such attenuation lines are necessary, but not necessarily with each blast.

A number of approaches to blast design for vibration control are now available that employ a single-delay, single-hole test blast and a number of instruments to record the attenuation and frequency change around the site. These single-time histories are then synthesized to reproduce the additive time history effects of multiple delay, multiple hole blasts at the differing instrument sites. Such synthesis of time histories to guide blast design has met with variable success, but does not replace monitoring of blast effects at critical structures during production blasting.

9.2.2.5 Evaluation of Measurements

Documentation of blast effects involves two radically differing endeavors: measurement of ground and air disturbances as well as observation of cosmetic cracking. Measurement can now be accomplished remotely with computers to eliminate completely human interaction, whereas scientific observation must involve meticulous human inspection immediately before and after a blast. While the focus of this section is instrumental monitoring, the alleged appearance of cracking by neighboring property owners is nonetheless a very serious consideration.

Principal problems in the evaluation of measured effects involve (1) accounting for geologic and weather effects on the overall attenuation with a small number of instruments and (2) incorporating structural response and frequency effects. Principal problems with the observation of blast-induced cracking involve (1) separating blast-induced from environmentally and human-induced cracking and (2) reducing the enormous amounts of time necessary for direct observation. Observational problem are normally overcome by employing instrumentationally measurable blasting controls at low enough levels to prevent the threshold of cosmetic cracking to even old, degraded structures and eliminating observation altogether. Otherwise blast-induced cracking can be observed only with immediate before and after blast inspection. The remainder of this discussion concentrates on the instrumentation approach and calculation of structural response.

Structural Response and Frequency Effects: Structures respond to both ground and airborne disturbance, as shown by the bottom four time histories in Fig. 9.2.2.1. Walls respond more to the higher frequency (15 to 20 Hz) waves in the early portion of the ground motion, while the superstructure or overall skeleton of the structure responds more to the last or lower frequency (5 to 10 Hz) portion. Walls are again excited by the arrival of the air pressure wave. Structural response can be calculated from the ground motions if the natural frequency and damping of structural components are known or estimated.

Langan (1980) has shown that measured structural response has a higher correlation coefficient with calculated single degree of freedom (SDF) response than with peak ground motion. Therefore, structural motions can be estimated more accurately by assuming that they are proportional to response spectrum values at the particular structure's natural frequency than by assuming that they are proportional to the peak ground motion. This improved correlation is largely a result of the consideration of frequency in the response spectrum, which is calculated from SDF response.

Origin of the SDF Model—One of the critical structural response factors is the amount of differential displacement (δ in Fig. 9.2.2.8) that occurs between or along structural members because it is proportional to strain, which, in turn, causes crack-

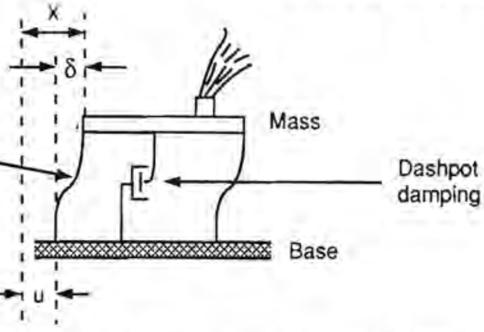


Fig. 9.2.2.8. Single degree of freedom model of house that shows relative displacements of the walls δ, and the analogy between model mass and roof mass, model stiffness, and wall stiffness. (Dowding, 1985.)

ing. Such displacements can be computed with a mathematical idealization of the SDF model shown in Fig. 9.2.2.8.

It is necessary to simplify a structure so that computations are practical. The fundamental characteristics of a structure that govern its behavior under vibratory or dynamic loading are (1) the masses of the main components (analogous to floor and roof masses), (2) the spring stiffnesses of the main components (analogous to wall stiffness), and (3) the amount of damping or energy dissipation (analogous to differential movement in cracks, joints, and connections). Behavior of one- or two-story buildings is directly analogous to the behavior of an SDF system when movement in only one direction is considered. When multistoried structures are considered, it is necessary to model the structure as multidegree-of-freedom systems. However, even such a system may be idealized as a single-degree-of-freedom system to calculate the fundamental mode of response.

If a structure's damped natural frequency f_d and its fraction of critical damping β are known, values of dynamic properties, f_d and β, can be accurately measured from a free vibration time history of the building response. These measured parameters automatically account for the factors that are difficult to quantify, such as the degree of fixity of the columns and the damping coefficient. As shown in Fig. 9.2.2.1, these parameters can be measured from the structure's free response. Time between peaks is the period, $T = 1/f_d$, and the decay of free oscillation is proportional to the damping, β.

Estimation of Dynamic Response Properties—The fundamental natural frequency f_d of the superstructure of any tall building can be estimated from compilations of work in earthquake engineering (Newmark and Hall, 1982):

$$f_d = \frac{1}{0.1N} \tag{9.3.2.5}$$

where N is the number of stories. Substitution of 1 and 2 for residential structures for N yields f values of 10 and 5 Hz for one- and two-story structures, which compares favorably with the results of actual measurements.

Damping β is a function of building construction and to some extent the intensity of vibration. Thus it cannot be simplified as easily as the natural frequency. Measurement reveals a wide range of damping for residential structures with an average of 5% (Dowding et al., 1981). This value is also appropriate for initial estimates involving taller engineered structures. Further details for engineered structures can be found in Newmark and Hall (1982).

Table 9.2.2.2. Natural Frequencies for Unusual Structures

Type	Height, m	f, Hz
Radio tower[a]	30	3.8
Petroleum distillation tower[a]	21	1.2
Coal silo	60	0.6
Bryce Canyon rock pinnacle[b]	27	3

Sources: [a]Medearis (1975b).
[b]Dowding and Kendorski (1983).
Conversion factor: 1 ft = 0.3048 m.

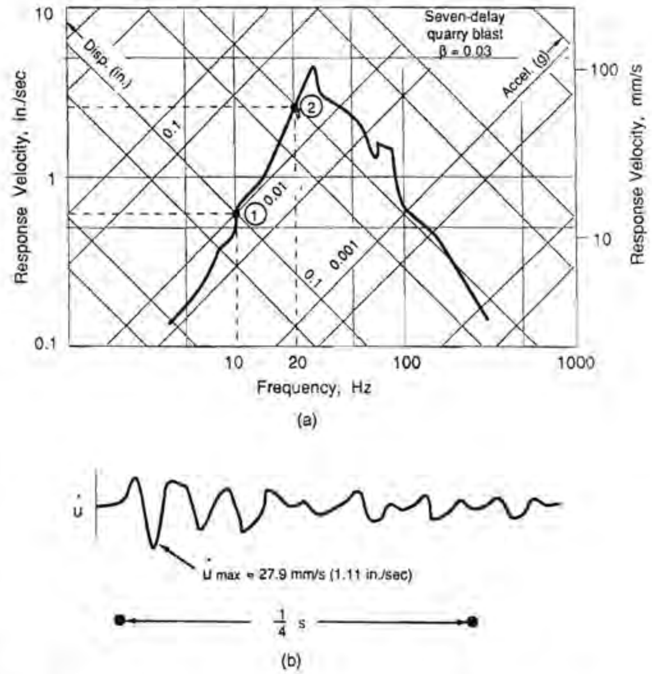


Fig. 9.2.2.9. Construction of pseudovelocity response spectrum: (a) response spectra; (b) associated excitation time history. The time history is operated upon by the single degree of freedom (SDF) equation to produce a computed relative displacement (δ), which is then multiplied by the circular natural frequency ($2\pi f$; or $2\pi 10$ for point (1)) to produce the pseudo response velocity. (Dowding, 1985).

Walls and floors vibrate independently of the superstructure and have their own, but similar, fundamental frequencies of vibration that range between 12 and 20 Hz with an average value of 15 Hz (Dowding et al., 1981). Floors tend to have lower natural frequencies in office buildings with large floor spans, but are similar to wall natural frequencies in residential structures.

Dynamic response properties of some tall, unique structures cannot be estimated with the $1/0.1\,N$ equation. Field-measured natural frequencies for these types of structures are given in Table 9.2.2.2.

Response Spectrum—The pseudovelocity response spectrum of a single ground motion, such as that of a seven-delay quarry blast in Fig. 9.2.2.9, is generated from the relative displacement δ_{max} values of a number of different SDF systems when excited by that motion. Consider two different components of the same structure, the 10-Hz superstructure and the 20-Hz wall. If the ground motions, $\dot{u}(t)$ in Fig. 9.2.2.9b, of the seven-delay quarry blast are processed twice by the SDF response equation with

$f = 10$ and 20 Hz and β held constant at 3%, two δ_{max} values will result.

The first computation is made with the 10-Hz system, which has a circular natural frequency of

$$p = 2\pi(f) = 2\pi(10) \tag{9.2.2.6}$$

and results in an SDF equation computed

$$\delta_{max} = 0.25 \text{ mm } (0.01 \text{ in.}) \tag{9.2.2.7}$$

This δ_{max} is then converted to pseudovelocity, PV, as

$$PV_{10} = p\delta_{max} = 2\pi(10)(0.25)$$
$$= 15.7 \text{ mm/s } (0.62 \text{ ips}) \tag{9.2.2.8}$$

and is plotted as point 1 in Fig. 9.2.2.9a. The same computation is then repeated for the 20-Hz system.

$$p = 2\pi(20)$$
$$\delta_{max} = 0.5 \text{ mm } (0.02 \text{ in.})$$
$$PV_{20} = 2\pi(20)(0.5) = 63.5 \text{ mm/s } (2.5 \text{ ips})$$

and PV_{20} is plotted as point 2 in Fig. 9.2.2.9a. If the same ground motion time history in Fig. 9.2.2.9b is processed a number of times for a variety of fs with β constant, the resulting pseudovelocities will form the solid line in Fig. 9.2.2.9a.

Fourier Spectra DO NOT Directly Predict Response—With increasing use of computers, calculation of various spectra from time histories have become commonplace. The two most common are the Fourier frequency and pseudovelocity response spectra. Although they are essentially different in meaning and typical use, they are similar for undamped response where the maximum motion occurs near the end of the time history (Dowding, 1985; Hudson, 1979). Since response spectra are calculated for damped response and peaks normally occur in the middle as well as the beginning of the time history, the two spectra are not usually the same.

Only the pseudovelocity response spectrum can be employed to calculate directly structural response. Because of the similarity of Fourier and response spectra, either can be employed to determine the dominant frequency in the ground motion.

Case Histories Demonstrate Importance of Response Spectrum Analysis: Fig. 9.2.2.10 compares time histories and response spectra from the longitudinal components of an urban construction blast and a surface coal mine blast. Although the peak particle velocities are similar: 0.15 ips (3.8 mm/s) for the construction blast A; and 0.13 ips (3.3 mm/s) for the surface mining blast B; the response spectra differ radically. This difference is greatest in the range of natural frequencies of residential structures and their components, 5 to 20 Hz. In this range the surface mining motions produce response velocities that are 10 times greater than the construction blast.

Surface Mine Blast—Surface mining induced ground motion was produced by a multiple row blast. Some sixty 83-ft (25-m) deep, 15-in. (380-mm) diameter, holes were arranged in a four-row pattern. The burden between rows was 20 ft (6.1 m) and the hole spacing was 25 ft (7.6 m). Each hole contained four decks (or charges that are detonated at intervals separated by at least 17 ms). The ammonium nitrate-fuel oil (ANFO) charge weight per deck ranged from 100 to 130 lb (45 to 60 kg). Therefore, the largest charge per delay was 130 lb (60 kg), and the total charge was 27,700 lb (12,600 kg).

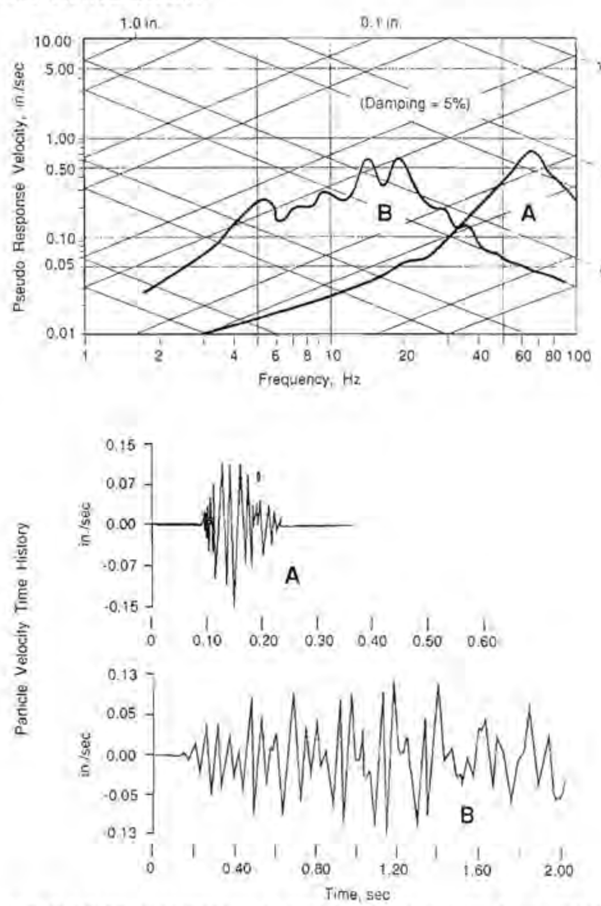


Fig. 9.2.2.10. Comparison of time histories spectra from construction and surface mining blasts respectively lasting 0.15 and 2.0 sec. Even though the particle velocities are approximately equal, responses in the 5 to 20 Hz frequency range differ greatly. Conversion factor: 1 ips = 25.4 mm/s.

Geology between the blast and the transducer, located at the nearest residence, consisted of sedimentary rock with 10 to 30 ft (3 to 10 m) of overlying silty glacial till. A small, 33-ft (10-m) deep, gully was located some 1300 ft (400 m) north of and between the blast and transducer. Soil depth at the transducer was 10 to 15 ft (3 to 4 m). Some 2750 ft (825 m) separated the shot from the transducer, where the longitudinal velocity time history in Fig. 9.2.2.10 with a peak of 0.13 ips (3.3 mm/s) was recorded. The accompanying transverse and vertical peak particle velocities were 0.18 and 0.09 ips (4.6 and 2.3 mm/s) with dominant frequencies between 11 and 13 Hz. Other similarly designed shots with distances between 1900 and 2700 ft (580 and 825 m) produced peak particle velocities between 0.17 and 0.23 ips (4.3 and 5.8 mm/s) with dominant frequencies between 13 and 17 Hz.

These particle velocities are high at 2700 ft (825 m) according to scaled distance relationships. These greater than normal particle velocities may be a result of unusual confinement (too large a burden) or delay overlap. For instance, if two delays had overlapped to add, then the maximum charge per delay would have doubled and the square-root scaled distance would have declined by 30%. Particle velocities measured at smaller scaled distances were closer to expected levels.

Urban Construction Blast—Construction blast-induced motion was produced by a much smaller shot than the surface mining example. Some five 12-ft (3.6-m) deep, 1.5-in. (38-mm) diameter holes were arranged in a single row. Each hole was charged with a stick gelatin dynamite and initiated separately with a constant 25 ms between each delay. The burdens and hole spacings were small, approximately 2 to 3 ft (0.6 to 0.9 m). The total charge was 20 lb (9 kg), and the maximum charge per delay was 5 lb (2.3 kg).

The structure of concern, a historic theater, and the recording transducers were located some 50 ft (15 m) away. Rock being fragmented consisted of granitized biotite schist. This shot produced peak particle velocity in the L, T, and V axes of 0.15, 0.16, and 0.28 ips (3.8, 4.1, and 7.1 mm/s), respectively, with dominant frequencies between 75 and 125 Hz.

Because these dominant frequencies were so high compared to the natural frequencies of the theater's structural components, their response was less than the peak excitation particle velocity. The transducer recording the excitation motions was located in the basement because there was no stable location outside as rock was being removed immediately adjacent to the theater's wall. Another vertical axis transducer was placed at the mid span of the theater's balcony, whose left most support was immediately above the transducer measuring the excitation motions. The peak vertical particle velocity of the theater's balcony was only 0.14 ips (3.6 mm/sec), approximately half of the peak excitation motion in that axis.

Restrained Structures and Rock Masses: Capacity for free response allows aboveground structures such as homes and rock pinnacles to amplify selectively incoming ground motions. On the other hand, buried or restrained structures such as pipelines and rock masses cannot respond freely. Regardless whether response is restrained or free, cracks are initiated by strains in either case. Whereas strains in a freely responding structure are proportional to the relative displacement between the ground and the superstructure as shown in Fig. 9.2.2.11, strains in a restrained structure such as pipelines will usually be those of the surrounding ground and can be approximated as those produced by plane wave propagation and are

$$\epsilon = \frac{\dot{u}}{c_c} \text{ and } \gamma = \frac{\dot{u}}{c_s} \qquad (9.2.2.9)$$

where ϵ and γ are axial and shear strains, c_c and c_s are compressive and shear wave propagation velocities, and $\dot{u}$ are maximum compressive and shear wave particle velocities, respectively (Dowding, 1985). For cases involving one critical location along a pipeline, the pipe strains should be measured directly on the metal (Dowding et al., 1990). For cases involving tunnel and/or cavern liners critical strains can be estimated through calculation of the relative flexibility of the rock and liner (Hendron and Fernandez, 1983).

9.2.2.6 Controlling Blast Effects

Direct regulation or specification of effects, rather than design, is the most effective control from a regulatory viewpoint because effects are so dependent upon details of the shot geometry and initiation sequence. Such dependency renders control impossible by simple regulatory specification of two- or three-shot design parameters. For instance, consider control by specification of the maximum charge weight detonated per instant at given distances from the nearest structure. Even with such detailed specification, intended vibration limits at the structure may be exceeded because of poor choice in the location of holes and/or their relative time of initiation. Discussion of shot design is presented earlier in this chapter (segment 9.2.1).

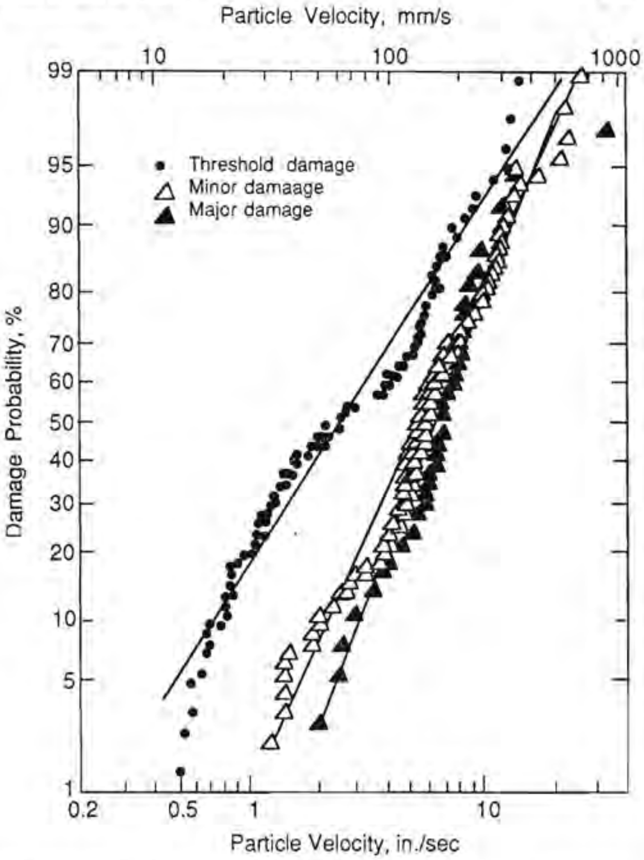


Fig. 9.2.2.11. Probability analysis of worldwide blast cracking data. (Siskind et al., 1980b.) Threshold damage is the occurrence of hair-sized, cosmetic cracks similar to those caused by natural, environmentally induced expansion and contraction.

Present regulatory control limits in many countries are below those levels at which cosmetic cracking may appear. There are two principal reasons for such tight restrictions. First, regulatory limits are influenced heavily by human response to blast-induced vibration and noise. Since humans are approximately 10 times more sensitive than structures to vibration, low regulatory limits are understandable. Second, many regulations appear to have been adopted without the documented, scientific experimentation necessary to determine the vibration levels that cause cracking.

Statistical Analysis of Data with Pre- and Post-Blast Inspection: Unmeasurables in observation can be taken into account indirectly by considering the appearance of cosmetic cracks as a probabilistic event. In order to investigate the effects of certain data sets on the overall conclusions, the probability computations of cracking at given particle velocity levels have been made several times (Siskind et al., 1980b, 1981). All of these studies involve both immediate pre- and post-blast inspection of walls in residential structures, many of which were old, distorted, and whose walls were covered with plaster. Definitions of the observed cracking in each study are described in 9.2.2.2.

Data from various sets of observations were analyzed with cracking points and the assumption that every cracking point excludes the possibility of noncracking at a higher particle velocity (Siskind et al., 1980b, p.55). If the probability of cracking is calculated as the percentage of points at lower levels of velocity, the result is the log-normal scaled plot of the probability of

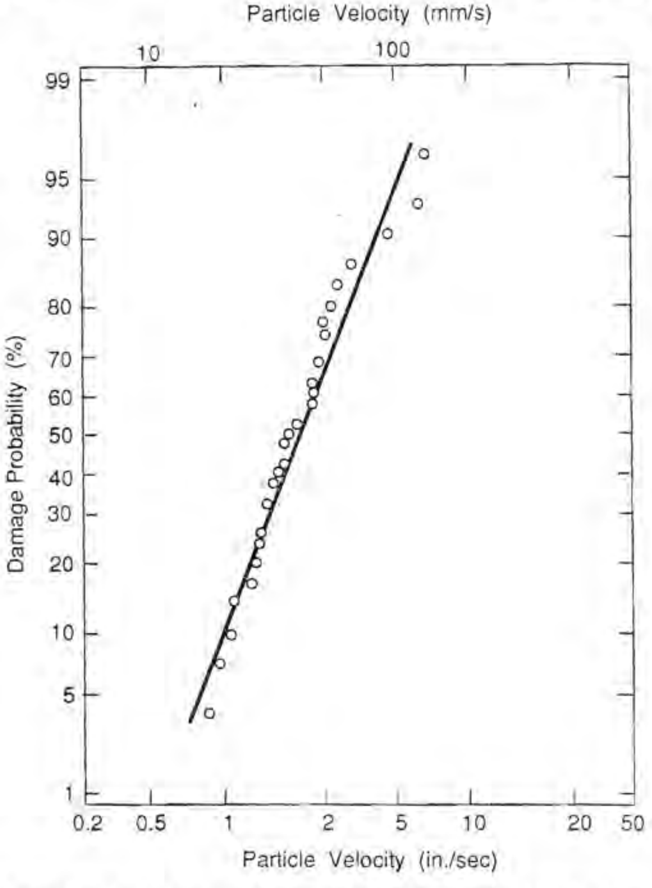


Fig. 9.2.2.12. Probability analysis of blast-induced threshold cracks observed by US Bureau of Mines. (Siskind, 1981.)

cracking vs. particle velocity in Fig. 9.2.2.11. This approach seems conservative as low particle velocity observations do not count noncracking at higher levels.

According to Fig. 9.2.2.11, there appears to be a lower limit of particle velocity of 0.5 ips (12 mm/s) below which no cosmetic or threshold cracking (extension of hairline cracks) has been observed from blasting anywhere in the world. This observation includes the data with unusually low frequencies that were collected by Dvorak (1962). His data are those that tend to populate the lower region of Fig. 9.2.2.11. High-frequency data (> 40 Hz) show that a 5% probability of minor cracking does not occur until particle velocities reach 3 ips (75 mm/s) (Siskind et al., 1980b).

Admissibility of Dvorak's data has been questioned because of the absence of time histories; some of the other studies, such as that by Langefors et al. (1958), are also plagued by the same lack of time histories. To resolve this difficulty, only the new US Bureau of Mines observations have been included in a recomputation of probabilities in Fig. 9.2.2.12. The observations include low frequency motions associated with surface mining. Again there is a particle velocity, 0.79 ips (20 mm/s), below which no blast-induced cracking was observed.

Comparison of Blast and Environmental Effects: Crack width changes from ground motions less than 1 ips (25 mm/s) are less than those caused by the passage of weekly weather fronts (Dowding, 1988). This conclusion was reached after measuring the displacement response of a poorly built, nonengineered house to surface coal mining vibrations for some 8 months. Displacements were measured at ten different wall positions that included cracked and uncracked wall covering. Weather and blast-induced crack displacements across the most dynamically responsive wall covering crack are compared in Fig. 9.2.2.13. The continuous and highly cyclical curve is that of displacements produced by environmental change. The small circles are the maximum, zero-to-peak, dynamic displacements recorded by the same gage. Even though the maximum recorded particle velocity was as high as 0.95 ips (24 mm/s), the maximum weather induced displacements were three times that produced by blasting. On other gages, weather changes produced displacements that were 10 times greater than those produced by blasting.

Special Considerations: The statistically determined control limits are too low for basement walls and engineered structures. They were based on response of residential structures and the lower limit cases involving cracking of above ground plaster or gypsum wall board wall coverings in older, distorted structures.

Engineered Structures—Concrete is a good deal stronger than plaster. Therefore, engineered structures constructed of concrete can withstand maximum particle velocities of at least 4 ips (100 mm/s) without cracking (Crawford and Ward, 1965). Furthermore, buried structures such as pipelines and tunnel linings are not free to respond, as were the above ground residential structures whose response provides the data from which most limits are chosen. Therefore underground structures are able to withstand even greater excitation motions (Dowding, 1985).

Specific engineered structures should be analyzed in terms of the strain that can be withstood by critical elements and the strain should be measured. This approach is particularly appropriate for singular structures with isolated portions nearer to the blast source.

Fatigue or Repeated Events—Since current regulatory limits are so low as to restrain blast-induced displacements below those caused by the passage of weekly weather fronts, the question of repeated events becomes moot. Weather by itself over the years produces greater repeated event effects than does blasting.

A repeated event experiment conducted at the US Bureau of Mines test house (Stagg et al., 1984), confirms the low level of current regulatory controls with respect to fatigue cracking. The test house was framed in wood with paper-backed gypsum board interior walls. When continuously vibrated at an equivalent ground particle velocity of 0.5 ips (12 mm/s), no response was observed until 52,000 cycles, when a taped joint between sheets of gypsum board cracked. These taped joints are the weakest and most compliant zones in a house with paper-backed gypsum board walls.

Rock Mass Displacement and Cracking—Cracking in rock immediately adjacent to a blast can be controlled by limiting the particle velocities to 27 ips (700 mm/s) in the volume of rock to be protected (Holmberg and Persson, 1978).

Rock displacement by forces produced by delayed gas pressure cannot be controlled by specifying an allowable particle velocity. Fortunately, these displacements occur only very close to a blast, within 100 to 165 ft (30 to 50 m), and are associated with blocks that are unconstrained by other surrounding rock.

Sliding instability of individual rock blocks must be evaluated on a case by case basis. Each block must have an adequate factor of safety to prevent static failure (Dowding and Gilbert, 1988).

Frequency Based Control with Dominant Frequency: Fig. 9.2.2.14 shows the limit adopted by the US Office of Surface Mining that is based on a suggested, but not rigorously validated, proposal by the US Bureau of Mines (Siskind et al., 1980a). Corner 1 represents the lowest particle velocity at which USBM

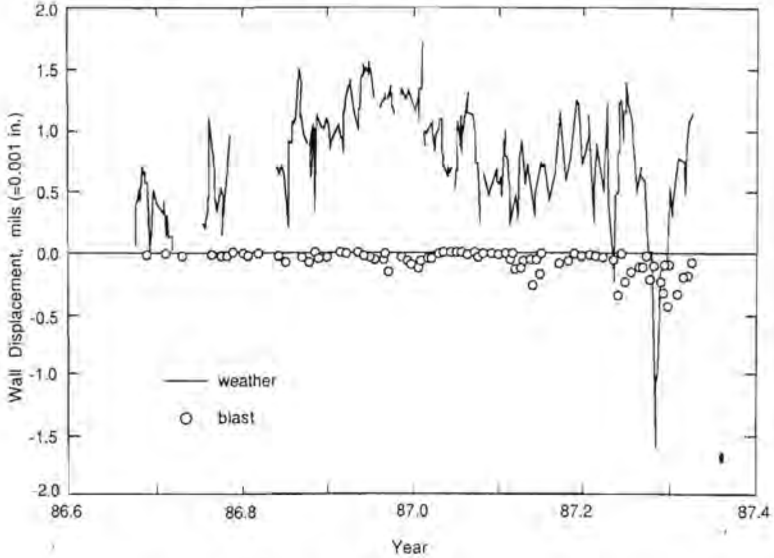


Fig. 9.2.2.13. Comparison of displacements produced by weather-induced changes in humidity and temperature (continuous line) with those produced by surface coal mine induced ground motions (O's). Conversion factor: 1 mil = 0.0254 mm.

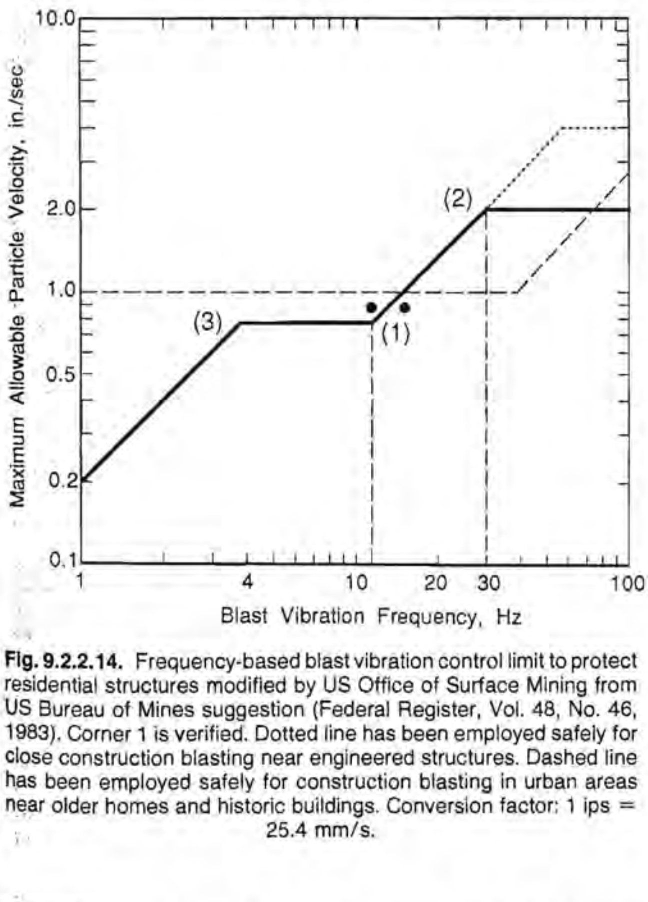


Fig. 9.2.2.14. Frequency-based blast vibration control limit to protect residential structures modified by US Office of Surface Mining from US Bureau of Mines suggestion (Federal Register, Vol. 48, No. 46, 1983). Corner 1 is verified. Dotted line has been employed safely for close construction blasting near engineered structures. Dashed line has been employed safely for construction blasting in urban areas near older homes and historic buildings. Conversion factor: 1 ips = 25.4 mm/s.

personnel have observed cosmetic cracking. Neither of the corners 2 and 3 have been confirmed.

Dominant frequency that is consistent with Fig. 9.2.2.14 is that associated with the peaks in the time history with amplitudes greater than 50% of the peak or maximum particle velocity. The frequency of these peaks were calculated from the zero crossing method as shown in the inset for Fig. 9.2.2.3. Determining frequency from that associated with the peak particle velocity is a good first approximation and eliminates the need for sophisticated Fourier or, alternatively, response spectra analysis. Response spectrum analyses are the most precise approach to account for the frequency effects of structural response and should be employed in singular cases where an exacting analysis is required.

Table 9.2.2.3. Air Overpressure Control Limits as a Function of Instrument Frequency Weighting

Lower frequency limit of measuring system, in Hz (−3 dB)	Maximum level, in dB
0.1 Hz or lower—flat response[a]	134 peak
2 Hz or lower—flat response	133 peak
6 Hz or lower—flat response	129 peak
C-weighted—slow response[a]	105 peak dBC

[a]Only when approved by the regulatory authority.

Regulatory Compliance for Air Overpressure: Although broken glass is normally associated with excessive air blast overpressures, limits in the United States are based upon wall response necessary to produce wall strains equivalent to those produced by surface coal mining-induced ground motions with peak particle velocity of 0.75 ips (19 mm/s). These limits are presented in Table 9.2.2.3. If a wall strain level equivalent to that produced by 1.0 ips (25 mm/s) particle velocity (measured in the ground) were chosen, the allowable overpressure would increase by 3 dB. Most cases of broken glass are reported to have been observed at air overpressures of 136 to 140 dB (as measured with a linear transducer).

Because of the different sound weighting scales that might be employed by monitoring instruments, the recommended levels in Table 9.2.2.3 differ by instrument system. Since structures are most sensitive to low frequency motions and the greatest air pressures occur at these inaudible frequencies, A-weighted scales cannot be employed at all. Since C-weighted scales are the least sensitive at low frequencies, their use requires the most restrictive limits.

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