Blast Induced Vibrations and Strain Rate Effects in Dynamic Capacity of Underground Concrete Structures

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ABSTRACT: When performing blasting operations in open pit mines, large amounts of energy is released and transmitted through the rock. The energy released can have significant impacts on mining operations and can adversely affect the mine production. This technical paper presents recommendations for blast design to prevent damage from blasting on underground concrete structures. Emphasis is placed in vibrations generated by blasting and its effects are studied referring to an existing hard rock mine expansion and asks whether production blasting would damage adjacent underground concrete structures due to blast stress waves. A set of criteria was developed to implement when blasting near-by underground structures. Knowledge of particle velocity and wave propagation theory for site-specific conditions to determine a safe level of vibration was recommended. The results suggest that an increase in structural capacity (dynamic capacity) is expected when structures are subjected to loads at very high strain rates, such as those of blasts.

Keywords: Blasting loads, wave propagation velocity, peak particle velocity, earthquake loads, dynamic capacity, underground structures.

1 INTRODUCTION

When expanding the limits of a surface operation, generally there are no geometric constraints if the slope design, operability, and ore to waste ratios allow for a safe and economically feasible operation. However, geometric constraints can lead to issues when blasting rock in proximity to near-by structures as blast waves produce stresses on the structures that structures may or may not be designed to sustain. The enhanced dynamic capacity of the structure when subjected to high strain rate loading allows an elevated ground vibration (PPV) or particle velocity as it is investigated in this paper through a case study.

2 CASE STUDY

This case study involves a hard rock open pit mine located in western USA. At this site, the northwest area of the pit was undergoing drilling explorations when high-grade deposit was identified. The mineral evaluation indicated that the upper portion was mostly low grade and waste and the profitable portion was at the bottom of the pit, at approximately 300 meters deep (4200 level). Initial concerns with the proposed expansion were funded on the fact that the final crest would be set at approximately 50 meters from the ventilation exhaust system at the surface. The cross section of the vent shaft consisted of a 5 m (15 ft) by 5 m (15 ft), 0.91 m (3 ft) thick with #6 reinforcing bars 6-in o.c. of reinforced concrete wall that extends underground 340 m (1245 ft). The vent shaft fan operates in normal conditions at 18,000 rpm and moves an air flow Q of 70,800 m^3/sec (2.5 million ft^3/min).

3 SITE CHARACTERIZATION

A signature blast was used to develop site specifc attenution curves in order to predict the response of the geology of the site to the loads. A total of seven seismographs were placed spaced accordingly and measurements peak particle velocity (PPV), frequency of vibration for a given scale distance were recorded. The signature blast involved a single blasthole 12-m (40-ft) deep and 241 mm (9.5 in.) in diameter with 5.18 m (17 ft) of stemming loaded with 270 kg (596 lb) of 94% ammonium nitrate (NH_4NO_3) and 6% fuel oil (FO) or ANFO explosive detonated about 350m from the shaft. The production blast consisted of a pattern designed with 300 blastholes loaded with an equivalent maximum charge weight of 1,632 kg (3,600 lb) of ANFO per 8-ms delay. The delay timing included 17 ms between holes and 34 ms between rows with same blasthole dimensions as the signature blast in terms of diameter, depth, and confinement. A summary of the recorded measurements is included in the table below:

Table 1. Blast data.

Seismo ID	Distance (m)	Scale Distance ($m/kg^{1/2}$)	PPV radial (mm/sec)	PPV Vertical (mm/sec)	PPV Transverse (mm/sec)	Max PPV (mm/sec)	Dominant Radial Frequency (Hz)	Dominant Vertical Frequency (Hz)	Dominant Transverse Frequency (Hz)
5118	50	3	79	58	50	79	5	12	5
5119	70	4	48	22	39	48	5	9	7
5120	120	7	41	13	22	41	6	10	8
4263	145	9	15	14	13	15	6	6	5
3029	220	13	6	4	6	6	5	10	5
3448	254	15	4	3	3	4	4	6	5
5121	313	19	2	2	2	2	4	4	6

The next step in analyzing the vibration levels was to obtain the site-specific attenuation curve. The scale distance was calculated using equation (1) below:

$$SD_2 = R / \sqrt{W} \qquad (1)$$

where: SD2 = square-root scale distance, m/kg1/2 (ft/lb1/2), R = distance to point of interest, m (ft), W=maximum charge weight per 8-ms delay, kb(lb).

Based on the recorded measurements, a regression analysis was conducted to obtain a characteristic curve. The regression analysis yielded the PPV attenuation equation described by Equation 2, as plotted in log-log scale chart (Figure 1).NOTE that Equation (2) represents the upper bound of an envelope that encompasses at least 95% of the data se for the signature hole blast.

$$PPV = 1860\ SD_2^{\ -1.99} \qquad (2)$$

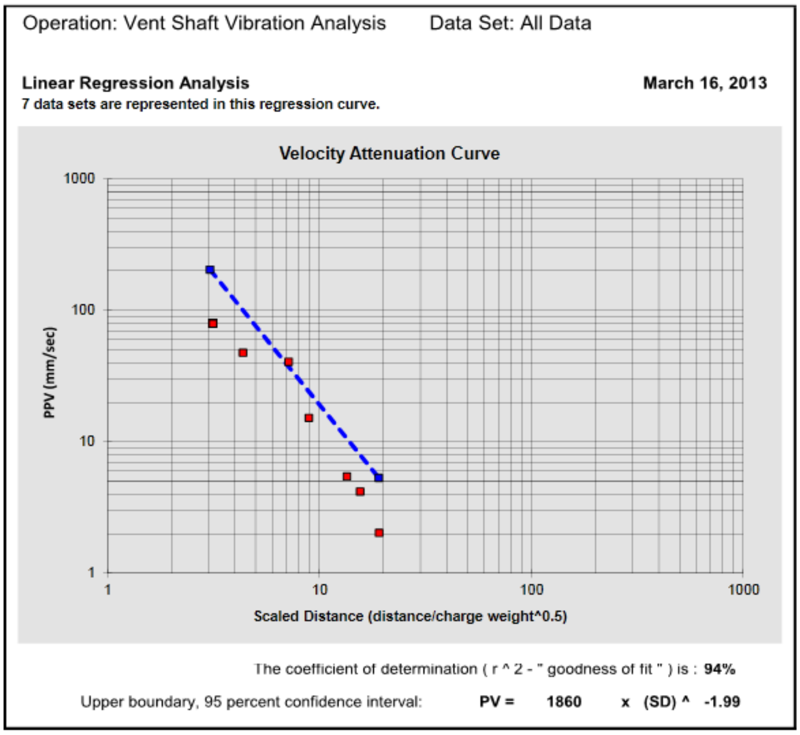

Figure 1. Site Attenuation Curve (PPV vs SD).

4 DAMAGE ANALYSIS

According to Equation (2), a PPV of 204 mm/sec of vibration is expected at the face of the shaft for a charge load of 270 Kg per delay, at a distance of 50 meters which is equivalent to a SD of 3. The blast wave is approximated to simple harmonic motion by equation (3):

$$u\,(x,t) = \sin\,(x-ct) \tag{3}$$

Since *u* is the displacement as defined by its position (x) at a particular time (t) and the velocity is defined as the change of displacement with respect to time. Hence:

$$\dot{u} = \frac{du}{dt} = \frac{d(\sin(x-ct))}{dt} = -\,c\,\cos\,(x-ct) \tag{4}$$

and therefore the ground strain can be defined as the change of displacement with respect to position,

$$\varepsilon = \frac{du}{dx} = \frac{d(\sin(x-ct))}{dx}$$
$$\varepsilon = \cos\,(x-ct) \tag{5}$$

by replacing Equation (4) in (5), the ground strain, ε, can then be expressed in function of particle velocity and wave propagation velocity (Equation 6). Note that particle velocity is different than wave propagation velocity; The particle velocity is the measured change in position of a particle in time. Whereas, the wave propagation velocity is an internal property of the material that depends on the stiffness and density of the material at which the elastic waves travel through.

$$\varepsilon = \dot{u}\,/\,-\,c \tag{6}$$

Where: $\dot{u}$ = particle velocity or PPV, m/sec (ft/sec). c = compressional (p-wave) propagation velocity of material, or Cp, m/sec (ft/sec), and ε = ground strain (με).

4.1 Static and Quasi-Static Properties of Concrete

Concrete properties can be obtained from the testing using ASTM standards. The stress-strain relationship in the elastic region is defined by Hook's law as; $\sigma = E\,\varepsilon$, where σ = stress, Mpa (psi); ε = strain ($\mu\varepsilon$), E = Young's modulus, Gpa (ksi). For instance:

For typical 20 Mpa concrete, Young's modulus is 22.5 x 10^3 Mpa. The concrete compressive strength is about 20 MPa and the tensile capacity is about 10% of that in compression, or 2.0 Mpa. Field data from the mine indicated that the rock Young's Modulus was 22.6 x10^3MPa. Therefore, a PPV of 204 mm/s would represent a wave stress σ_{ppv} of 2.58 Mpa:

$$\sigma_{ppv} = 22.6 \times 10^3 \times 114 \times 10^{-6} = 2.58 \text{ MPa}$$

This stress level does not exceed the concrete strength in compression of 20 Mpa. However, this stress level would exceed the concrete strength in tension of 2.0 Mpa.

Alternatively, if the quasi-static tensile strain at peak is used, and assuming linear behavior, the quasi-static strain, ε_S , is given by:

$$\varepsilon_S = \sigma_t / E \qquad (7)$$
$$= 6.5\sqrt{f'c} \,/\, 57000\sqrt{f'c} = 114\ \mu\varepsilon$$

Where: ε_S= quasi-static strain, $\mu\varepsilon$; σ_t = quasi-static tensile strength according to ASTM C496 tensile splitting test = 2- 5 Mpa (290- 725 psi); E = concrete Young's modulus, 20-40 Gpa (3000 – 6000 ksi); f'c = concrete compressive strength, 20-69 Mpa (3000 -8000 psi). Note that the quasi-static strain of concrete is based on ASTM C496 tensile splitting test which is normally conducted at a strain rate of 100-200 psi/min. Even though the quasi-static strain, ε_S, would not be exceeded by the ground strain generated by the blast, some cracking would start to develop under these quasi-static loading conditions. However, this conclusion was not sufficient since the ground strain produced by the PPV is not static strain but a dynamic one. Therefore, an analysis of dynamic strain was necessary to determine whether the structure's capacity would be exceeded.

4.2 Dynamic Properties of Concrete

Research on dynamic capacity of concrete under high strain rates by Malvar and Ross (1998) provided initial insights into dynamic increase factor (DIF). According to this study, the tensile dynamic strength essentially falls within static-quasi-static for strain rate loads of at 10^{-6} s^{-1}, but a sharp slope change occurs at strain rates of 10^{-0} s^{-1}or higher (1/3 slope in a log-log plot, Figure 2). This means that the concrete capacity is expected to increase significantly, up to factor of 10 when the structure is subjected to high strain rate loads. The equations were developed by Malvar and Ross (1998) and produced a modified formulation for tensile DIF described by equation (8), where: σ_{dyn} = dynamic tensile strength at $\dot{\varepsilon}$; σ_{ts} = static tensile strength at $\dot{\varepsilon}_S$; $\dot{\varepsilon}_S$= quasi-static strain rate, 1x10^{-6} s^{-1}; $\dot{\varepsilon}$ = strain rate in the range of 10^{-6} to 160 s^{-1}; log $\beta = 6\delta - 2$; ;$\delta = 1/(1+8f'_c/f'_{co})$; f'_c = static compressive strength, 30 – 70 Mpa (4350-10150 psi); f'_{co} = 10 Mpa (1450 psi).

Ngo et al (2007) developed strain rates associated to different types of loads. The loads associated with the ordinary quasi-static strain rate range from 10^{-6} to 10^{-5} s^{-1}, the earthquake loads produce seismic strain rates in the range of 10^{-3} to 10^{-1}, the impact strain rates range from 10^0 to 10^2, while blast loads yield loads associated with strain rates in the range of 10^2 to 10^4 s^{-1}.

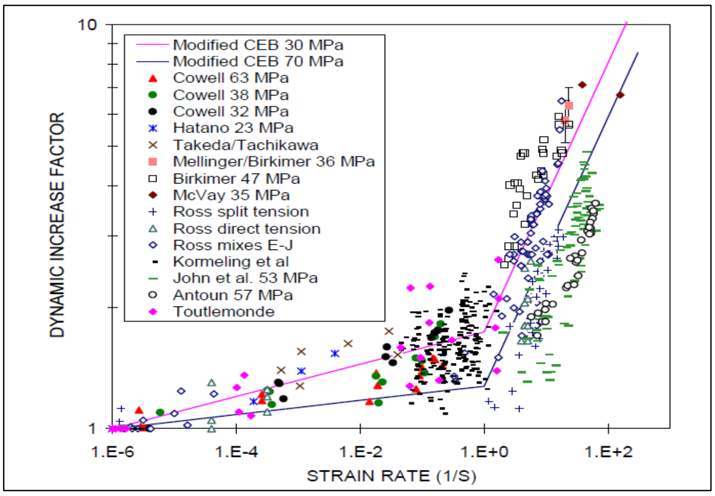

Figure 2. Dynamic Increase Factor (DIF) vs Strain Rate (Malvar and Ross, 1998).

$$DIF = \sigma_{dyn} / \sigma_{ts} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{\delta} \quad \text{for} \quad \dot{\varepsilon} \le 1\ s^{-1} \qquad (8)$$
$$= \beta \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s}\right)^{1/3} \text{for} \quad \dot{\varepsilon} \ge 1\ s^{-1}$$

The strain rate of the blast, $\dot{\varepsilon}$, is 10^2 s^{-1}, the corresponding DIF from Figure 2 would be approximately 4. By increasing the quasi-static tensile strength of concrete four times, the dynamic tensile strength becomes 8.0 Mpa. Hence:

σ_{dyn} = 4 x 2.0 Mpa = 8.0 Mpa ≥ σ_{ppv} = 2.58 Mpa => OK

4.3 Structural Approach

Criteria for structural design requires that all structural members and sections must be proportioned to meet demand under the most critical load combinations for all possible actions (flexural, axial, and shear), $\emptyset M_n \geq M_u$ and $\emptyset \mathbf{V}_n \geq V_u$ respectively. The dynamic stress, σ_{ppv}, of 2.58 Mpa produced by the PPV was considered acting along the face of the shaft wall normal, the resulting load, w_u, was then estimated per linear meter (ft). Thus: $w_u = \sigma_{ppv}$ x 1 m = 2510 KN/m/m (54 Kip/ft/ft). In addition, since the incident stress is acting on the front face of the vent shaft, a fixed-fixed condition was assumed to represent the condition at the shaft wall face. For a fixed-fixed end condition, the maximum demand loads are given by: $M_{u\ (center)} = w\,l^2/24$; 248 KN-m (183 Kip-ft), $\mathbf{M_{max\ (ends)}} = \mathbf{w\,l^2/12}$ **= 494 KN-m (365 Kip-ft)**, $V_u = V_{max\ (ends)} = w\,l/2$ **= 1,080 KN (243 Kip).**

The increased structural capacities are given by: ϕMn = ϕ [As fy (d-a/2)] = 0.9 [18 X 0.88 x 60,000 x (33-13.2/2)]xDIF=**10,200 KN-m** (1,881 K-ft x 4); and ϕVn = ϕ (Vc+Vs) =ϕ ($2\sqrt{f'c}$ bw d + $A_v f_y d/s$) = ($2*\sqrt{5000}$*12*33+2*0.2in^2*60*33/6)xDIF=**2,845 KN** (160 Kip x 4). Since moment and shear capacities are greater than the demand loads, structural failure would not occur.

4.4 Stress Transmission at the Interface

If the stress at the interface between two different materials (rock/concreteor rock/air), is analyzed, reflection and transmission of the stress wave at normal incidence must be conservation of mass, momentum, and energy. These requirements translate to dynamic equilibrium that is consistent with elastic behavior (Pariseau, 2012). Thus:

$$\sigma_I + \sigma_R = \sigma_T \qquad (9)$$

where: $\sigma_R = (\rho_2c_2-\rho_1c_1)/(\rho_2c_2+\rho_1c_1)\,\sigma_I$, $\sigma_T = (2\rho_2c_2)/(\rho_2c_2+\rho_1c_1)\,\sigma_I$, $\sigma_R/\sigma_T = (\rho_2c_2-\rho_1c_1)/(\rho_2c_2+\rho_1c_1)$. In addition, at the rock-concrete wall interface, the principle of conservation of momentum (Newton 2nd Law) applies: $F = m\,\ddot{u}$; Thus: $\Delta\sigma$ A = (ρ Δx A) $\frac{d(\dot{d}u}{dt(dt)})$, canceling out the areas and with $\Delta x / \Delta t = C_p$ and $\frac{du}{dt}$ = peak particle velocity, one has wave stress given by:

$$\sigma_{ppv} = \rho\, C_p\, PPV \tag{10}$$

where $\rho\, C_p$ = density x p-wave propagation velocity (also known as acoustic impedance); PPV = peak particle velocity.

Note that the acoustic impedance in the above expression can be demonstrated by the principles of wave transmission and reflection at the interface. According to the theory of longitudinal impact of prismatic elastic bar, the stress waves travel axially at a velocity of, $C_p = \frac{\sqrt{E}}{\rho}$, where E is the modulus of elasticity and ρ is the mass density of the bar material. The seismic velocity C_p is constant for a given solid elastic material, and the longitudinal stress in the bar at any point is related to the particle velocity (υ) at that point by the expression $\sigma = \rho\, C_p\, \upsilon$ (Peck, 1974). Furthermore, the transmission and reflection of waves depends directly on the impedances of the materials; The mismatch of impedances between materials will dictate the magnitude of the transmitted stress. Thus, full reflection would occur at the concrete/air interface. Using equation (9) with concrete properties of , ρ_2 = 2,850 kg/m^3, and C_{p2} = 2,850 m/sec gives: σ_I (σ_{ppv})= 2.0Mpa + 0.57 Mpa=2.58 Mpa . Then comparing stresses, σ_T = 2.0 < DIF X 2.0 = 8 Mpa (σ_{dyn}). Therefore, failure in tension (or spalling) would not occur. Based on the above, an expression for the maximum PPV_{max} was then derived as follows:

$$PPV_{max} = DIF^* \sigma_t / E^* C_p \tag{11}$$

Where: $PPV_{max=}$ maximum peak particle velocity, mm/sec (in./sec), C_p = p-wave propagation velocity of rock, m/sec (ft/sec), σ_t = quasi-static tensile strength of the material (concrete), Mpa (psi), E = Young's modulus of the material (concrete), GPa (ksi), DIF = Dynamic Increase Factor from Eq. (8) or Figure 2.

5 CONCLUSIONS AND RECOMMENDATIONS

Based on the site-specific attenuation curves obtained for this site, this study suggests that a blast load of 270 kg (596 lb) would not undermine the integrity of the vent shaft. This finding was based on the following conclusions:

Blasting near the shaft structure would exceed static tensile strength of the concrete under static loading. However, based on the increase in dynamic capacity when concrete and steel are subjected to high strain rate loads, these materials can increase their capacity up to 600% in tension. Therefore, failure in tension is not expected.

At higher strain rates, microcracks will not have time to develop laterally into regions of lower strength and will instead follow a more direct path through stronger regions.

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