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Blasting vibration assessment of rock slopes and a case study

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Abstract

The most common types of blasting damage are caused by ground vibration. The sudden acceleration of the rock by the detonation energy acting on the drill hole generates an intense stress wave of both transverse and longitudinal wave motions in the surrounding rock. Key issues associated with the process of excavation and tunnelling include blast and, to a lesser extent, other construction vibration that affects the integrity of the surface structure and potentially slope stability.

The stability of slopes subject to blasting induced ground vibration may be assessed by different approaches including the pseudo-static approach, the dynamic analysis, the empirical approach and the energy approach. For soil slope stability analysis, the Pseudo-static Approach and Dynamic Analysis are generally used. However, for rock slope stability assessment under blasting vibration effects, an energy approach is normally used.

The energy approach, combined with the empirical correlation of shear strength and stiffness of rock joints developed by Barton (1990), with various joint characteristics, has been used in the analysis described in this paper. Peak particle velocity (PPV) of the potential rock block failure is a key parameter to determine the allowable charge weight per delay of the blast. Detailed discussion of the energy approach is presented in this paper and a case study illustrates the use of the method.

The allowable charge weights per delay for rock blasting which may impact on the stability of slopes can be estimated using the simple energy approach. This approach can give controllable safety limits for the works. Thus, blasting works can be carried out safely with minimum to no damage or excessive ground movements to the slopes and other sensitive receivers, if the allowable PPV and charge weights are followed, and the specified monitoring works are carried out.

1 Introduction

The most common types of blasting damage are caused by ground vibration. The sudden acceleration of the rock by the detonation energy acting on the drill hole generates an intense stress wave of both transverse and longitudinal wave motions in the surrounding rock. These wave motions are called ground vibration. The blasting effects include both the direct impact of blasting induced ground vibration, and the potential secondary effects of environmental issues such as noise and dust. Key issues associated with the process of excavation include blast and other construction vibration affecting the integrity of buildings, structures, services, and geotechnical features such as slopes and retaining walls. To prevent property damage and complaints due to nuisance to neighbouring properties, much of the proposed excavation works by means of blasting are required to assess the vibration effects.

To maintain the safety and integrity of the sensitive receivers under vibration effects, blasting assessment should be conducted to demonstrate a maximum allowable vibration limit and a maximum charge weights. The assessment results provide guidance to excavation works blasting is not to exceed the predicted (or permissible) vibration limit, and is not to damage the slope and sensitivity receivers.

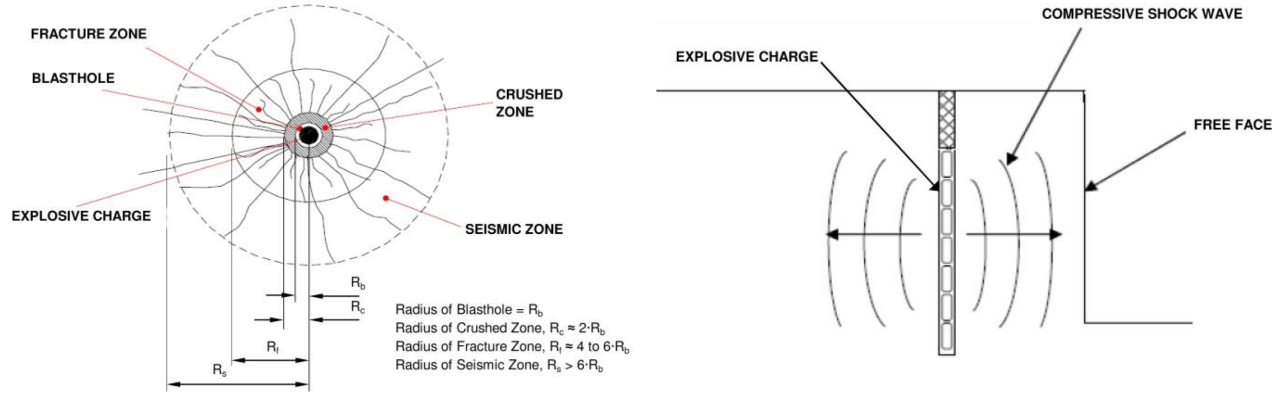
In this paper, the energy approach is adopted, which may be used in the early risk assessment for the analysis of dynamic response, i.e. ground vibration against slope stability.

1.1 Vibration limit

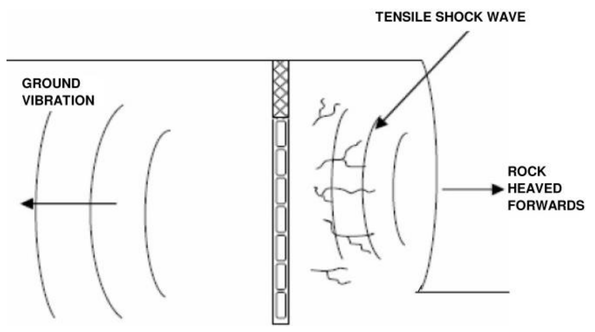
With regard to the blasting assessment, peak particle velocity (PPV) is one of the widely used indexes of damage to structures. The USACE (1972) study indicated that structures in good condition can readily tolerate a peak particle velocity (PPV) of 50 mm/sec with no damage reported. However, different ground conditions or geological formations will be the amplification of the peak ground motion in order to affect the sensitivity of vibration effects to structures or geotechnical features. Therefore a PPV of the ground motion is often set to 25 mm/sec to prevent damage to buildings or geotechnical features. The vibration criterion, i.e. 25 mm/sec, is specified by the Hong Kong General Specification for Civil Engineering Works (HKG, 2006) and by the Australian Standard (2006) AS2187.2-2006.

2 Blasting mechanism

The transfer of energy from explosive charges to the ground mass, i.e. in homogeneous ground and the subsequent propagation of the stress waves involve some complex processes which are usually described in terms of wave motion. The wave motion of blasting is outlined in Figure 1.



(a) (b)



(c)

Basic principles of blasting:

- Confine energy within blast-hole to get efficient transfer into rock mass (a).
- Compressive wave transmitted through rock mass (b).
- Compressive wave reflected as tensile wave from free are surface (c).
- Free face (or surface) could be a blast face, ground surface, or relief hole(s) (b).
- Gas will penetrate the cracks to enlarge them and heave the rock (c).

Figure 1 Principle of wave motion of blasting (after Kutter and Fairhurst, 1971)

When an explosive detonates it undergoes an almost instantaneous change of state, i.e. detonation reaction in the explosive from a solid to a gas. This sudden change of state within a confined space produces very high energy to clash the borehole wall, which gives rise to the shock/stress wave in the surrounding rock mass. The extreme pressures exerted by the gas may exceed 1 GPa. Stress waves are produced causing very localised crushing and fracturing in the ground. Displacement occurs immediately in all directions, but will clearly be less in those directions where the confining forces are greatest. In those directions, where a free face exists or where the burden is less, bulk movement will occur. For heterogeneous rock mass, ETI (1980) gave detailed discussion on rock movements under blasting.

Outside the immediate vicinity of the blasting permanent deformation is very unlikely (Figure 1). The rapidly decaying stress waves cause the ground to exhibit elastic properties whereby the ground returns to its original position as the stress wave passes. The stress wave motion in the ground spreads out concentrically from the blast site, particularly along the ground surface. The energy of the motion attenuates as it moves further from the origin, as the initial energy is spread over an ever-increasing volume of material.

3 Slope instability under vibrations

For soil slope stability analysis, the conventional limit equilibrium methods (Bishop, 1955; Janbu, 1972; Morgenstern and Price, 1965) used in geotechnical practice are to investigate the equilibrium of a soil mass tending to move downslope under the influence of gravity. The methods allow for the comparison between forces and moments tending to cause instability of the mass, and those that resist instability. In addition to slope stability assessment subject to blasting vibration, conventional limit equilibrium methods may be combined with a number of seismic analysis approaches such as the pseudo-static approach or dynamic analysis approaches (Kong, 2012).

When a rock slope is subject to seismic shaking, failure does not necessarily occur when the dynamic transient stress reaches the shear strength of the rock. Furthermore, if the Factor of Safety (FS) on a potential sliding plane or surface drops below unity (i.e. FS <1.0), at some time during the ground motion, it does not necessarily imply a dramatic failure issue. The excessive driving force of the ground motion (in terms of displacement or peak ground acceleration) permits the FS to drop well below unity. The rock block will then be unstable or slide accordingly. This phenomenon has been discussed in Newmark (1965) to determine the permanent displacement of failed ground mass (rock and soil) on slopes as the result of earthquake motions.

In critical situations, it may also be advisable to check the sensitivity of the slope to seismic deformations using Newmark analysis. The Factor of Safety of a plane failure (without tension crack) can be written as:

$$FS = \frac{c' \cdot A + [\, W \cdot (\cos\psi_p - \alpha \cdot \sin\psi_p) - U + T \cdot \cos\theta] \cdot \tan\phi}{W \cdot (\sin\psi_p + \alpha \cdot \cos\psi_p) - T \cdot \sin\theta} \tag{1}$$

Where:

c' = cohesion of rock joint (kPa).

A = contact area of failure plane (m^2 or m/m-run in 2D analysis).

W = weight of sliding block (kN).

ψ_p = failure plane angle.

ϕ = friction angle of rock joint.

θ = Inclination of T (external load such as rockbolt) to normal of failure plane.

T = external force (i.e. rockbolt) (kN).

α = peak horizontal ground acceleration in term of g (gravity acceleration) (m/s^2).

U = water pressure acting on failure plane.

The term α, peak horizontal ground acceleration is interpreted as a ground motion during a seismic event. However, this ground motion is hard to determine and to analyse of the dynamics response of slopes (for both soil and rock slopes) in terms of vibration limits under the blasting events.

4 Energy approach

Lucca (2003) and ETI (1980) discussed that large ground accelerations are often induced during blasting (e.g. peak particle accelerations, (PPA) of 1 g or higher have been recorded). However, despite large PPAs, blasting pulses often possess relatively low vibration energy. The vibration energy and PPA in the rock block system under blasting can be modelled relative to the charge weight of explosive (per delay), blasting source distance and rock conditions.

The energy approach tackles the problem by considering the blasting vibration energy transmitted to the potential failure rock block resting on the rock slope. This vibration energy also dissipates at the rock joint (the failure plane), as distance from the blast source increases. The stability and the downslope displacement (the drag force) of the rock block are assessed using equations based on the principle of conservation of energy. This approach, combined with the empirical formulae developed by Barton (1990) correlated the shear strength and stiffness of rock joints with various joint characteristics used in the analysis.

Wong and Pang (1992) have discussed the energy approach in which simplicity of consideration as a rock block rests on an inclined plane subjected to blasting vibration. The total energy of this rock block system subjected to blasting vibration (Figure 2) will be modelled to consist of two parts: (1) potential energy of the rock block; and (2) kinetic energy (or vibration energy) to the rock block.

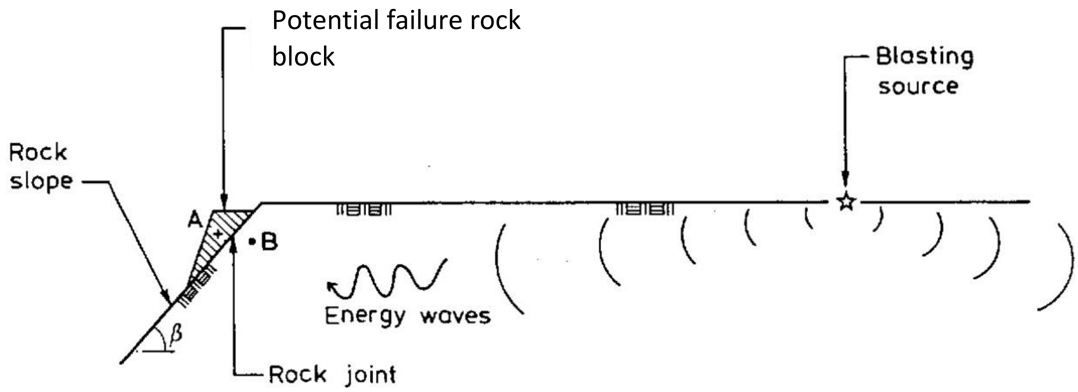


Figure 2 Rock block system subjected to blasting vibration (Wong and Pang, 1992)

By considering the principle of conservation of energy, the following equation can be written for the rock block system under vibrating motion.

$$\left(\begin{array}{c}\text{Total energy of rock}\\\text{block system at rest}\end{array}\right) = \left(\begin{array}{c}\text{Total energy of rock block}\\\text{system under vibration}\end{array}\right) + \left(\begin{array}{c}\text{Energy loss at the}\\\text{boundaries}\end{array}\right) \quad (2)$$

$$\text{Energy Loss} = \tfrac{1}{2}\cdot(W/g)\cdot V^2 + W\cdot\Delta u\cdot\sin\beta \quad (3)$$

Where:

W = weight of rock block.

g = ground acceleration.

V = peak resultant velocity of rock block.

Δu = displacement (move downslope) of rock block.

β = failure plane angle.

As the linear shear displacement model was considered in the rock block vibration system and combined with Barton (1990) rock joint models, the Wong and Pang (1992) study showed that the critical PPV, at which the block will be driven to a state whereby peak shear stress (i.e. FS drops to unity) is developed at the rock joint, and the equation can be expressed:

$$\text{PPVc} = \sqrt{\left[\frac{g}{0.91}\cdot\delta_p\cdot\sin\beta\cdot\left(\frac{\tan\varnothing_p'}{2\cdot\tan\beta}+\frac{\tan\beta}{2\cdot\tan\varnothing_p'}-1\right)\right]} \quad (4)$$

Where:

g	=	ground acceleration.
β	=	failure plane angle.
δ_p	=	weight of rock block.
ϕ'_p	=	displacement (move downslope) of rock block.

The terms δ_p and ϕ'_p (Equation 4) are interpreted in the empirical formulae given by Barton (1990). They are:

$$\phi'_p = JRC \cdot \log (JCS / \sigma'_n) + \phi'_r + i \quad (5)$$

$$\delta_p = [L / 500] [JRC / L]^{0.33} \quad (6)$$

Where:

JRC	=	joint surface roughness coefficient (as defined in Barton, 1990).
JCS	=	joint wall compressive strength (as defined in Barton, 1990).
σ'_n	=	normal stress of failure block.
L	=	joint length.
ϕ'_r	=	residual angle of shear resistance of joint.
i	=	roughness component of shearing resistance expressed as an angle.

(note: $\phi'_r + i$ can be defined as designed friction angle of rock joint).

To apply Equation 4, the field inspection of rock joints is important. A field study includes rock face mapping recording, the joint characters and the nature of infilling to determine the of designed friction angle the rock joint.

5 Ground vibration prediction

It is observed that ground vibration induced by blasting has a peak velocity that is related to the instantaneous charge weight and the distance from the blast source. A preliminary assessment of the ground vibrations likely to result from blasting can be made using the formula derived by the United States Bureau of Mines (ETI, 1980).

$$PPV = K (R / \sqrt{W})^{-B} \quad (7)$$

Where:

PPV	=	peak particle velocity (mm/s).
K	=	ground transmission constant.
R	=	distance between blast and measuring point (m).
W	=	maximum charge weight per delay interval (kg).
B	=	attenuation exponent.

Due to variations of ground conditions, type of explosives used and type of blasting, Equation 7 has different responses within the ground. The attenuation relationship for surface and underground confined blasting is known to be different. For instance, Australian Standard AS2187.2-2006 recommended average conditions of PPV are:

$$PPV = 1140 \times \left(\frac{R}{\sqrt{W}}\right)^{-1.6} \quad (8)$$

Variations to the constants K and B in Equation 7 may be necessary to take into account the ground conditions of the blast zone. The ground mass (or soil materials) causes significantly larger attenuations from vibrations than hard rock, and therefore the predicted PPV values may not be indicative of actual conditions. Observations made during trial blasts at the early parts of the construction can be used to derive site specific parameters to more accurately represent site-specific conditions, which increase certainty of prediction and can optimise production. In addition, the trial blasts will also verify the constants being adopted for the blasting vibration. In general practice at least several tens of numbers of blasts are required to be monitored with reliable data before the initial constants used for blasting are revised.

6 Case study

A quarry, after a total production of 14.7 million tonnes of granite rock products in which the majority of aggregates products are supplied to the local construction industry, was proposed to be rehabilitated and be transformed as a park-and-water recreation centre. This quarry site was located on the north side of Sok Kwu Wan of Lamma Island in Hong Kong, covering an area of 49 hectares, with about 1 km of coastline. An aerial view the quarry site is shown in Figure 3.

The proposed quarry rehabilitation works included:

- Formation of a new landform comprising a broad, gently sloping series of platforms bounded to the north with slopes which merge with the natural hillsides.
- Formation of a 4 hectares man-made lake with a natural, non-engineered appearance and gently shelving edges which can support reed bed.
- Landscaping and planting of exotic and native trees to create a self-sustaining, maintenance-free, green environment with a similar biodiversity to the surroundings.

Under the rehabilitation scheme, the proposed man-made lake was developed from the original quarry site's sump pit. A 4.2 m high and 4 m wide spillway was constructed in close proximity to the existing sea shore. An aerial view of the rehabilitated quarry site is shown in Figure 4. During the rehabilitation period earthworks and normal quarrying activities were undertaken, including blasting for rock excavation.

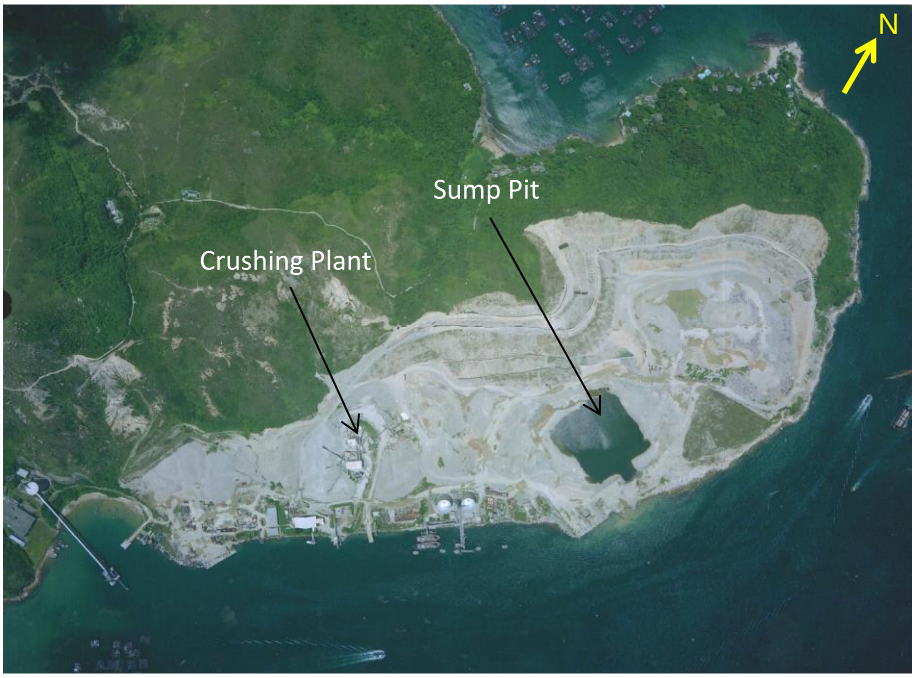


Figure 3 Aerial view of the quarry site (in operation)

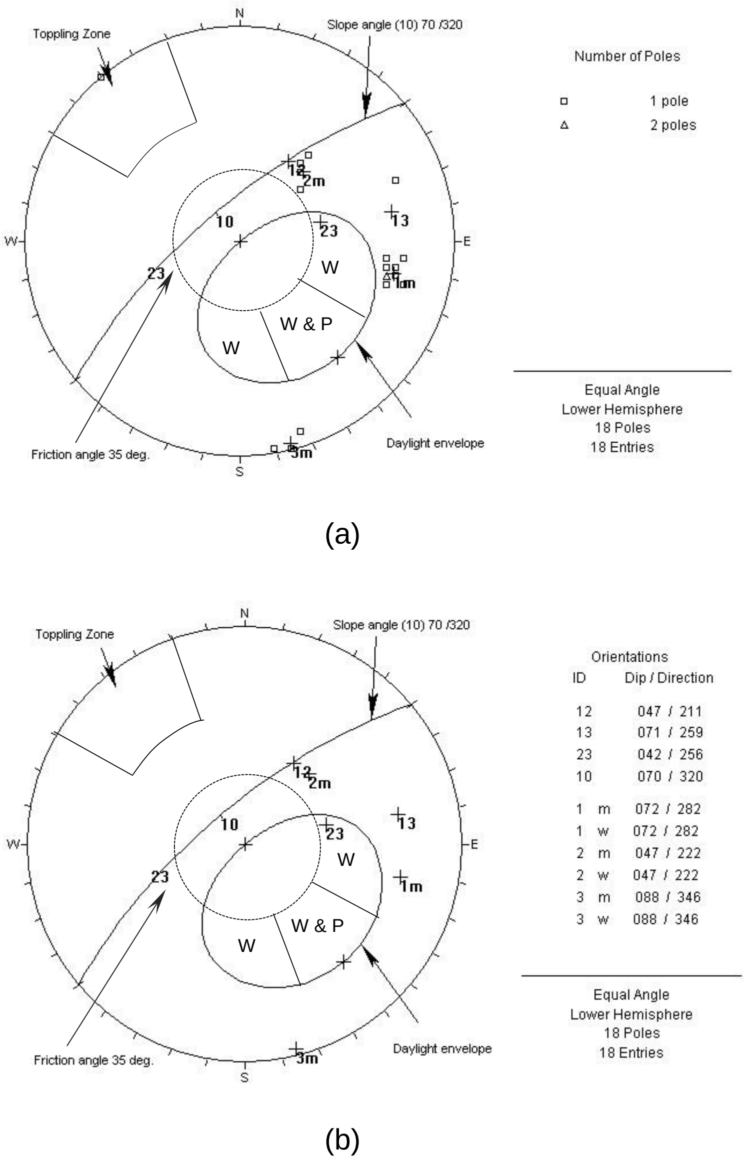
Figure 4 Aerial view of the quarry site after rehabilitation

For the construction of the spillway, an in situ rock bank foundation was formed. The final rock face of the bank was to be protected to avoid further damage by blasting activities during rehabilitation period, thus, control blasting was implemented. The final rock face of the bank is a 4 m high sub-vertical slope face of 70° offset 10.5 m away from the spillway foot-print and faces towards the man-made lake.

Figure 5 Final inspection (rock mapping) photo record of the rock face (partial view)

Rock mapping to face comprised subjective (biased) discontinuity surveying whereby visually significant joints of 1 m persistence or greater that may provide potential instability were mapped. The mapped rock joint data had been processed using stereographic hemispherical projection methods, and analysed for the face slope orientation by kinematic stability checks. A total of 18 rock joint data were taken at the final rock face, partial photographical record is shown in Figure 5. A basic friction angle of 30° plus 5° for roughness (as described rough undulated joint surface), which had been employed for surface stained and/or clean moderately decomposed or better granite (Brand et al., 1983), has been adopted in the checking of stability of the final slope excavation. The results from kinematic analysis are shown in Figure 6.

Referring to the kinematic analysis (Figure 6 (b)), only joint intersection 23 (intersected by joint set 2 and joint set 3) lies within the window of daylight envelope, indicating that wedge failure was kinematically feasible. The field mapping record confirmed that a potential unstable wedge failure block of about 2 m high located at the middle top of the rock bank was identified. This block was a blasting vibration sensitive subject. Other location rock face confirmed no instability blocks, were observed. A detailed assessment and remedial measure proposal were required for this potential unstable wedge block.



(a)

(b)

Figure 6 Kinematic analysis to the rock face, (a) pole plot; (b) major planes and its intersection, 'W' and 'P' represent wedge and planar failure zones respectively

Based on field inspection, rock joint characters and laboratory testing results, the UCS of granite is greater than 150 MPa; JCS of 75 MPa, JRC of 9, and L = 3 m (as block height is 2 m and sliding angle of the intersection '23' of 42 degrees) were also interpreted. Using Equation 4, the calculated PPV was 9.9 mm/sec for the identified potential wedge failure block. When adopting the agreed site specific blasting equation (i.e. Equation 7, PPV = 644 × $W^{0.61}$ × $R^{-1.22}$), the governed maximum charge weight of explosive per delay at various distances is shown in Figure 7.

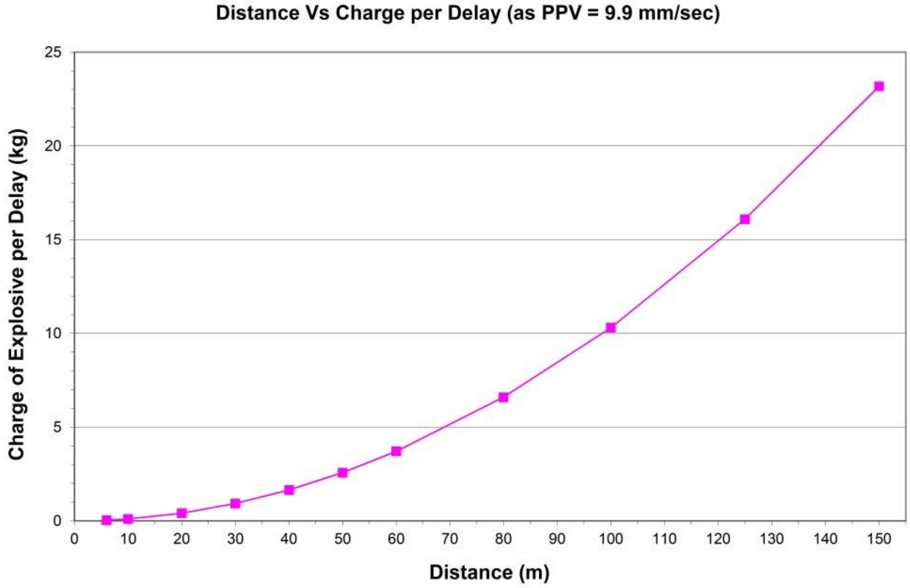


Figure 7 Guidance chat of blasting source distance against charge weight of explosives

With the guidance of Figure 7, the initial 15 trial blasts were conducted at various distances, i.e. radial separation between the subject block and the blasting source, with the allowable charge weight of explosive. The vibration monitoring records demonstrated that the PPVs were recorded less than the permissible PPVc of 9.9 mm/sec and had only of about 25 to 80% of the estimated value. The potential unstable wedge block remained in place.

Having considered the overall rock face conditions, trial blast monitoring record and rock excavation production rate, the construction team decided to remove this potential unstable wedge failure block. A new vibration limit of 25 mm/sec, as specified by the Hong Kong Government's General Specification for Civil Engineering Works (HKG, 2006) was adopted. By doing so, the production rate was increased in order to allow for early completion of the project. After completion of all excavation works by blasting, final inspection to the rock face confirmed that the slope was still stable and no loose blocks were observed.

7 Discussion on case study

The energy approach and this case study provide good reasons to believe that this approach can provide a realistic performance prediction of rock slope. Where a potential unsafe rock condition is anticipated under blasting effects, the need for stabilisation works should be considered.

In any blasting event to maintain a safe manner to the surroundings, PPV for each critical sensitive receiver and slope should not be allowed to exceed the permissible PPV. Field monitoring is recommended to check the actual level of vibration of the potential unstable rock blocks caused by blasting. This would provide the data required for the improvement of blast design, as well as of methods for assessing rock slope stability.

Energy approach is relatively simple. It involves only routine techniques for rock mapping, rock joint analysis and blast control techniques which include the measurement of PPV, and the use of the available attenuation laws relating PPV with scale distance and site factors. However, the approach discussed here is applicable for rock slopes only. Soil slope stability assessment under vibration effects is not covered in this paper. In addition, due to high variation of determining rock joint parameters, particularly of basic friction angle, JCS and JRC, laboratory testing on rock joints and site trial blast should be conducted to verify the analysis.

In this case study, the result obtained by the energy approach cannot compare to the pseudo-static approach as they are based on entirely different consideration of loading conditions and ground mass model. Further study and discussion to compare those approaches will not be given in this paper.

8 Conclusion

Blasting works can be carried out safely with no damage or excessive ground movements to the slopes and other sensitive receivers (e.g. structures, building and services) if the allowable charge weights are followed and the specified monitoring works are carried out. The allowable charge weights per delay for blasting excavation of tunnelling (or open pit excavation) can be estimated using the energy approach.

Methods for assessing the stability of rock slopes subjected to blasting vibration using the energy approach, taking dynamic slope response into consideration, has been outlined in Sections 4 and 5. Guidelines for applying the values of PPV assessed by the methods to blast control are described. Although the approach is considered conservative, the estimated PPV values still lie within controllable safety limits.

The presented case study demonstrated that the monitored PPVs of the blasting works had only 25 to 80% of the estimated value when using the energy approach for rock slope stability assessment. The vibration monitoring records also showed that all blasts undertaken were a controllable manner less than the assessed permissible PPV.

Early risk planning including the proposal of blasting containment and control measures should be carried out. Planning ensures the blasting works be carried out in a safe manner, particularly for governing the integrity of slope, in order to minimise the impact on the surrounding environment. The permissible PPV obtained from a blasting risk assessment for each feature and structures should be documented in the project's performance requirements of the construction contract.

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