27 Apr 1981, 10:30 am - 1:00 pm

In-situ Measurements of Blast Seismic Waves and Their Safety Distance

Rui-geng Zhu
Institute of Rock & Soil Mechanics, Academia Sinica, Wuhan, China

Li Zhen
Institute of Rock & Soil Mechanics, Academia Sinica, Wuhan, China

Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

In-situ Measurements of Blast Seismic Waves and Their Safety Distance

Zhu Rui-geng, Assoc. Professor & Li Zhen, Engineer
Institute of Rock & Soil Mechanics, Academia Sinica, Wuhan, China

ABSTRACT:
Empirical formulae and their applications are given after systematically analysing the seismic wave data as measured in various blasting operations. The computed results are in close agreement with the field measured data

Evaluating stability of tunnel and underground gallery under dynamic loading and determing safe operating distance from blast source are often required in mining, railway tunnelling and hydroelectrical engineering etc. Such dynamic loadings are mainly from blasting operations. In order to do that, it is necessary to consider the dynamic loading effect of earth blast waves in addition to static loading effect of rock and soil themselves. The intensity of dynamic loading relates to the amplitude of the blast wave, the loading time, etc.[2]

I. Propagation of earth blast wave

Propagation and attenuation of blast seismic wave in earth media depend on the conditions of blast source, topography, inhomogeneity, and geological features, e.g. joints, fault, etc. The results of mathematical calculation, therefore, often do not tally with the actual situations. Currently in engineering practice, a lot of field measured data is still used as the basis on which empirical formulae are deduced.

The particle vibrating velocity is usually used as the criterion for determining stability of structures in engineering practice. In China vertical vibrating velocity V is a function of scaled distance ($Q^{1/3}/R$) shown as

$$\left(\frac{V}{C_e}\right) = K\left(\frac{Q^{1/3}}{R}\right)^{\alpha} \qquad (1)$$

where:

K--coefficient related to the properties, explosive charge and type of blasting
Q--weight of spherical charge,T.N.T. kg
R--distance from blast centre to measuring point m
α--attenuation index related to geotechnical properties of media
C_e--longitudinal elastic wave velocity cm/sec

Following problems should be taken into account in studying the propagation laws of blast wave:

1) Shock compressional seismic wave produced by ground-surface blast

For surface blast, after blast seismic wave propagations, shock compressional seismic wave is then produced due to impact of air shock wave against the ground surface. Fig. 1 shows the wave forms actually measured. The sustaining time of the high-frequency compressional seismic wave is equal to that of the air shock wave. The time interval between shock wave and the blast seismic wave can be determined by following equation,

$$t_s = \frac{R}{C_1} - \frac{R}{C_e} \qquad (2)$$

(C_1--wave front velocity of air shock wave, m/sec).

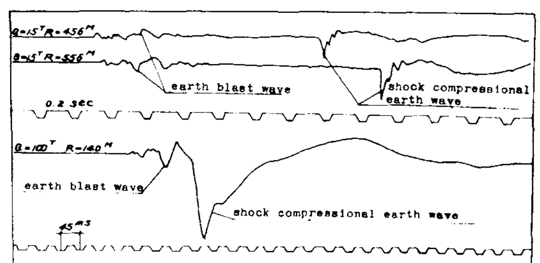

Fig.1 Field measured wave forms

Fig. 1 Field measured wave forms

The vertical vibrating velocity of compressional seismic waves V_{vert} is calculated from a series of field data obtained from surface blasts (Q = 1t, 5t, 15t, 100t) by

$$V_{vert} = 276(Q^{1/3}/R)^{1.69} \qquad (3)$$

2) Effect of topography

The influence of local topographical feature on blast effect is evident. The seismic effect decreases in gully, gallery and excavated areas while it increases in slopes or rock pillars. The change of topography can be approximately indicated by change in elevations. From the field measured data it can be seen that for an elevation of 25-104 m the particle acceleation in rock increases by a factor of 1.23-3.04 in horizontal direction and by 3.26-3.80 in vertical direction. Within the top soil it increases by 1.18-1.53 times horizontally and by 1.31-1.79

times vertically.(1)

3) Propagation characteristics of particle velocity in earth media

Analyses were made for vertical vibration velocities (V_{vert}) in earth media from extensive data obtained from many surface blasts, open-pit mining explosions and blasting operations in tunnel openings. The empirical formulae showed in Tab. 1 were then derived for various different conditions. The curves relating vertical velocity V_{vert} to scaled distance ($Q^{1/3}/R$) are shown in Fig. 2 and 3.

Table I Empirical Formulae of Geotechnical Particle Vertical Vibrating Velocity Affected by Blast Seismic Wave

Type of Blast	Blast Condition & Charging Amount	Geological Condition	Empirical Formulae $V_{vert} = K\left(\frac{Q^{1/3}}{R}\right)^{\alpha}$ K	α
1 Surface Blast	Charge concentrated Q=1,3,5, 10,15,40,100 T	Granite	98.76	1.37
2 Open-air Blast				
1)Loose Blast	A.One delay-action Charge Q=9320 T	Diabase	84	2.42
		"	630	2.80
		"	206.4	1.81
	B. Instantaneous blast total Charging Q=1000 T	Metamorphic Rock	180	1.47 1.39
	C. " Q=534 T	Phyllite	82.5	1.32
	D. " Q=20 T	Granite & Marble	150	2.00
	E. " Q=20 T	Phyllite Marble	158	1.93
2)High-bench Instantaneous Blast	A.Instantaneous Q=200 T	Limestone	77.6	2.33
	B. " Q=103 T	Quartzite	624	2.41
	C. "	Limestone	130	1.80
3)High-bench Short-delay Blast	A.6-section Short-delay Q=45.9 T	Gneisses	180	1.83
	B.10-section Short-delay Q=4.23 T	Marble	378	1.60
	C. "	Quartzite	142	1.61
	"	"	153	1.60
4)Oriented Blast	A. Oriented Blast Q=1394 T	Sandstone	240	2.0
	B. " Q=503 T	Diabase	115	2.0
3 Blast Inside Tunnel	Linear Charging l/d (Pin-Point Blast) l--charge length d--charge radius Q=8-150 T	Granite	99.6	1.72
		Granite	111.2	1.92
		Granite	591.4	1.75
		Granite	92	1.83

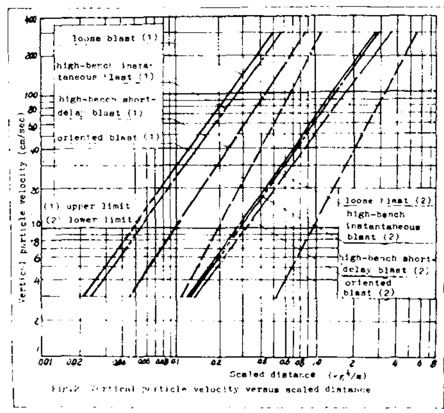

Fig.2 Vertical Partcle Velocity Versus Scaled Distance

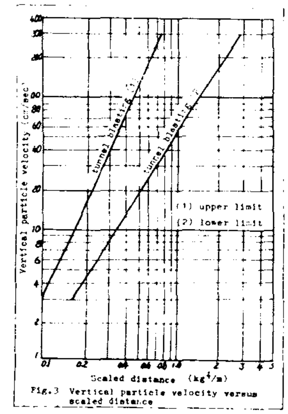

Fig.3 Vertical Particle Velocity Versus Scaled Distance

2. Dynamic strength of rock

It is shown by the experiments that under dynamic loading the larger the loading rate, the higher the rock strength will be. The increased value depends on rock properties and loading rate. Tab. II shows the increase of compressive strength at different loading rates for granite, marble and rock salt. It can be seen from the table II that by increasing the loading rate, the compressive strengths of rocks with different mechanical properties are all increased. But under the same order of magnitude of rates the increased values are different. With increase of loading rate the compressive strength, tensile strength and dynamic elastic modulus are all increased as functions of logarithm of loading rate. The general expression is given in [1] [3].

Table II Relationship Between Loading Rate and Rock Compressive Strength

Rock	Loading rate V_1 (kg/cm/sec)	Compressive strength σ_1 (kg/cm^2)	Loading rate V_2 (kg/cm^2/sec)	Compressive strength σ_2 (kg/cm^2)	$\frac{\sigma_2}{\sigma_1}$
Granite	5	1220	20	2000	1.64
Marble	5	500	3×10^4	980	1.96
Rock Salt	7.5	116.5	3.3×10^4	285	2.45

3. Dynamic strength of rock mass

Since fissures, joints etc. exist in rock mass, its strength is always less than that of intact rock itself and a coefficient of reduction of 0.80–0.90 is commonly used. According to reference 1 for loading rate of 1×10^2 – 1×10^4 kg/cm/sec, the compressive strength of rock mass is increased at least by a factor of 1.16–1.43 and the tensil strength of 1.24–1.48 as compared with the static values. For intense blast the loading rate periphery suitable for underground construction ranges from 1×10^2 kg/cm/sec to 1×10^4 kg/cm/sec. Considering that failure of rock mass at tunnel is usually controlled by its tensile strength, the dynamic strength is taken as 1.3–1.4 times the static strength. Expression for dynamic strength of rock mass is given as

$$\sigma_D = K_D \sigma_p \qquad (4)$$

where

σ_D --dynamic tensile strength of rock mass, kg/cm^2

σ_p --static tensile strength of rock, kg/cm^2

K_D --increment factor for dynamic strength of rock

When surface rock of gallery is stable, it is a common practice to apply 5 cm thick shotcrete on the surface, taken K_d = 1.04–1.20. When surface rock is unstable, anchor bolts are required for reinforcement and 5 cm thick shotcrete is then used for lining, taken K_d = 1.30–1.40.

4. Stability of tunnel under action of blast earth waves

1) Dynamic and static stress concentration

Tab. III shows the dynamic and static stress concentration factors under a horizontal blast wave for the side wall-and-crown tunnel without lining.

Table III Dynamic and Static Stress Concentration Factors for Unlined Tunnel Opening

Tunnel Type	Location	Dynamic Stress Concentration Factor K_d: Analytical solution $\frac{D}{\lambda}$=0.2–1.5	Dynamic Stress Concentration Factor K_d: Numerical Calculation $\frac{D}{\lambda}$=0.5	Static Stress Concentration Factor K_c
Circular Arch with Vertical Side Walls	Crown	+(3.25–2.25)	+3.00	-3.25
	Side Wall	+(2.0–1.65)	+1.80	-1.50

2) Calculation of stability for tunnel affected by blast seismic wave

The requirement for rock mass stability around the tunnel without lining is that the sum of static stress by mountain body and the dynamic stress by blast wave is less than the dynamic strength of the rock mass, i.e.

$$\sigma = \sigma_{CT} + \sigma_{DT} \leqslant [\sigma_D] \qquad (5)$$

where σ --the total stress in the rock mass, kg/cm^2

$[\sigma_D]$--the allowable dynamic strength in rock mass, calculated by equation (4), kg/cm^2

σ_{DT}--rock mass dynamic stress affected by the blast seismic wave

$$\sigma_{DT} = \frac{K_g \gamma}{2g} C_e V \times 10^{-3} \ \ \mathrm{Kg/cm^2} \qquad (6)$$

where γ-- unit weight of rock T/M^3

C_e — longitudinal elastic wave velocity, m/sec

V — particle vibrating velocity at tunnel periphery, cm/sec

σ_{CT} — static stress produced by rock mass itself

3) Calculation of critical particle velocity where rock mass is located in elastic and elastoplastic zones

From Eq. (5) critical particle velocity can be derived as

$$V_e = \frac{2(K_D\sigma_p - \sigma_{CT})g}{K_g \gamma C_e} \times 10^3 \qquad (7)$$

where V_e--critical particle velocity in elastic and elastoplastic zones, cm/sec

g--gravity acceleration, g = 9.81 m/sec

To calculate critical particle velocity of rock in the elastic zone, the longitudinal elastic wave velocity C_e is used. When critical velocity is required for rock in zones where cracking begins to occur, longitudinal elastoplastic velocity C_p should be used. If actually measured data of elasto-plastic wave velocity are not available, $C_p = \frac{1}{2} C_e$ may be used.

4) Calculation of critical particle velocity when collapse happens in rock mass

Under blast wave cracking often initiates in areas where maximum force exerts in weak fabric zones of rock mass (e.g. joint planes, fissures, etc.) or in zones with low strength (soft rock, weathered rock, etc.). After cracking initiates, the blast wave, if continuously exerting will enlarge the cracks and develops new crack, thus, the fractured rock loses its stability and collapse ensues.

The experiments showed that after tunnel rock undergoes into plastic state crack occurs at the springline of the tunnel. With the prolonged action of the blast wave, deformation at the crown and the side walls increase without increasing of stress in rock and the entire tunnel structure is in unloading condition.

In case of intense blasting natural vibrating frequenciea in rock generally range from 10–15 Hz and the loading time of blast seismic wave is about 0.4–0.6 sec. If a tunnel is regarded as single degree of freedom system, unloading coefficient of tunnel in plastic state is same as plastic dynamic coefficient of single degree of freedom system at various ductility ratios. The plastic dynamic coefficient can be obtained by method of structural dynamics. Thus, when collapse occurs in rock the critical vibrating velocity can be computed from Eq. (7) and

$$V_p = \frac{2(K_D\sigma_p - \sigma_{CT})g}{K_g \gamma C_p} \cdot \frac{1}{K_z} \times 10^3 \qquad (8)$$

V_p--critical vibrating velocity when collapse happens, cm/sec

where K_z--unloading coefficient of tunnel structure in plastic state

From the measured data on tunnel deformation under blast wave we can get ductility ratio and then the unloading coefficient. When ductility ratios are 2,3,5, the corresponding unloading coefficients are 0.65, 0.50, 0.35. According to in-situ investigation as unloading coefficient K_z = 0.80–0.65, local collapse occurs, and the volume of collapsed rock is generally less than 1 m^3. When unloading coefficient K_z = 0.50–0.35, large-scale rock collapse happens.

5) Safe distance of unlined tunnel under blast wave

The safe distance can be derived from Eq. (1).

$$R = \frac{1}{(V/KC_e)^{\frac{1}{\alpha}}} \cdot Q^{\frac{1}{3}} \qquad (9)$$

where

R--safety distance of unlined tunnel under blast wave

α,K--chosen from Fig. 2 and Table 1.

Q--charge weight Kg, taking total weight for simultaneous blasting and maximum weight of each delay for second delay blasting

V--critical velocity, cm/sec, calculated from Eqs. (7), (8)

6) Comparison between calculated results and in-situ measured data

Given: span(1 = 3m), height (h = 3m) of a circular arch-vertical wall unlined tunnel, no anchor bolt used, coarse-grained granite (weathered). The measured data of mechanical properties of the rock are as follows: Protodyakonov coefficient f = 4-6, unit weight = 2.64 T/m^3, longitudinal elastic wave velocity C_e = 2060 m/sec, dynamic elastic modulus E = 0.928 x 10^5 kg/cm^2, Poisson's ratio = 0.30, angle of internal friction = 41^o, static tensile strength = 23.0 kg/cm^2.

Taking f = 5, C_e = 2060 m/s, longitudinal elasto-plastic wave velocity C_p = 1030 m/s, dynamic strength increment ratio of rock mass K_o = 0.65, unloading coefficient for local collapse K_z = 0.65, unloading coefficient for large-scale collapse K_z = 0.35.

From Table IV it can be seen that the calculated and measured results will agree with each other. Therefore, the safety distance to ensure the stability of tunnel can be calculated from the critical vibrating velocity from Eq. (9).

Table IV Comparison of Critical Vibrating Velocity between Calculated Values and Measured Data

Rock Failure Extent / Vibrating Velocity cm/sec	no failure	cracking occurs	local collapse	large-scale collapse
actually measured data	30	30-50	50-100	100-200
calculated results	30.36	30.36-60.72	60.72-93.42	93.42-173.48

5 Conclusion

After comparing field measured data under various types of blasting with the calculated results, the method of calculation for determining stability of unlined tunnel under blast wave is given and the following conclusions can be drawn:

1) Empirical formulae applicable for practical engineering projects are given after studying and summarizing a large amount of field measured data on particle vibrating velocities of rock and soil under surface blast, open-air intense blast, blasting inside tunnel, etc.

2) Based on the analyses of the measured wave forms and numerical calculations, the existence and effect of the "an shock induced compressional seismic wave" for surface blast are found and the equations of midium's particle vibrating velocity are given.

3) Based on the stress theory and the criterion in which dynamic strength for rock mass is in equilibrium with the sum of dynamic and static stress acting on tunnel opening, the equations for computing critical particle vibrating velocity under the action of blast seismic waves are derived. These equations are applicable to unlined tunnels in the states of elastic deformation, crack initiation, local and large-scale collapse. The calculated results will agree with the actually measured data.

From the critical vibrating velocity the safety distance to ensure the stability of tunnel under blast wave can be determined.

REFERENCES

1. Zhu Rui-geng, Li Zhen (1979) The Safe Distance of the Tunnel Affected by the Seismic Wave due to Blast. Jour. "Underground Engineering" 1980

2. Zhu Rui-geng (1979) Blasting and Its Mechanical Effects in Soil and Rock Media. Jour. "Rock and Soil Mechanics No. 2. 1979

3. **Мосинец В. Н.** **Дробящее и сейсмическое действие взрыва в горных породах** **Недра Москва 1976**