Ground Vibrations Generated by Blasting

By

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Abstract

In tunnel driving through hard or soft rocks, the use of blasting has been a conventional and typical excavation method, as well as more efficient and economical. With the demands of tunnels and the development of rock blasting technique, the question of ground vibrations has become increasingly important, and it has become more or less routine to excavate rocks close to or below houses and buildings.

Blasting is carried out for the purpose of fragmenting and demolishing rocks for removal by utilizing the vast energy which the explosion of explosives generates. Near the explosive charge, the high amplitude stress wave and borehole gases produced by the detonation process crate crushed and fractured zones. Beyond these zones, where the amplitude of the stress wave does not exceed the strength of rock, the wave is propagated elastically. The elastic stress waves spread out in all directions and cause the surface of the underground opening to oscillate as a free surface. If the amplitudes of the wall vibrations, and particularly of the roof vibrations, are sufficiently high, loose blocks of rock may be dislodged or fractures in rock may be extended, further weakening the tunnel.

The empirical propagation equation is discussed with the consideration of following: a) particle velocity, b) distance from blasting point, c) types of explosive, d) quantity of charge, e) method of initiation, f) drilling pattern, g) path of propagation, h) tamping condition, i) characteristics of ground and j) state of ground formation.

1. Introduction

In tunnel driving through hard or soft rocks, the use of blasting has been a conventional and typical excavation method, as well as more efficient and economical. With the demands of tunnels and the development of rock blasting technique, the problem of ground vibrations has become increasingly important, and it has become more or less routine to excavate rocks close to or below houses and buildings.

Blasting is carried out for the purpose of fragmenting and demolishing rocks for removal by utilizing the vast energy which the explosion of explosives generates. Near the explosive charge, the high-amplitude stress wave and borehole gases

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produced by the detonation process create a crushed zone in the rock whose thickness is about equal to the borehole radius. Beyond the crushed zone, additional structural damage to the rock ranges from the micro-structural level up to visible fractures that may extend several meters. This type of rock damage, resulting primarily from the explosive charges near the final excavation lines, weakens the rock and increases the possibility of failure of the tunnel from subsequent vibrations or stresses. Beyond the zone of crushing and fracturing, where the amplitude of the stress wave does not exceed the strength of the rock, the wave is propagated elastically. The elastic stress waves spread out in all directions and cause the surface of the underground opening to oscillate as a free surface. If the amplitudes of the wall vibrations, and particularly of the roof vibrations, are sufficiently high, loose blocks of rock may be dislodged or fractures in the rock may be extended, further weakening the tunnel. The propagating vibrations may also influence the support systems and other facilities, houses with sensitive inhabitants or other structures on the surface. If the vibration levels can be predicted by using empirical propagation equations, and if the vibration differences can also be correlated with factors, such as delay type, shot location, explosive type, the vibration amplitudes may be reduced without materially altering the efficiency of the blast. With a vast increase of underground excavation anticipated for the future, a better understanding of vibrations from underground blasting is necessary.

In this paper, vibration levels from blasting in tunnelling, correlated factors, such as explosive type, delay type and shot location, damage to nearby structures and how to reduce vibration amplitudes are discussed.

2. Differences among types of explosive charge

As an explosion of blasting agents generates ground vibration, a type of explosive charge is considered as one of the major correlated factors determining amplitudes of vibration.

An amplitude of vibration generated by blasting is influenced very much by the time-dependent characteristic of the detonating gas pressure, even if an equal maximum gas pressure might be given. Namely, the raise time of the gas pressure, defined by a time reaching its peak from the initiation of the detonation, can be an important factor.

Let us discuss the behavior of vibration generated around a spherical cavity with the radius a, which is subjected to an inner pressure given by the following equation:

$$P(t) = P_0(1-e^{-qt}) \tag{1}$$

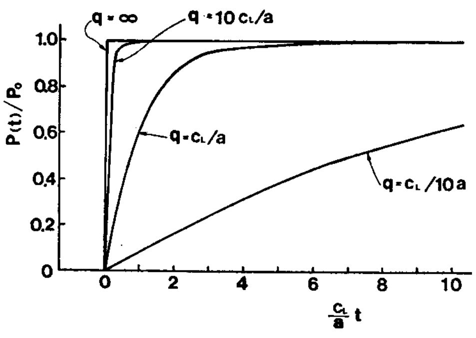

Fig. 1. Relation between Raise Time of Gas Pressure at Detonation and Value of q.

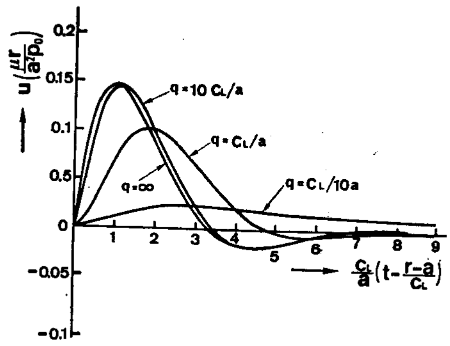

Fig. 2. Relation between Displacement and Dynamic Pressure for the Spherical Cavity

where $P(t)$, P_0 and q are detonating gas pressure, peak of detonating gas pressure and a constant to specify the raise time of the gas pressure, respectively.

Fig. 1 shows the changes of $P(t)$ with respect to time under the various values of q such as ∞, $10\ C_L/a$, C_L/a and $C_L/10a$, and based on Senozawa's solution [1]. Fig. 2 shows the displacements of the cavity wall (u) subjected to the inner pressures as shown in Fig. 1. C_L denotes the propagating velocity of the longitudinal wave in an elastic medium.

By comparing Fig. 1 with Fig. 2, it is well understood that the values of constant q, that is, the raise time of the detonating gas pressure, correlate much with the amplitudes of vibration; and that amplitude of vibration rapidly decreases when the raise time becomes long in some degree. On the other hand, the raise time of the gas pressure acting on the wall of a charged borehole is related to the detonation velocity of the explosive. It is better to take a longer raise time in such a way that the detonation velocity becomes lower. Additionally, it should be remembered that the magnitude of the detonating gas pressure itself has a great influence with regard to the amplitude of vibration.

There are very few examples in which the relation between the amplitude of vibration and types of explosive was confirmed practically under an equal situation. Here, several results obtained by the U.S. Bureau of Mines and the author can be picked up.

The U.S. Bureau of Mines had carried out its series of researches and investigations on blasting in the 1960s. In them, J. Olson et al. obtained interesting results with respect to the difference on vibration between AN-FO (Ammonium Nitrate Fuel Oil Blasting Agents) and dynamite from the investigations at the sandstone roof of the White Pine Copper mine, Michigan. [2] Fig. 3 shows the results under the conditions by which these investigations were carried out on

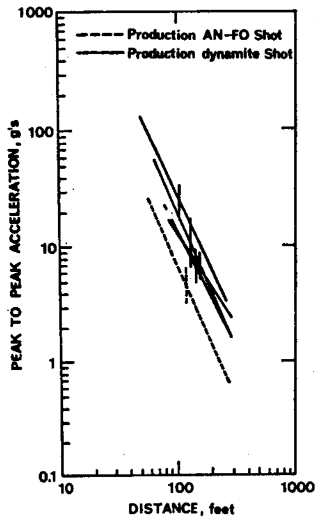

Fig. 3. Comparison on Peak-to-peak Accelerations at White Pine Copper Mine between AN-FO and Dynamite (after J.J. Olson [2])

the same face, with the same drilling pattern for V-cut centering and with the same quantity of charge (20 kgf). As shown in Fig. 3, the vibration amplitude by AN-FO might be reduced to 1/3 of dynamite. For reference, it should be pointed out that detonation velocities and detonating gas pressures of AN-FO and 60 %-ammonium dynamite are 2700 m/s, 15000~2000 kgf/cm^2 and 3600 m/s, 40000 kgf/cm^2, respectively and that production efficiency did not differ in either case.

Since 1970, the author has been doing research work aiming at the reduction of vibration generated by underground blasting operations. ([3]–[8]) Based on the results obtained by the above mentioned researches, the author confirmed the following empirical propagation equation:

$$V = K \cdot L^{2/3} \cdot D^{-2} \tag{2}$$

where A: maximum amplitude of particle velocity [cm/sec], L: quantity of charge per delay [kgf], D: distance from the center of the charge [m] and K: constant determined by four kinds of conditions such as the geological situation, namely characteristics of the rock and state of overburden, type of charged explosive, method of initiation (instantaneous shot, MS- or DS-delayed shot, centering or relief (easer) cut) and paths of the propagating wave. The appropriateness of Eq. (2) will be discussed in the following section, but in this section let us consider the differences among the types of explosives which were used under the same conditions.

Table 1. Properties of Explosives Manufactured in Japan

Explosive	Detonation velocity (m/s)	Specific volume (l/kg)	Force of explosive (l·kg/cm^2)	Purpose
Shinkiri dynamite	6500–7000	870–880	9600–11100	tunnel, open cut
2-Enoki dynamite	5800–6300	870	9000	tunnel
SB-dynamite	3500	—	9300	controlled blasting
AN-FO	2500	950	9600	open cut
Urbanite	1500–2000	—	9600	demolition in urban area
Black powder	300	280	—	quarry
Concrete cracker (CCR)	150	50	—	demolition in urban area
Detonating fuze	7000	—	—	initiation

Table 2. Coefficient K Observed in Granite (Kompira Tunnel)

Explosive	Detonation velocity (m/s)	Point *P* on rock			Point *O* on overlaying soil		
		Vert.	Hor. EW	Hor. NS	Vert.	Hor. EW	Hor. NS
2-Enoki dynamite	6000	7.2	2.9	2.1	4.7	5.3	8.1
Smokeless powder	2500	2.8	0.84	1.0	3.0	4.6	7.7
Concrete cracker	300	0.54	0.45	0.6	0.66	—	—

Notation Vert.: in vertical direction,
Hor.EW: in horizontal, east-west direction,
Hor.NS: in horizontal, north-south direction

For a discussion on the differences derived from the types of explosives, Table 1 shows the properties of explosives commonly used in Japan.

The author had discussed the differences through several field measurements in tunnelling sites and surfaces which were carried out in and on granite, shale and Izumi Formations, i.e. the alternative layers consisting of sandstone and shale widely spread over the south-west part of Japan. Table 2 shows the result obtained in the construction of the Kompira water supply tunnel driven through granite. Three types of explosives, 2-Enoki dynamite with a high detonation velocity, smokeless powder with a medium velocity and concrete cracker CCR with a low velocity, were discussed. [3], [4] The explosives were charged on the wall of a tunnel with a 50 m thick overburden. Two groups of vibration sensors were located on the surface in such a manner that one group is directly on the foundation rock (Point-P) and the other is on the overlaying earth (Point-Q), as shown in Fig. 4. The applied sensors were moving coil typed ones manufactured by Geo Space Co. (Type-GS-11D). One group for measuring one point consisted of three sensors which were set in the directions of vertical, horizontal in east-west and horizontal in north-south, respectively.

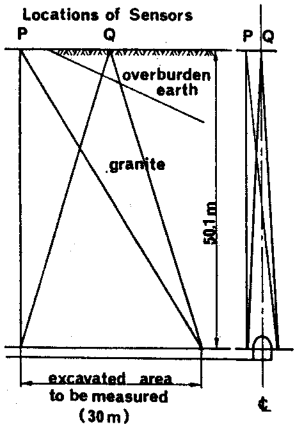

Fig. 4. Locations of Blasting Points and Sensors in Kompira Tunnel

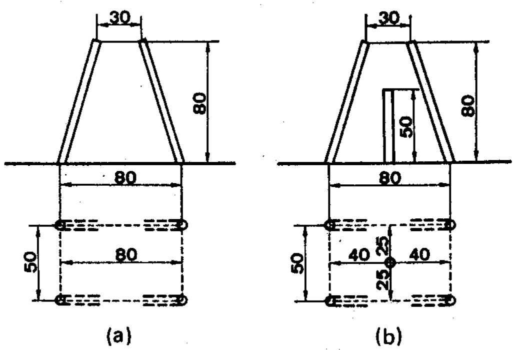

Fig. 5. Drilling Pattern for Tests in Kompira Tunnel

The drilling pattern for the charged explosives is as shown in Fig. 5, which corresponds to the most popular centering cut in rock tunnelling in Japan. The total quantity of 400 grams of explosive with 100 grams for each borehole was fired instantaneously by Zero-delayed electric blasting caps.

From Table 2 it can be considered that the amplitude of the ground vibration by CCR is approximately 1/10 of 2-Enoki Dynamite, and that the one by smokeless powder is 1/2 to 1/3 of Dynamite.

In analyzing the characteristics of blasting vibrations by different types of explosives, spectrum analyses were also made. It was realized that each type of explosive shows a unique distribution of prominent peaks of spectrum, corresponding to its detonation velocity. Fig. 6 shows the result of spectrum analyses for 2-Enoki dynamite and CCR. In the case of dynamite with 6000 m/s detonation velocity, the spectrum shows that the highest peak is found in the vicinity of 200 Hz, and the other prominent peaks are distributed in the range of 200–800 Hz.

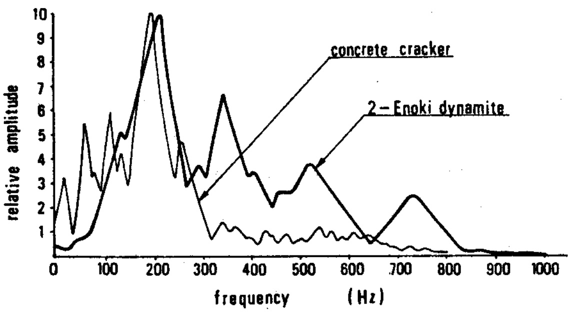

Fig. 6. Spectrum of Blasting Vibration

On the other hand, in the cases of smokeless powder and CCR, it was found, in the same manner as in the case of dynamite, that their highest peaks were around 200 Hz, but no prominent peak was found in the range over 300 Hz. Actually, most of the prominent peaks existed in the range below 200 Hz. From these results, it is concluded that an explosive with a higher detonation velocity is apt to generate higher frequency. When we discuss the damages of structures and the assessment to inhabitants influenced by the blasting vibrations, it will be important to consider not only the amplitude of vibration, but also the prominent frequency to be generated, with a consideration of the natural frequencies of surface soil on the rock as well as the structures on it.

In the project of the Kinki Regional Highway south of Osaka, the differences among Shinkiri-dynamite, AN-FO and Urbanite, which had been newly developed, in the reduction of vibration were considered, along with a consideration of the decoupling effect of the charged explosive. The rock at the experimental sites, at the faces of vertical shafts, was mainly Izumi-Formation. The same

Table 3. Coefficient K Observed in Izumi Formation (Vertical Shafts for Yamanaka Bridge)

Explosive	Charge per delay (kg)	Distance (m)	Particle velocity (cm/s)	Coefficient K	Mean	Relative ratio
Shinkiri dynamite	0.4	16.7	0.47	2.41	0.81	1.0
	0.9	15.6	0.02	0.05		
	1.5	13.8	0.56	0.82		
	1.5	15.9	0.16	0.30		
	1.2	12.5	0.35	0.49		
Urbanite	0.8	13.6	0.06	0.12	0.65	0.80
	0.6	15.3	0.38	1.24		
	0.2	16.0	0.08	0.58		
AN-FO	1.05	14.9	0.01	0.03	0.39	0.48
	0.7	14.2	0.01	0.03		
	0.9	16.7	0.38	1.12		

measurement system as used at the Kompira Tunnel was applied. The result obtained in this investigation is as shown in Table 3. It concludes that the amplitudes of vibrations by Urbanite and AN-FO are 50 % and 35 % of dynamite, respectively. (It should be noted that the effect of the decoupling charge was lost, mostly due to the inflow water at the face, and therfore Urbanite with a 1500–2000 m/sec detonation velocity showed a higher amplitude than AN-FO with 2500–3000 m/sec.)

3. Empirical propagation equation

The amplitude and behavior of the ground vibration generated by blasting are determined by many factors such as distance from the point of blasting, types of explosives, quantity of charge, tamping condition, drilling pattern, method of initiation, path of propagating waves, characteristics of ground, state of formation and so on. Therefore, it is extremely difficult from the practical point of view to establish to general equations for blasting vibration which can be applied uniquely to any case.

Therefore, several empirical propagation equations for blasting vibration have been proposed as the second best approach in such a manner that only two factors, distance (D) and quantity of charge (L), are picked up as variables under the fixed blasting conditions in specified rocks. Correlations among the amplitude of vibration (V), distance (D) and quantity of charge (L) are obtained empirically. These equations can be only applied in a reliable range depending on the frequency of measurement, the number of measured points, the certainty of geo-mechanical condition and the measurement system. Corresponding to the requirement for an empirical equation in each case, it is necessary to obtain information about the geological condition and blasting methods through well prepared planning.

The referential empirical propagation equations which had been proposed in the past are as follows: U, V, A, L, D and K denote the amplitudes of displacement in μ, particle velocity in cm/sec, acceleration in cm/sec^2, quantity of charge in kgf, distance in meter and constant.

1) Empirical equation for maximum amplitude of displacement

Hatanaka's equations [9], [10] are

$$
\begin{aligned}
U &= 4\times 10^{4}\cdot K\cdot L^{2/3}\cdot D^{-2.0} && \text{for } 15\text{ m} < D < 250\text{ m} \\
U &= 5.2\times 10^{2}\cdot K\cdot L^{2/3}\cdot D^{-1.2} && \text{for } 250\text{ m} < D < 1500\text{ m}
\end{aligned}
\qquad (3)
$$

where K is determined by the thickness of the overlaying soil formation (h). If λ denotes wave length,

$$
\begin{aligned}
K &= 7.0 && \text{for } h>\lambda \\
K &= 2.5 && \text{for } h<\lambda \\
K &= 1.0 && \text{for } h=0
\end{aligned}
$$

and $K>10.0$ for water bearing alluvial layers.

Through the investigations carried out in the 1930s, the U.S. Bureau of Mines [11] gives

Table 4. The List of Investigation Carried out in Japan

No.	Site	Object	Purpose	Cs [m²] ×APR [m]	Geology ⟨V_p⟩ [km]	Explosives	Charge [kg] per delay	total	Q_c [kg/m³]	Nh	Nh per m²	Observed equation	f [Hz]	D (m)	V_a (cm/s)	
1	K-Paper Mill Tunnel	Houses	channel	3×1.5	liparite 45	2-Enoki	Cc 1.2 Ec 0.6	26.2	5.82	41	13.6	$V=500L^{3/4}D^{-2}$ $V=170L^{2/3}D^{-2}$	90 ~100	50 ~100	0.3	Cc; Is
				3×1.0		Urbanite	Cc 0.8 Ec 3.5	17.4	5.80	41	13.6	$V=300L^{2/3}D^{-2}$ $V=100L^{2/3}D^{-2}$				Ec; DS2~8
2	Koi T. (East Sector)	Houses	railway	14.7×1.0	weathered granite ⟨1.3–2.6⟩	Modified Sugi-Dynamite	Cc 0.16 Ec 1.2	59	0.59	58	40.1	$V=500L^{3/4}D^{-2}$ $V=300L^{3/4}D^{-2}$	120 ~180	16 ~20	0.2	Cc; DS1~4 Ec; DS5~15
	-do-	-do-	-do-	14.7× 0.9 1.0	-do-	-do-	Cc 0.2 Ec 1.2		0.6~1.0	58	3.95	$V=405L^{3/4}D^{-2}$ $V=260L^{3/4}D^{-2}$	65 ~100	30 ~80	0.2	Cc; DS1~4 Ec; DS5~1
	-do-	-do-	-do-	35.1× 0.9 1.0	-do-	-do-	Cc 0.4 ~0.5 Ec 5.9		0.55 ~0.86	127	3.62	$V=350L^{3/4}D^{-2}$ $V=150L^{3/4}D^{-2}$	65 ~100	30 ~80	0.3	Cc; DS1~4 Ec; DS5~15
3	Koi T. (East Sector)	Existing tunnel	railway	14.7×1.5	Hard rock ⟨4⟩	2-Enoki	Cc 3.2 Ec 7.0	48.8	2.11	68	4.60	$V=500L^{3/4}D^{-2}$ $V=300L^{3/4}D^{-2}$		40 ~70	2.0	Cc; DS1~4 Ec; DS5~15
	-do-	-do-	-do-	14.7×1.5 35.1×0.9	-do-	-do-	Cc 0.2 Ec 2.2 Cc 0.2 Ec 2.2	30.0 27.7	1.36 0.88	70 127	4.76 3.62	$V=320L^{3/4}D^{-2}$ $V=208L^{3/4}D^{-2}$		~30	2.0	
4	A-Tunnel	Houses		7.0×1.1	Hard rock (Slate) ⟨3.0 3.5⟩	2-Enoki	Cc 0.8 Ec 2.0		1.08	38	5.43	$V=60L^{3/4}D^{-1.5}$ $V=40L^{3/4}D^{-1.5}$		50	0.35	Cc; Is Ec; DS2~7
5	B-Tunnel	Houses		7.8×1.0	Andesite	2-Enoki	Cc 1.2 Ec 4.9		1.52	39	5.00	$V=30.5\pm13L^{3/4}D^{-1.5}$ $V=9\pm5.5L^{3/4}D^{-1.5}$		63 ~200	0.1	Cc; Is
				7.8×1.5			Cc 1.6 Ec 4.9		1.41	4	5.77	$V=30.5\pm13L^{3/4}D^{-1.5}$ $V=9\pm5.5L^{3/4}D^{-1.5}$				Ec; DS2~7
6	C-Tunnel	Houses	railway	(9.5, 12.4) ×1.0	weathered granite	2-Enoki	Cc 0.2 Ec 0.5		0.25	23 31	2.42	$V=106L^{3/4-1.5}$ $V=82L^{3/4}D^{-1.5}$		40	0.45	Cc; Is Ec; DS2~8
7	D-Tunnel	Houses		25.2×1.0	weathered granite	2-Enoki	Cc 0.8 Ec 3.6		0.75	62	2.16	$V=44.2L^{3/4}D^{-1.5}$ $V=15.7L^{3/4}D^{-1.5}$		154 ~267	0.2	Cc; Is Ec; DS2~10
8	E-Tunnel	Houses		32×1.0	Hard rock	2-Enoki	Cc 1.2 Ec 4.0		0.72	87	2.72	$V=10L^{2/3}D^{-2}$		67 ~120	0.1	
9	Kompira T.	Houses			granite	2-Enoki	Ec 0.4			4						

10	Sasago T.	Existing tunnel	Highway	7.9×1.2	Soft rock Shale 〈3.5-4〉	2-Enoki	Cc 2.4 Ec 3.9		1.42	35	4.43	$V=1540L^{2/3}$ $V=580L^{2/3}$	300 ~400	140 ~150	6.53	Cc; Is Ec; DS2~10
				6.0×0.9			Cc 0.3 Ec 7.1		0.70	116	1.93	$V=200L^{2/3}$ $V=170L^{2/3}$	300 ~400	20~30		Cc; MS1~5 Ec; DS2~10
11	Suma water Supply T.	Existing tunnel	Channel	8.9×1.2	Soft rock weathered granite	2-Enoki	Cc 1.6 Ec 3.2		1.41	39	4.38	$V=503L^{2/3}$ $V=190L^{2/3}$		30~40		Cc; Is Ec; DS2~7
12	Anya water power sta.	Existing tunnel		13.8×1.3	Hard rock conglomerate		Cc 0.6 ~2.4		1.48 ~2.54	66~77	4.71 ~5.58	$V=1514L^{0.87}D^{-1.33}$			3.0	
13	Uryuno T.	Existing tunnel	Railway		Hard rock granite		Cc 0.5 Ec 1.0			47		$V=300L^{3/4}D^{-1.92}$		50	2.5	
14	2nd Yokoo T.	Existing tunnel	Railway		Hard rock granite	2-Enoki	Cc 0.4 Ec 1.2	8.2		35		$V=470L^{3/4}D^{-2}$		2~27	1.0	
15	Kitakyushu T.	Houses	Railway	15.3×1.5	porphyry	2-Enoki	Cc 1.6 Ec 5.0	28.4	1.23	54~58	3.66	$V=3880L^{1/2}D^{-1.6}$	20~30	50 ~200		
16	Hiroshima T.	Houses	Railway	15×1.5	Hard rock granite 〈4-5〉	2-Enoki	Cc 4.8 Ec 7.0	35.8	1.51	67	4.47	$V=10L^{2/3}D^{-2}$ $V=5L^{2/3}D^{-2}$		20~50		Cc; Is
				15×0.9			Cc 0.8 Ec 2.5	21.8	1.61	64	4.27	$V=10L^{2/3}D^{-2}$ $V=5L^{2/3}D^{-2}$				Ec; DS2~13
17	Bingo T.	Houses	Railway	16×1.5	Hard rock granite	2-Enoki	Cc 2.4 Ec 2.0	15.8	0.66			$V=1000L^{2/3}D^{-2}$ $V=890L^{2/3}D^{-2}$		170 ~350		Cc; Is Ec; DS2~10
				38×1.5				19.7	0.35							
18	Kameura T.				Soft rock Sand Stone 〈1.3〉	3-kiri ANFO Concrete cracker						$V=700L^{2/3}D^{-2}$ $V=700L^{2/3}D^{-2}$ $V=70L^{2/3}D^{-2}$				
19	Vertical Shaft				Soft rock Shale-Sand Stone 〈0.8-3.7〉	Shinkiri Shinkiri Urbanite Urbanite	3.8~5.4			37		$V=1800L^{2/3}D^{-2}$ $V=1000L^{2/3}D^{-2}$ $V=1200L^{2/3}D^{-2}$ $V=600L^{2/3}D^{-2}$		30		
20	F-Tunnel	Houses		38×1.2	Hard rock granite			47.5	1.04	125	3.29			127		
21	G-Tunnel				Hard rock basalt 〈3〉	2-Enoki		31.9						40 ~122		

Notation:

Cs: Cross section [m²]
APR: Advance per round [m]
Vp: P:wave velocity [km/s]
Qc: Charge per m³ [kg/m³]
Nh: Number of drill holes
f: frequency of wave [Hz]
D: Distance from shot point [m]
AL: Allowable limit
Va: particle velocity [cm/s]
Cc: Center cut
Ec: Easer cut
Is: Instantaneous shot

$$U = 30.1 \cdot K \cdot (e^{-0.00469D} + 0.0143) \cdot L^{2/3} \tag{4}$$

for $152\ \mathrm{m} < D < 1829\ \mathrm{m}$ and $454\ \mathrm{kgf} < L < 4536\ \mathrm{kgf}$, and K is 1.0 for the overlaying soil and 3.0 for water saturated layers of sand, gravel or loam.

2) **Empirical equation for maximum amplitude of acceleration**

Olson et al., the group belonging to the U.S. Bureau of Mines, obtained the equations

$$AL^{1/2} = 29000\ (D/L^{1/2})^{-2.07} \tag{5}$$

for the limestone-dolomite of the Shullsburg zinc-lead mine, Wis. [12].

3) **Empirical equation for maximum amplitude of particle velocity**

Based on many field investigations associated with the author, Asahi Chemical Industry has suggested the equation [13]

$$V = K \cdot L^{2/3} \cdot D^{-n} \tag{6}$$

for $30\ \mathrm{m} < D < 1500\ \mathrm{m}$, $10\ \mathrm{kgf} < L < 3000\ \mathrm{kgf}$.
K is 500–1000 for the center-cut in tunnelling, 200–500 for the bench-cut, and n is 2.5–3.0 for the clay layer and 2.0 for the rock.

Also, the author has proposed the following empirical propagation equation obtained in the investigations [3]–[8].

$$V = K \cdot L^{2/3} \cdot D^{-2} \tag{7}$$

where $K=800$–1000 for the center cut by gelatine dynamite
$K=400$–600 for the center cut by AN-FO,
$K=50$–100 for the center cut by CCR,
and each value of K should be reduced to 60 %–70 % for relief cut in tunnelling through medium hard rock. Table 4 shows the results obtained from 21 tunnelling sites and the employed equations in the past fifteen years. Table 5 shows the summary of the empirical propagation equations which are employed rather widely in Japan.

It can be concluded by comparing the above mentioned equations that a general empirical equation for blasting vibration takes the form of

$$A,\ U,\ V = K \cdot L^{m} \cdot D^{-n} \tag{8}$$

and the indices of m and n take the values of $1/3 \sim 3/4$ and 2.0 as the mean.

Fig. 7 is obtained by plotting the results obtained by the U.S. Bureau of Mines [11], Reid [14], Itoh [15], [16] and Hatanaka [17] onto the monograph expressed with the maximum displacement (A in mm) in the ordinate and scaled distance

Table 5. Summary of Empirical Propagation Equations

	Proposer	Equation	Conditions	Remarks	
Maximum particle velocity (V)	Itoh, Sassa and Tanimoto	$V=K\cdot L^{2/3}\cdot D^{-2}$	$8<D<300$	$K=250\sim1000$ $K=15\sim21$	gelatine dynamite black powder
	Asahi Chemical	$V=K\cdot L^{2/3}\cdot D^{-n}$	$30<D<1500$ $10<W<3000$	$K=500\sim1000$ $K=300\sim500$ $K=200\sim500$ $n=2.0$ $n=2.5\sim3.0$	for center-cut shot for easer shot for bench cut for rock for clay
	Nihon Chemical	$V=K\cdot L^{3/4}\cdot D^{-2}$	$5<D<3000$ $0.2<W<4000$	$K=450\sim900$ $K=200\sim500$ $K=300\sim700$ $K=100\sim300$ $K=300\sim2000$	for tunnelling for easer, roof shot in tunnelling for lifter shot in tunnelling for bench cut using large boreholes for lifter, loosening shot
	Nihonyushi	$V=K\cdot\left(\frac{D}{\sqrt{L}}\right)^{-1.5}$		$K=80\pm40$ $K=60\pm20$ $K=20\pm10$ $K=50\pm30$ $K=45\pm25$ $K=15\pm10$	for tunnelling, dynamite (2-Enoki, 3-Kiri) for tunnelling, control blasting for tunnelling, concrete cracker for open blasting, dynamite (2-Enoki, 3-Kiri) for open blasting, control blasting for open blasting, concrete cracker
Maximum amplitude (A)	Hatanaka	$A=400K\cdot L^{2/3}\cdot D^{-2}$ $A=5.2\cdot K\cdot L^{2/3}\cdot D^{-1.2}$	$15<D<250$ $250<D<1500$	$K=700$ $K=250$ $K=100$	for the case of $H_b>L_w$ for the case of $H_b<L_w$ for the case of no surface soil
	Yoshikawa et al.	$A=K\cdot L^{3/4}\cdot D^{-2}$		$K=100\sim700$	
	U.S. Bureau of Mines	$A=30.1\cdot K\cdot(e^{-0.00469D}+0.0143)\cdot L^{2/3}$	$152<D<1829$ $454<L<4536$	$K=0.1$ $K=1$ $K=3$	for the case of no surface soil $H_b<(1/2-1/4)L_w$ $H_b>1/2L_w$

V: Velocity of vibration [cm/sec],
A: Maximum amplitude [μ],
K: Constant,
L : Charge per delay [kg]
D : Distance from blasting site [m]
H_b: Thickness of surface soil [m]
L_w: Wave length [m]

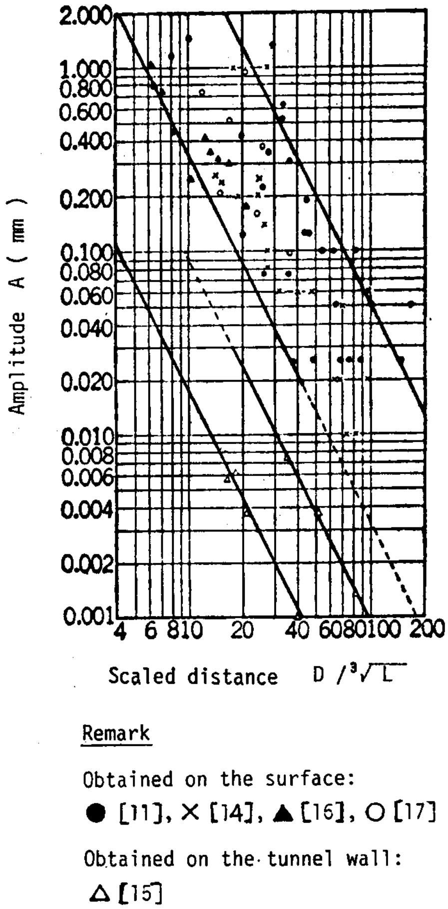

Fig. 7. Particle Velocity and Scaled Distance

$D/L^{1/3}$ in the abscissa. It verifies that appropriate indices, m and n, for the general empirical equation are 2/3 and 2, respectively.

The author also carried out an investigation aiming at the behavior of the attenuating vibration generated by blasting in Izumi Formation under a constant condition. [18] Fig. 8 shows the result, giving $n=2.012$ as a mean value.

4. Influence to amplitude of vibration by ground condition

In general, the amplitude observed on the roof or the wall of a tunnel is smaller than the one observed on the surface of the ground, for it is considered that the rock near the surface is affected much by weathering and overlaid by earthen material. In Fig. 7, the measured values of displacement related to the ground surface show a wide scattering in the range of 10 times to the same scaled distance.

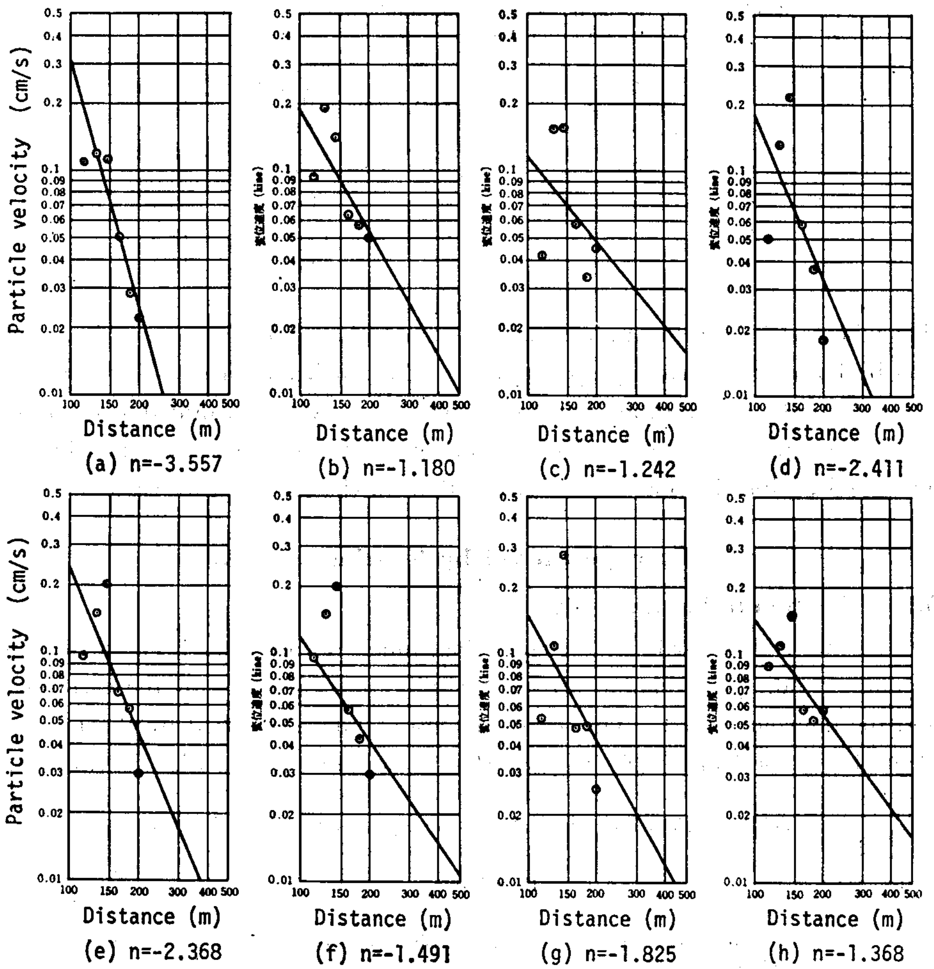

Fig. 8. Attenuation of Blasting Vibration in Izumi Formation

This suggests the magnitude of the influence by the ground condition.

Aptikaev [19] analyzed the relation between the peak amplitude of displacement and the specific impedance (defined by the product of density of the ground and propagation velocity of longitudinal wave), and obtained the result as shown in Table 6. It agrees with the above mentioned observation that the amplitude of displacement of the ground vibration varies inversely as the specific impedance of the ground.

Water content is also another factor correlated to the amplitude of vibration. It is pointed out that a wave propagating through water bearing ground shows a higher amplitude than ground in a dry state. For example, Kirillov [20] reported

Table 6. The Relation between Amplitude and Specific Impedance of Ground (after Aptikaev [19])

No.	Specific impedance ρc ($g/cm^3 \cdot km/s$)	$(\rho c)_1/(\rho c)_i$	Amplitude A (μ)	A_1/A_i
1	14.5	1	120	1
2	4.4	3.3	400	3.3
3	1.25	11.6	1500	12.5
4	0.75	19.5	2200	18.3

that vibration in a wet state became 2–2.5 times that of a dry state. Also, vibration by blasting in a river propagated five times farther than in a field. Gaskell [21] reported that vibration caused by blasting in water-saturated clay ground reached 3–4 times farther than in dry sandy ground; and that underwater blasting generated a vibration four times greater than in the case of clay ground.

5. Damage caused by blasting vibration

5.1 Vibration to human body

In the field of earthquake engineering, much research has been carried out, aiming at finding the relation between the vibration generated by the earthquake and the damage caused. The seismic intensity scale established by the Japanese Central Meteorological Observatory shows the practical relation between the

Table 7. JMA Intensity Scale (by the Japan Meteorological Agency)

Intensity	Description	Acceleration in gal
0	No sensation: registered by seismographs but no perception by the human body.	below 0.8
I	Slight: felt by persons at rest or persons especially sensitive to earthquake.	0.8 – 2.5
II	Weak: felt by most persons; slight rattling of doors and Japanese latticed paper sliding doors (shoji).	2.5 – 8.0
III	Rather strong: shaking of houses and bulidings; heavy rattling of doors and shoji; swinging of chandeliers and other hanging objects; movement of liquids in vessels.	8.0 – 25.0
IV	Very strong: strong shaking of houses and buildings; overturning of unstable objects; spilling of liquids out of vessels four-fifths full.	25.0 – 80.0
V	Very strong: cracking of plaster walls; overturning of tombstonse and stone lanterns; damage to masonry chimneys and mudplastered warehouses.	80.0 –250.0
VI	Disastrous: demolition of up to 30% of Japanese wooden houses; numerous landslides and embankment failures; fissures on flat ground.	250.0 –400.0
VII	Ruinous: demolition of more than 30% of Japanese wooden houses.	over 400.0

amplitude of acceleration and the observed phenomena, classified in eight categories. [22] (Table 7) According to this seismic intensity scale, the sensitive limit of human being to vibration is specified by the amplitude of acceleration, and it is 0.8 gal ($=0.8\ cm/s^2$). Also, the safety limit without any damage to houses in Japan is considered 80 gal.

As much study and investigation dealing with acceleration have been done, it may be convenient to employ the factor of acceleration in the discussion on the vibration-damage relation.

Commonly in both cases of earthquake and blasting, longitudinal waves, lateral waves and various surface waves were observed in the same manner. (For example, [23], [24], [25]) Though there is substantially no difference on the vibration between an earthquake and blasting, we should consider in detail the behavior of vibration generated by them that there may exist some tight correlations among the magnitudes of vibration energy sources, the mechanism of occurrence, the relative positions of vibration source and observed point, the path of propagation and so on. Comparing the vibration by blasting with that of an earthquake in general, we should point out two distinctive differences between them.

One of them is the difference in the range of frequency. In most cases of earthquakes, vibrations with relatively low frequencies, such as 0–10 Hz have been observed. In blasting, higher frequencies such as 30–300 Hz, sometimes over 1000 Hz, have been observed. Another distinctive difference is the duration. In earthquake records the duration of the vibration is ordinarily observed in a range from several seconds to several minutes, but vibration by blasting diminished within several hundred milliseconds at longest.

Reiher and Meister [26] reported the relations among human sensitivity to vibration, the amplitude of displacement and the frequency of vibration as shown in Fig. 9, which have been quoted very frequently. In Fig. 9, in which thick curves were shown by Reiher-Meister, the author added two relations, namely the relation between the displacement amplitude and the frequency under the constant acceleration 0.8 gal with a broken line, and the relation between the displacement amplitude and the frequency under the constant particle velocity 0.038 cm/s with a dotted line. In Fig. 9, the sensible limit to the human body can be specified with the acceleration, 0.8 gal, in cases with frequencies below 2 Hz, as well as in most cases of earthquakes, but in cases with frequencies higher than 5 Hz it becomes unappropriate to specify the sensible limit in terms of the acceleration amplitude. Hence, it is concluded that an appropriate expression as a major correlated factor in the discussion on the sensible limit to the human

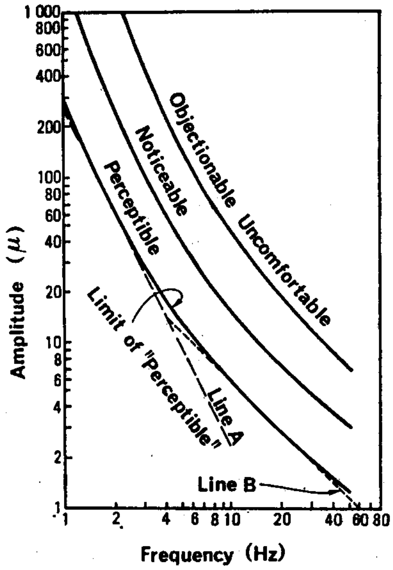

Fig. 9. Relation among Human Sensitivity, Amplitude and Frequency (after Reiher-Meister)

body should not be specified by the acceleration amplitude, but by the particle velocity amplitude in cases generating vibrations with frequencies of 5–60 Hz.

Thus, in cases of blasting ,the vibration accompanied by a higher frequency compared with an earthquake, the influence by the blasting vibration, that is, the sensitivity to the human body, has a tight relationship with the particle velocity amplitude, and the sensible limit in the particle velocity can be specified by 0.038 cm/s.

On the other hand, Goldman [27] gave a little higher value of 0.051 cm/s to the sensible limit in the cases of 6–60 Hz, based on the results about the relations among the particle velocity, frequency and human sensibility, which were obtained by applying the vibration in the form of a sine wave. Also, related to vibrations with 6–30 Hz, the Japanese Society of Architecture has specified the value of 0.06 cm/s as the allowable limit in designing structures and buildings with inhabitants against mechanical vibration, caused by the equipped machinery. [28] I.S.O. announced the specification on the vibration with respect to the allowable limit of exposure to the human body in the range of 1–80 Hz. [29] The objects in this specification are aircraft pilots and vehicle drivers who are exposed to constant vibration lasting for a long time. In considering the influences caused by blasting vibration, the differences of frequency and of exposure time should be thoroughly discussed.

5.2. Damage to structures, buildings and houses

The magnitude of vibration, which causes some damage to structures, buildings and houses, depends on frequency as well as the sensitivity of the human body ,as mentioned in the former section. Langefors et al. [30] gave the summarized relations among the displacement amplitude, frequency and damage, as shown in Fig. 10. It shows that in the range of 50–500 Hz, even if the amplitudes of the displacement and acceleration do not vary, the magnitude of damage may greatly differ. It also shows that the amplitude of the particle velocity has a

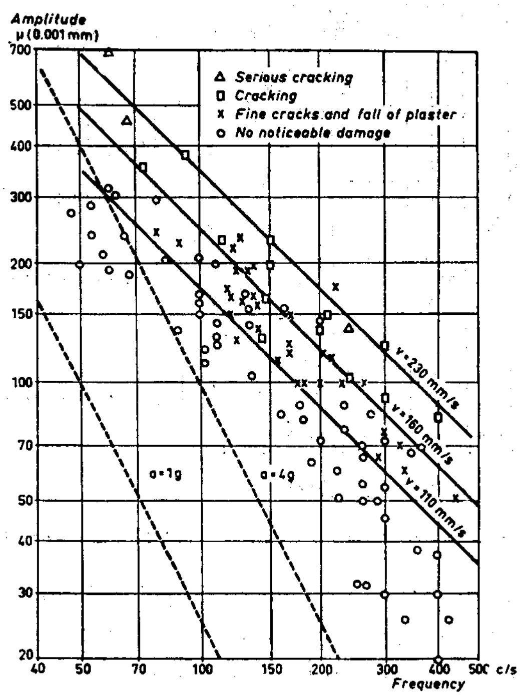

Fig. 10. Relation among Amplitude, Frequency and Damage (after Langefors)

proportional relationship with the damage of structures. (A numerical expression for the magnitude of damage is difficult, but conceptionally, the above mentioned proportional relation can be accepted.) With regard to the possibility of occurrence of damage to structures and buildings, it can be concluded from Fig. 10 that the safety limit may exist round 10 cm/s with respect to the particle velocity. Fig. 11 is the summary, based on the data reported by Thoenen-Windes [11], Reiher-Meister [26], Banik [31], and Crandel [32]. By adding the allowable limit of vibration investigated by Wiss [33], Fig. 12 can be obtained.

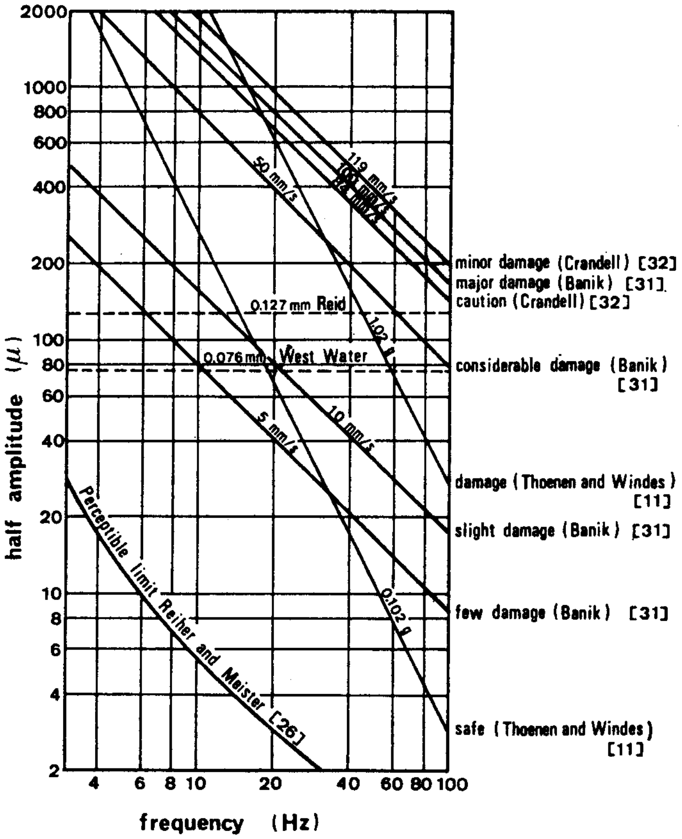

Fig. 11. Relations among Amplitude, Frequency and Damage

Langefors [30], Edwards [33] and Bumines [33] pointed out the indication by the particle velocity as the most predominant factor correlated with damage to structures and buildings. It also should be remarked that they gave almost the same value, that is 5 cm/s, as the safety limit. According to Banik [31], the safety limit of 0.5 cm/s is rather lower than 5 cm/s. This disagreement comes from the differences of conditions related to the ground. The investigations by Langefors and others were carried out on ground consisting of rocks with a longitudinal wave propagation velocity of 2000–3000 m/s. Banik's conclusion was obtained from an investigation related to buildings on alluvial layers, which were considered to respond excessively against vibration.

The U.S. Bureau of Mines had carried out its large scaled experiments aiming at the relation between the vibration and the damage of buildings. [33] In this project the measurement of blasting vibration was done 171 times totally. The method of initiation was mainly the MS-delayed blasting, the charge weight per

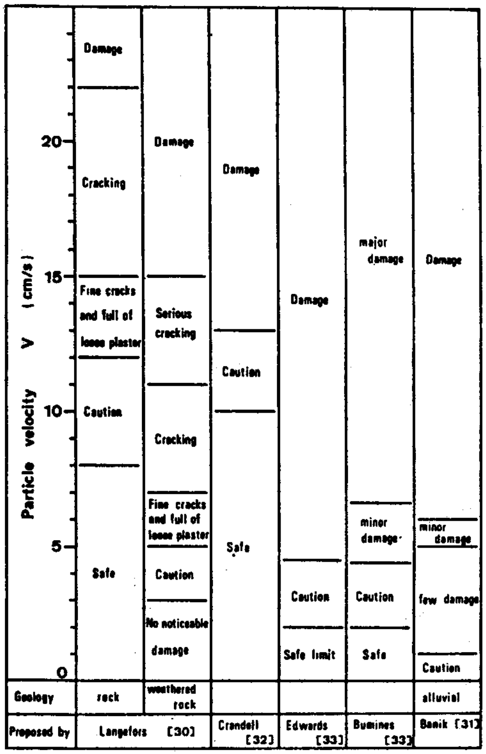

Fig. 12. Relations between Particle Velocity and Damage

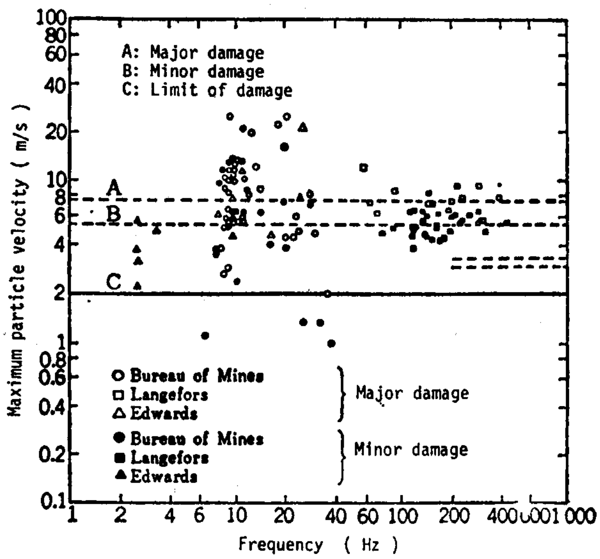

Fig. 13. Relation between Particle Velocity and Damage Observed by U.S. Bureau of Mines.

delay was 11–8900 kgf, and the objective rocks were limestone, dolomite, granite, diabase, schist, sandstone and so on. Fig. 13 shows the results obtained by the U.S. Bureau of Mines, whose conclusion suggested that the particle velocity should be dealt with as the main parameter in any discussion on the safety of buildings for vibration, and that the allowable limit might be 5 cm/s.

Concerning the designation for the allowable limit of vibration, Crandell [32] proposed the application of Energy Ratio (ER). Denoting acceleration and frequency, A (in ft/s^2) and f (in Hz), respectively, Energy Ratio is defined by A^2/f^2. According to ER, no damage is prospected for $ER<3$, to be cautious and expect fine cracks for $3<ER<6$, and damage to houses and brick-structures for $ER>6$. Energy Ratio, ER, can be expressed metrically as:

$$ER = 0.01674 f^2 U^2 \tag{9}$$

where U is the amplitude. Fig. 14 shows the safety limit by ER. In the same

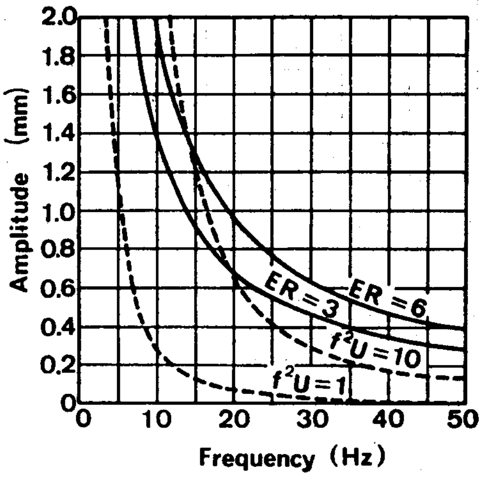

Fig. 14. Allowable Limit Specified by Energy Ratio

manner as Crandell, Thoenen [11] introduced another expression defined by $f^2 \cdot \bar{U}$ (in metric unit), which indicates the occurrence of damage for $f^2 \cdot \bar{U} > 10$ and to be safe for $f^2 \cdot \bar{U} < 1$. In Fig. 14, the results by Thoenen are shown by broken curves. In the range of low frequency, the results of both Crandell and Thoenen agree well, but a disagreement is recognized for high frequency.

Here, the author introduced several reports which were published in foreign countries, together with his own opinions. The research on the relation between vibrations and damage to buildings is quite practical, but with the exception of Hatanaka [17], there are very few investigations which aim at the systematic study about the detailed relation between the blasting vibration and the damage to buildings. Adding to the behavior of propagating blasting vibration mentioned above, further research on related damage should be advanced widely.

6. Reduction of blasting vibration

So long as the breakage mechanism of rock by blasting depends on dynamic action generated more or less by explosion, it is impossible to carry out blasting without any vibration propagating beyond the crushed zone in the vicinity of the charged boreholes. Therefore, it should be realized that the blasting operation is always accompanied by vibration and noise whatever their magnitudes may be. From the point of environment assessment during the construction of rock work, including tunnelling, it is necessary to discuss the reduction of vibration generated by blasting. Before a detailed consideration on the reduction of blasting vibration, it should be emphasized to execute the blasting in such an adequate way as to utilize the energy most efficiently.

The author chooses the following items as practical methods for the reduction of blasting vibration.

(1) To employ explosives with low detonation velocity:

The fundamental breakage mechanism of rock blasting consists of two actions, namely a dynamic one by the propagation of highly stressed waves and a static one by quasi-static gas pressure. It is pointed out by Murata [34]. Langefors [35], Fogelson et al. [36] that for a high explosive, for example, dynamite with a high detonation velocity, the shock wave energy most probably amounts only to 5–15 % of the total energy of the explosive, at a theoretical estimate. At least 2/3 of its energy will disappear without affecting the breakage, as the shock wave is distributed all around a charge. Hence, it is concluded that the shock wave is not responsible for the actual breakage of the rock, but only for providing the basic conditions for this process. The efficiency of explosives is determined by several factor such as the detonation velocity, the specific volume (The properties of representative explosives available in Japan are shown in Table 1) and the geological conditions of rock. As mentioned in Section 2, it is concluded that explosives with lower detonation velocities generate a lower magnitude of vibration.

In a comparison between dynamite and AN-FO, whose specific volumes are almost the same, there was not any noticeable difference on the efficiency of the blasting operation. However, there may exist rather remarkable differences on the blasting effect in rock excavation among other various explosives with quite different values of specific volume, such as the case of dynamite to CCR or AN-FO to black powder. Anyhow, the efficiency and the necessary quantity per unit volume for several types of explosives should be confirmed first of all by test blasting, and then it is recommended to choose a suitable type of explosive according to the indication obtained in the discussion of Section 2.

By making the magnitude of vibration caused by the explosion of gelatine dynamite in the case of one free face blasting applied to medium hard rock (center cut in tunnelling) 1.0 as a basis for comparison, relative values for some types of explosives and some proposed values of constant K in the empirical propagation equation, Eq. (2), are shown in Table 8.

Table 8. Comparison on Relative Magnitude of Vibration among Different Types of Explosive

Explosive	in center cut	in easer cut
Gelatine dynamite	1.0 (800–1200)	0.4 (320–480)
AN-FO	0.35 (280– 420)	0.15 (120–180)
Smokeless powder	0.3 (240– 360)	0.12 (100–150)
Urbanite	0.5 (400– 600)	0.2 (160–240)
Concrete cracker	0.1 (80– 120)	0.04 (30–50)

(2) To initiate with delayed EB-caps:

Once the type of explosive and the distance from the blasting site are fixed under a limited condition, the only way to reduce the magnitude of vibration by blasting is to reduce the weight of charge per delay as shown in Eq. (2). Also, it is well known among experienced engineers that the vibration amplitudes gained by employing delayed shots (DS- or MS-delayed) are 1/2–1/3 of those by simultaneous shots. This means the same fact that center cut generates much

Table 9. The Values of Coefficient K given by Different Types of Initiation (for Granite)

Shot point	EB-cap	Observed at Point P			Observed at Point Q		
		Vert.	Hor. EW	Hor. NS	Vert.	Hor. EW	Hor. NS
A	Zero	10.4	3.8	3.7	6.7	9.1	11.1
	DS-2	5.2	2.1	1.8	2.9	2.6	5.7
	DS-2	3.6	1.4	1.1	2.4	3.9	4.7
B	Zero	8.5	3.5	2.1	5.8	6.9	7.5
	DS-2	1.7	0.9	0.7	1.6	2.8	4.4
	DS-2	2.7	1.1	0.87	1.7	1.9	3.2

Notation Zero: instantaneous cap, Vert.: vertical direction,
Hor. EW: horizontal, east-west direction,
Hor. NS: horizontal, north-south direction

Table 10. The Values of Coefficient K given by Different Types of Initiation (Sandstone-Shale)

EB-cap	Point D	Point E	Point F
Zero	6.19 (1.0)	12.44 (1.0)	8.25 (1.0)
MS-5	4.83 (0.78)	7.42 (0.60)	4.91 (0.60)
DS-2	2.02 (0.33)	3.18 (0.26)	2.13 (0.26)

higher vibration amplitudes than those observed in easer cut. The author tried to confirm this fact under constant conditions. Tables 9 and 10 show the results obtained in the investigations at Komira Tunnel [3], and the vertical shafts with 4 m dia. for Yamanaka Bridge [6]. These tests were carried out in such a manner as executed center cut (one free face blasting) by initiating with instantaneous EB-caps (zero-delayed), DS-2 and MS-5 EB-caps at the faces under a constant condition (equal type of explosive, equal weight of charge per hole, equal number of boreholes, equal geological situation etc.). From those results, it is concluded that vibration amplitudes in easer cut by DS-2 caps and MS-5 caps are 30 % and 60 % of the amplitude in center cut by zero-delayed caps.

Let us further discuss the reasons for the reduction of vibration. In ordinary blasting, the explosive is not loaded to a sole borehole as a concentrated charge, but by being loading to many drilled holes, it is initiated with many detonating caps as a distributed charge. Therefore, if the total amount of charge might be initiated just instantaneously (in the strict sense) for the case that amount of W [kgf] per hole is loaded to several boreholes with the number of N, the generated vibration amplitude should be subjected to the value of $L=NW$ [kgf] in Eq. (2). On the other hand, if every explosive in each hole might be initiated at a certain time interval, and the generated vibration from each hole might be separated completely, the vibration amplitude should be determined by $L=W$ [kgf], and it would be reduced to $1/N^{2/3}$ of the case for $L=NW$ [kgf]. This difference is considerable.

In order to get a better understanding about the time-dependent change of the particle velocity generated by the wave from the explosion of the sole charge, let us consider the results obtained from the both theoretical calculation and practical observations. Under the dynamic pressure acting on the inside of a spherical cavity in an elastic body as shown in Fig. 15, the change of the particle velocity of the radial component is as shown in Fig. 16. [37] Comparing with this theoretical result, it can be seen that many waves recorded in the investigations, ([3]–[8]), show a shape quite similar to the one as shown in Fig. 16 when the simultaneous shot is applied with zero-delayed caps. Thus, we realize that the vibration from

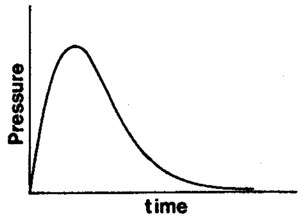

Fig. 15. Dynamic Gas Pressure Acting onto the Inside of a Spherical Cavity in an Elastic Body

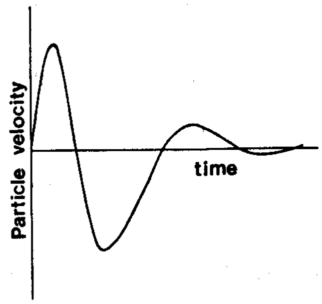

Fig. 16. Resultant Particle Velocity Subjected Dynamic Pressure

instantaneous shot, corresponding to oridnary center cut in rock tunnelling, behaves almost in the same manner as a solely charged explosion. When delayed shot is applied, vibration records show that 2–3 serial groups of waves, having a shape similar to a zero-delayed shot, are apt to be observed in each delay. (For example, see Fig. 17) This means the separation of the wave motion with respect

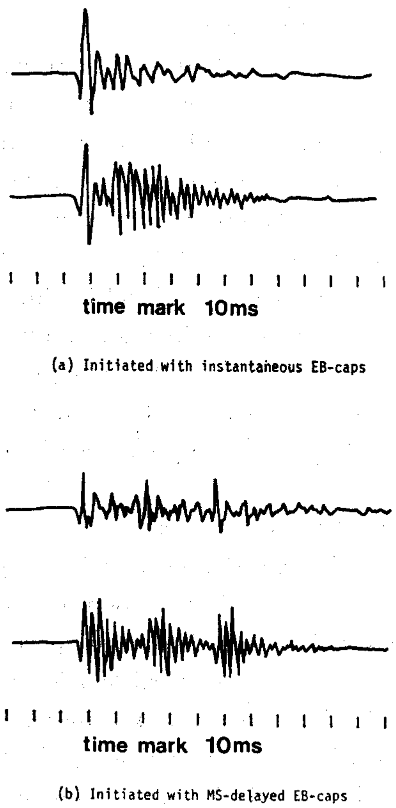

Fig. 17. Oscillogram of Blasting Vibration

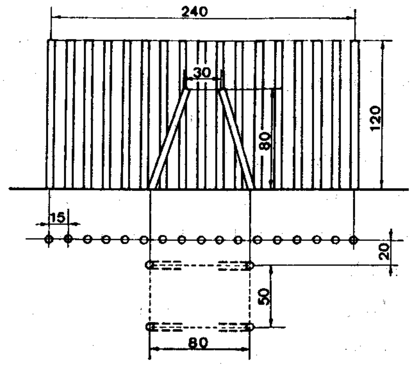

Fig. 18. Alignment of Boreholes for Pre-splitting

to each explosion per hole due to the scattering of initiation, for the accuracy of initiation of the zero-delayed EB cap is below 1.0 ms, and the DS-delayed cap has a scattering of ± 10 to 100 ms. The scattering of the MS-delayed cap is considered approximately 1/10 of the DS-delayed.

To speak of the separation of vibration with respect to various delays, being based on the observations, the duration of the main parts of the wave motion showing high peaks is considered approximately as 100 ms. [6], [37] When the EB-caps such as Zero-, MS-2 to MS-4 delay are used for initiation, the superposition of the wave motion is prospected more or less, but a noticeable superposition cannot happen with the usage of DS-delay caps and MS-delay caps with a number higher than MS-4.

Consequently, the amplitude of vibration can be specified by the quantity of explosive per delay in the case of DS- and MS-delayed blasting. Especially in the case of the DS-delayed shot, it is presumed that the amount per hole might be a more significant parameter in actual blasting.

Besides the above mentioned method to reduce blasting vibration, the application of pre-splitting blasting method, a controlled blasting, can be proposed. The author carried out an investigation aiming at learning the effect of the fractured zone produced by pre-splitting around the charged area for center cut as shown in Fig. 18. [3], [4], [37] The alignment of presplit holes as shown in Fig. 18 was not enough to give a noticeable effect.

The result of this investigation concludes that it is necessary to establish a considerably wide and thick presplit zone, which can cover the whole vicinity of the mining face in order to get a satisfactory effect for the reduction of vibration. From a theoretical point of view, the existence of an aperture produced by presplitting can be useful for the reduction of propagating vibration. Assuming the aperture distributed in the fractured zone to be one uniform layer consisting of an air gap as a whole, the vibration, namely propagating stress wave, which enters into the aperture perpendicularly with the particle velocity $w(t)$, removes the nearer free face along the aperture with the initial velocity of $2w(t)$.

The propagation equation of plane stress wave is expressed in terms of

$$w(t) = \sigma(t)/(1/\rho \cdot C_L) \tag{10}$$

where $w(t)$: particle velocity in propagating direction, $\sigma(t)$: dynamic stress in propagating direction, ρ: density of propagated medium, and C_L: velocity of longitudinal wave. By considering the stress wave as a part of the sine wave, the maximum particle velocity ($w_{\max.}$) is expressed in the form of

$$W(t) = W_{\max.} \sin 2\pi f \cdot t \tag{11}$$

where f: frequency

and assuming the amplitude of displacement to be zero at $t=0$, the maximum value of displacement, $U_{\max.}$ is

$$U_{\max.} = W_{\max.}/(\pi \cdot f)$$

Therefore, the necessary width of aperture, D_c, for the effective reduction should be

$$D_c > W_{\max.}/(\pi \cdot f) \tag{12}$$

In order to confirm this condition and the noticeable effect of presplitting, further field studies are required from a practical point of view.

7. Conclusions

In this paper, being based on many results obtained from field measurements, which were carried out in medium hard rock, the actual behavior of propagating vibration generated by blasting operation were discussed, and a simple empirical propagation equation was proposed from the practical point of view by the author. The reduction of blasting vibration has become an important subject in the field of environmental assessment accompanied by civil works due to the rapid and strong demands for tunnels with nearby structures and inhabitants. The main results derived from the study in this chapter are as follows.

(1) The behavior of ground vibration generated by blasting is governed by the following factors: a) distance from the blasted point; b) type of explosive; c) quantity of charge; d) method of initiation; e) drilling pattern; f) tamping condition; g) path of propagation; h) characteristics of ground; and i) state of ground formation.

(2) Differing from earthquakes with a frequency lower than 10 Hz, blasting generates a vibration with a frequency of 30–800 Hz. The frequency should be the predominant factor correlated with the magnitude of damage.

(3) In the discussion on the damage of structures and the sensitivity of the human body caused by blasting vibration, the particle velocity should be dealt with as the main parameter because the intensity of damage and human sensitivity has a proportional relation with the particle velocity in the case of being subjected to a vibration with a frequency over 30 Hz.

(4) For the prediction of blasting vibration, the following propagation equation can be applicable.

$$V = K \cdot L^{2/3} \cdot D^{-2}$$

where V: maximum particle velocity [cm/s],
L: charge weight per delay [kgf],
D: distance from blasted point [m],
and K: constant determined by the type of explosive, method of initiation, and geological condition. For example, $K=1000$ for the case of center cut by initiating gelatine dynamite with instantaneous *EB* caps.

(4) The raise time of gas pressure induced by an explosion correlates well with the amplitude of vibration, and this means that an explosive with a lower detonation velocity generates less vibration. The relative ratios of gelatine dynamite (D_e), AN-FO (A_0), smokeless powder (S_p) and concrete cracker CCR (C_r) are

$$D_e: A_0: S_p:C_r = 1.0: 0.35: 0.3: 0.1$$

(5) The usage of an explosive with a higher detonation velocity generates a vibration with higher frequency. According to spectrum analyses, the vibration generated by dynamite shows the prominent peaks in the range of 200–800 Hz; and the others show their prominent peaks on wave records in a range below 300 Hz.

(6) The sensible limit of blasting vibration to the human body is considered 0.038–0.05 cm/s in particle velocity.

(7) The allowable limit of vibration to buildings can be specified by the particle velocity in the range of 3–5 cm/s.

(8) To initiate with delayed EB-caps reduces the vibration. In general, center cut generates a higher vibration than easer cut, and the vibration amplitude in easer cut is 50 %–60 % of the amplitude in center cut with Zero-delayed caps. By utilizing the slight scattering of initiation time of delayed EB-caps such as DS-delay caps or MS-delay caps with a number higher than MS-4, a further reduction is possible.

(9) As the duration of the main parts of a wave motion showing high peaks is approximately considered as 100 ms, no noticeable superposition of wave motion can happen with the usage of DS-delay caps and MS-delay caps with a number higher than MS-4. Consequently, it is verified that the amplitude of vibration can be specified by the quantity of explosive per delay.

(10) Pre-splitting blasting is considered as one of the solutions for reducing vibration to nearby structures, and it is necessary to produce a considerably wide and thick presplit zone which can cover the whole vicinity of the mining face in order to obtain a fruitful effect.

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