M. RAMULU
T. G. SITHARAM
A.K. RAINA
P. B. CHOUDHURY
and
A. K. CHAKRABORTY

Effect of repeated blast vibrations on damage intensity of granitic rock mass at an hydroelectic construction project

Rock blasting induced ground vibrations produce deformations in the vicinity of blasting site. The effect of blast loading on structures is a growing concern of safety and stability. Extensive data are available on the behaviour of surface structures subjected to blast vibrations. However, only limited information is available on the effect of blast induced dynamic forces on the underground openings like tunnels and caverns. The reported findings state that blast induced dynamic stress, one or several cycles of repeated strains may cause deterioration in the rock mass or create damage to the dam foundation. This paper deals with the research work carried out at Jurala hydroelectric power project (JHPP) on the effect of repeated blast vibrations on powerhouse foundation in a jointed rock mass. The damage caused by blast induced vibrations can be categorized into two types: (i) near-field damage due to high frequency vibrations when the blast is occurring in the close proximity and (ii) far-field damage due to low frequency vibrations when the blast is occurring relatively farther distances. The near-field damage was assessed by monitoring blast vibrations and borehole camera inspection survey. The far-field damage was assessed by continuous monitoring of vibrations, borehole camera inspection survey and P-wave velocity measurement by ultrasonic testing machine. Borehole camera was used to examine the crack extension and damage inspection of rock mass. This paper reveals that repeated dynamic loading imparted on the jointed rock mass from subsequent blasts, in the vicinity, resulted in damage even at 22% of critical peak particle velocity (Vmax). The far-field damage due to the repeated blast loading of 40-45 rounds was more than 77% of the near-field damage. The results of the experimental study indicate that vibration levels, even at less than critical Vmax, can cuase safety and stability problems to the structures in/on jointed rock mass, when exposed to the repeated blast loading. The paper stresses the need for consideration of the effect of repeated blast loading in fixing the threshold limits of Vmax to avoid far-field damage.

Messrs. M. Ramulu, A.K. Raina, P. B. Choudhury and A. K. Chakraborty, Scientists, Central Institute of Mining & Fuel Research, Regional Centre, III-Floor, MECL Complex, Seminary Hills, Nagpur-440 006. cmrirc@dataone.in and T. G. Sitharam, Professor, Department of Civil Engineering, Indian Institute of Science, Bangalore-560012. e-mail: sitharam@civil.iisc.ernet.in

Introduction

The rock mass damage problem will be manifold if the blast loading is applied for repeated number of times, in contrast to the conventional single episode blast loading. Repeated blast loading causes progressive accumulation of damage in joints may lead to achievement of residual strength state in joints, with resultant large displacement at the joint surface (Brady, 1990). Studies on blast induced damage on underground openings are well documented by many researchers globally (Langefors and Kihlstrom, 1963; Hendron, 1977; Holmberg, 1993; Singh, 1993; Paventi et al, 1996; Yu and Vongpaisal, 1996; Chakraborty et al., 1998). In a series of experiments the Swedish Detonic Foundation has investigated the extent of cracking emanating from blastholes in controlled conditions (Olsson and Bergqvist, 1996; Ouchterlony, 1993; Ouchterlony, 1997). In this paper, it was aimed at prediction and assessment of blast induced damage and deterioration due to repeated dynamic loading produced by opencut blasting on the nearby underground openings of an hydroelectric project. The effect of repeated blast loading on jointed rock mass was qualitatively studied by many researchers globally (Atchison and Pugliese, 1964; Oriard, 1989; Law et al, 2001). By stressing the need for the study on the repeated dynamic loading, Brady (1990) states that, substantial progress has not been attained in the study of repeated exposures of dynamic loading on jointed rock mass in comparison to conventional blasting with single episode of loading. It was reported that rock mass subjected to repeated blast loading resulted in relatively excessive damage than a single fold blasting (Otuonye, 1997; Villaescusa, 2004). Brown and Hudson (1974) states that rock mass damaged by blast loading is predominantly due to joint motion, which is consistent with the experimental observation that joints decrease in shear strength under cyclic shear loading. This effect indicates pronounced modification of the peak-residual strength characteristic of a jointed rock specimen. Model studies of excavations in jointed rock under cyclic loading by Barton and Hansteen (1979) confirmed that excavation failure occurred by accumulation of shear displacements at joints. On the basis of these findings, St. John and Zahrah (1987) stated that, under dynamic loading, it is the number of excursions of joint motion into the plastic range that determines damage to an excavation. Wagner (1984) provided an indication of the general inadequacy of dynamic design based on Vmax of single blast round. A possible conclusion with regards to dynamic behaviour under a range of Vmax is that repeated dynamic loading may amplify problems of dynamic instability in jointed rock mass in the underground openings like tunnels and caverns. The major concern of this paper is to assess the effect of repeated blast vibrations on the damage intensity of jointed rock mass at the hydroelectric power project.

Assessment of rock mass damage due to blasting

The damage caused by blast induced vibrations can be categorized into two types: (i) near-field damage due to high frequency vibrations when the blast is occurring in the close proximity and (ii) far-field damage due to low frequency vibrations when the blast is occurring relatively farther distances. In case of opencut blasting, if the seismic source is within the distance of 25m from the monitoring point, that vibration is called a near-field vibration (Rustan, 1998). Although far-field damage is not a severe problem in single tunnel excavations, it was observed, by the authors, as an acute problem when the rock mass is subjected to repeated vibrations due to multiple excavations in the vicinity. The study was carried out at Jurala hydroelectric power project, India (JHPP), which is located at approximately 200 km South of Hyderabad in Andhra Pradesh, India. Rock excavation near the JHPP dam was carried out for construction of an hydroelectric power project with 6x39 MW units. The JHPP project construction required to excavate about 75000 m^3 of rock mass for making water conducting channels, called vents (5 to 60 m distance from dam foundation) and about 0.1 million m3 of rock for making tail race channel (TRC). A rock portion of 10 m wide, 60 m long and x 25 m height left unexcavated in between the vents for separating different units and water conducting systems in front of the dam. These unexcavated, rock structures were called as rock ledges. The ledges are formed in front of the dam across its axis, as shown in Fig.1. Two utility tunnels were also passing through the ledges in perpendicular direction to the ledge axis, as shown in Fig.2. The utility tunnels driven in ledge-I and ledge-II are called as tunnel-I and tunnel-II, where the instrumentation for assessment of damage was installed. Bench blasting in different slices was planned for the excavation of vents. There were 14-20 blasts conducted for each slice and there were 6-7 slices in each vent excavation. Therefore there would be about 200-250 blast rounds conducted in each vent excavation and there were 6 vents to be excavated. Thus, the rock mass at any part of ledge is going to undergo blast loading from both the neighboring vents and TRC blasts for several times, which is going to induce dynamic loading on the utility tunnels. The vibration intensity of these blasts would range from 50 to 1500 mm/s at a distance range of 2-50m. The experimentation for determination of repeated vibrations on jointed rock mass was conducted inside the utility tunnels, which are situated at the foundation rock of the ledges.

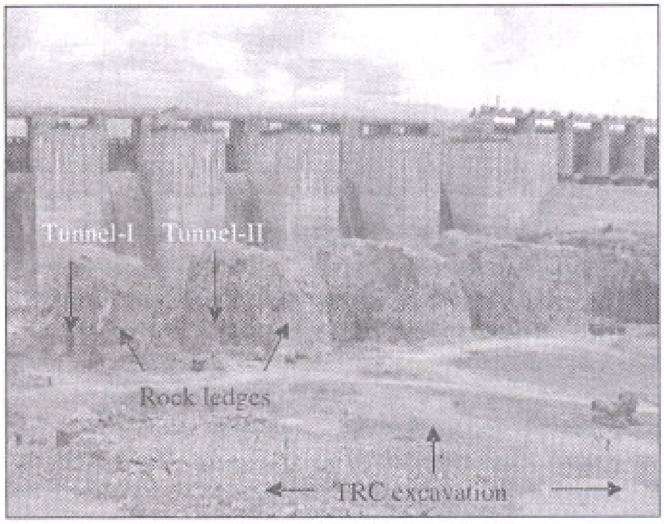

Fig.1 View of dam foundation and blasting location at PJHPP

Fig.2 View of instrumentation tunnels driven across the ledges

Geotechnical details of the experimental site

The rock mass at the ledges was moderately weathered, fresh and hard grey granite with thin pegmatite and dolerite dyke intrusions at the top 10-15 m. The foundation rock was pink granite with fractures and weak joints (Fig.1). Schmidt hammer rebound values were used to calculate in-situ properties of the rock mass. The rock formation was biotitic granite at first two ledges and pink granite at remaining ledges. The biotitic

Table 1. Geotechnical properties of the intact rock at PJHPP

	Rock type	Location	UCS, MPa	Young's Modulus, GPa	Tensile strength, MPa	P-wave velocity, m/s
1	Granite (biotite)	Ledge-I	85.6	134	10.75	6800
2	Granite (biotite)	Ledge-II	68.2	117	7.25	6400
3	Granite (pink)	Ledge-III	58	88	9.5	5520
4	Granite (pink)	Ledge-IV	47	79	6.7	5410

granite was relatively sound rock than pink granite. The average RQD of the ledge rock mass was calculated as 50%. The damage assessment study was conducted at the tunnels-I and II. The geo-technical properties of the intact rock observed at JHPP are given in Table 1.

Details of the blasting, instrumentation and methodology

The production bench blast details for TRC and vent excavations are given in Table 2. Instrumentation carried out in this study include, triaxial geophones for vibration monitoring, borehole cameras for observing crack extension and loosening of joints and ultrasonic instrument for P-wave velocity measurements. It has become common practice, recently, to use V_{max} a potential indicator for rock mass damage, as the V_{max} is directly proportional to the dynamic strain (Jaeger and Cook, 1979). Numerous authors used V_{max} as criteria for blast damage in rock mass (Langefors and Kihlstrom, 1963; Kutter and Fairhurst, 1971; Holmberg and Persson, 1978; Bauer and Calder, 1978; Oriard, 1982; Singh, 1993; Andrieux et al, 1994; LeBlanc, 1995; Yu and Vongpaisal, 1996; Villaescusa et all, 2004). Application of borehole camera for blast damage inspections was reported by many authors globally (Beyer and Jacobs, 1986; Stacey et al, 1990; Rocque et al, 1992; Singh, 1993; Andrieux et al, 1994; Doucet et al, 1996; Liu et al, 1998).

Table 2. Brief details of blast design for the production rounds of Vents and TRC excavations

	Blast parameter	Vent blasting with 32mm hole diameter	TRC blasting with 80mm hole diameter
1	Burden	2.0m	3.0m
2	Spacing	2.5m	4.5m
3	Hole depth	2.5m	5.0m
4	Charge per hole	4.17 kg	33.75 kg
5	Charge per delay	4.17 kg	67.5 kg
6	Specific charge	0.3 kg/m^3	0.5 kg/m^3

Ultrasonic methods have been used to detect flaws in metals and concrete. The use of such tests was sought in this study to estimate crack depths in rocks. Due to the inhomogeneous nature of the rock, only a few rock types such as rock salt and basalt are suited for flaw detection. This is because of the fact that acoustic velocity in a single material may vary over a large range due to the grain size and density variation (Koltonski and Malecki, 1958). The in situ values of the velocity also vary because of the pressure effect. An ultra sonic device called, "Telesonic" instrument (Roop Telesonics-India) was used for measurement of P waves of core samples. The instrument consists of a double probe (separate transmitter and receiver combination) which can be moved separately on the surface. The frequency range of the transducer is between 1 and 1000 kHz. The technique uses the indirect method of recording travel time of ultrasonic waves across a crack. The principle behind the technique is that a sound wave travels from a transducer to a receiver along the shortest path. Presence of fractures or crack extensions makes the P-waves to either slow down or traversing a longer path. P-wave velocity (V_p) of rock mass was determined by fixing the ultrasonic transducer at the monitoring station and receiver in a borehole, specially made inside the ledge wall as shown in Fig.3. The axis of the borehole made for ultrasonic receiver is parallel to the axis of the tunnel. The V_p levels gradually fall down as the damage extends from the surface of the tunnel wall. A room of 0.5 m^3 inside the ledge rock mass was made for installation of geophones to capture the blast vibrations from the surrounding excavations. A typical damage monitoring set-up with respect to blast site is shown in Fig.2, where the geophone room and borehole camera holes are shown. It was required to measure the blast vibrations in the near-field as well as far-field zones from blast sites to assess the rock mass damage. Therefore a room of 0.5 m^3 (1m x 1m x 0.5m) was excavated inside the Ledge wall for installation of geophones at the approximate height of 1 m from the bottom as shown in Fig.3. The geophone room excavations were carried out by controlled blasting by using mild explosive charges to avoid disturbance to surrounding rock mass. The blasting zones

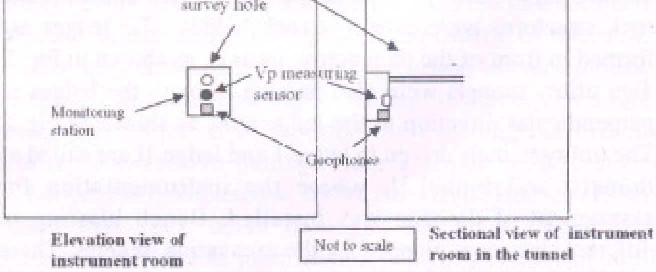

Fig.3 Schematic of installation locations of geophones and borehole camera survey holes and seismic survey holes

were ranging 2 - 75 m from the instrumentation location and the vibration monitoring was carried continuously, while the blasting face was proceeding towards the geophones from a distance of 75 m. The geophone sensors of higher frequency and recording equipment with faster sampling rates were used for near-field and ordinary geophone sensors were used for far-field vibrations monitoring. Borehole camera observation holes of 36 mm diameter and 4 m depth were drilled vertically downwards the ledge rock mass, approximately 1 m away from the geophone location (Fig.3). The camera observation holes were placed close to the geophone holes, so that the possible rock mass damage levels can be correlated with the measured V_{max}. The monitoring programme consisted of surveying the observation hole before and after each blasting round. The borehole camera used in this study was a robust unit with semi-rigid fiberglass signal cable. This camera contains a standard video output and can be connected with any TV or video-recording system with VCR input. Borehole surveys were made using a front view lens attachments, which could capture images from all the sides of the hole. This gave a clearer picture of the borehole wall, before and after blasting. All surveys were recorded in a computer attached to the camera and analyzed on surface to determine the frequency of cracks and crack extensions before and after each blast round.

Experimentation on the effect of repeated blast loading

Near-field blast damage assessment

The near-field damage to the rock mass of tunnel wall at the experimental station occurred mainly due to the production blast rounds of utility tunnel and the vents. The near-field damage was assessed by the Holmberg-Persson model (1978) and cross checked by borehole camera inspection survey.

The principle of Holmberg-Persson Equation, is to add the contribution of every small portion of the explosives column along the full charge length to derive the Vmax at a fixed sensor location. More details about the Holmberg-Persson model can be known from the sited references (Andrieux et al, 1994; LeBlanc, 1995; McKenzie et al, 1995; Meyer and Dunn, 1996; Liu and Proulx, 1996).

The Holmberg-Persson equation can be simplified to

$$V_{max} = K[a]^{\alpha} \qquad \text{... (1)}$$

where, a is here defined as the Holmberg-Persson term and K and α are the rock mass and explosive specific attenuation constants. K and α can be obtained by linear regression from experimental data i.e. vibration and distance. The mean values of K and α show the general trend of vibration attenuation in the rock mass.

In this study, The Holmberg-Persson approach was applied to determine the site specific constants K and α to model peak particle velocity attenuation across rock mass. Same type of explosive and design parameters were used for all the experiments and analysis of the results are presented in the following paras.

Near-field peak particle velocity (V_{max}) measurements were grouped and analysed separately for the two tunnels. Typical log-log plot of the measured V_{max} values obtained in this study against the Holmberg-Persson term. The V_{max} amplitudes were being experienced at similar distances and for the same design parameters. This plot is used to determine the K and α constants by fitting the linear relationship of the form, Log (V_{max}) = a Log (α) + Log (K). Results of the analysis are summarized in Table 3. The calculated K and α constants were slightly different for the tunnels-I and II. After determining the site specific attenuation constants, preliminary predictions of the extent of blast damage into wall-rock were made by applying the Holmberg-Persson model and by considering a site specific critical V_{max} or damage threshold (V_{cr}) given by the following relationship (Persson et al, 1994):

$$V_{cr} = \frac{\sigma_T V_p}{E} \qquad \text{... (2)}$$

where,

Vcr = Critical peak particle velocity before tensile failure (mm/s);

σT = Uniaxial tensile strength of rock (Pa);

Vp = Compressional wave velocity in rock mass (mm/s);

E = Young's Modulus of rock (Pa).

From the properties described in Section 4 (Table 1) and by adopting the above relationship, the value of damage threshold, V_{cr}, for the rock mass at tunnel-I was 590 mm/s and at tunnel-II was 460 mm/s. These threshold values were used to compare the extent of damage caused by the near-field blast rounds, which obviously generated maximum peak particle velocity. Fig.4 shows the results of this analysis. The above analysis indicates that the average extent of blast induced damage was 1.35 m and 1.58 m at the tunnels-I and II, respectively.

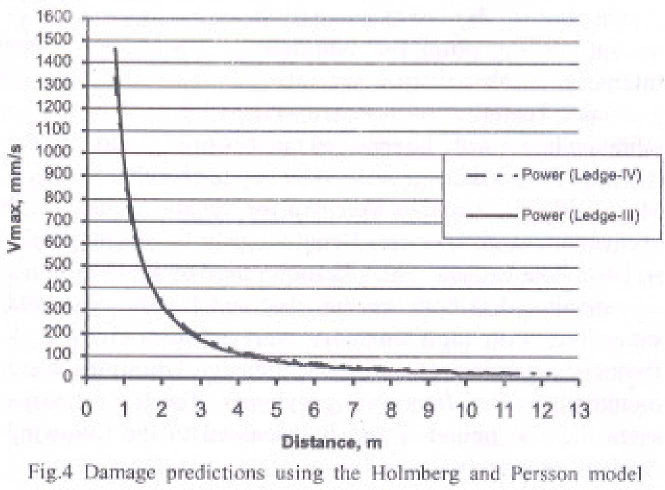

Fig.4 Damage predictions using the Holmberg and Persson model

TABLE 3. EXTENT OF PREDICTED ROCK MASS DAMAGE INTO THE TUNNEL WALL

	Rock type	K	α	V_{cr} from tensile failure criteria	Extent of damage from H-P model	Extent of damage from Borehole camera survey
1	Tunnel-I	940	0.55	590 mm/s	1.35 m	1.61 m
2	Tunnel-II	887	0.61	460 mm/s	1.58 m	1.85 m

The damage levels assessed by Holmberg-Persson model were cross checked by borehole camera survey before and after each blast round. An initial pre-blasting survey was performed in each hole to take pictures of pre-existing structural features for comparison with post blast surveys. This survey served as the reference survey in which pictures were taken at several centimeter intervals for comparison with the same pictures taken after each subsequent blast. A total of 2 borehole camera surveys were taken, one each at two tunnels. All the pictures were sorted out first by editing and matching the images of same depth before and after each blast. The images of observation holes captured by borehole camera clearly indicated that the near-field damage due to production blasts extended up to 1.61 m and 1.85 m at tunnels-I and II, respectively. Near-field damage observations of both the methods are given in Table 3.

FAR-FIELD BLAST DAMAGE DUE TO REPEATED VIBRATIONS

The multiple rounds of blasting activity was carried out for the tunnels, vents and TRC excavations of the of proposed powerhouse (Fig.1). Far-field rock mass damage observations were carried out by using borehole camera and by ultrasonic velocity measurements of pre-blast and post-blast rock mass. The V_{max} levels were also recorded for every blast round untill the vibration intensity attenuated to about 50 mm/s. The observation holes were under continuous monitoring with the borehole camera for about 65 rounds of blasts at both the surrounding vents (I and II), tunnel and TRC blasts. As the blast site is moving away from the monitoring point the vibration intensity, obviously, reduced gradually. The effect of these reduced vibration levels with the repeated number of exposures on the extent of further damage was studied by the borehole camera. Most of the blast rounds executed for the above mentioned excavations were monitored continuously by seismographs and borehole camera. The vibration range of 45-1475 mm/s was monitored at both the tunnels-I and II. The near-field vibrations with high intensity were monitored by high frequency geophones and low intensity vibrations were monitored by low frequency geophones. The blast damage assessment at tunnels-I and II discussed in the following sections separately.

Rock mass damage at tunnel-I

The vibration attenuation curve plotted for peak particle velocities versus scaled distance measured at the tunnel-I, is shown in Fig.5. The V_{max} recorded at tunnel-I were exceeding critical peak particle velocities (Vcr) for the initial 5 blast rounds, which were near-field vibrations. The V_p and V_{max} values measured before and after each blast round. The plot of Vp values versus V_{max} is shown in Fig.6. The trend of P-wave velocity with respect to the sequence of blast rounds is indicated by arrows in Fig.6. The following observations are made from the plot of V_p versus V_{max}:

(i) There was considerable reduction in V_p above V_{cr} and no reduction in V_p below critical peak particle velocity of near-field vibrations

(ii) Substantial reduction in V_p was noticed after 23 initial blast rounds.

(iii) The fall in V_p was continued uptill the V_{max} level reached 130 mm/s, which is about 22% of the V_{cr}

(iv) There was no considerable reduction in V_p below the 22% of V_{cr}, irrespective of the number of cycles of blast loading

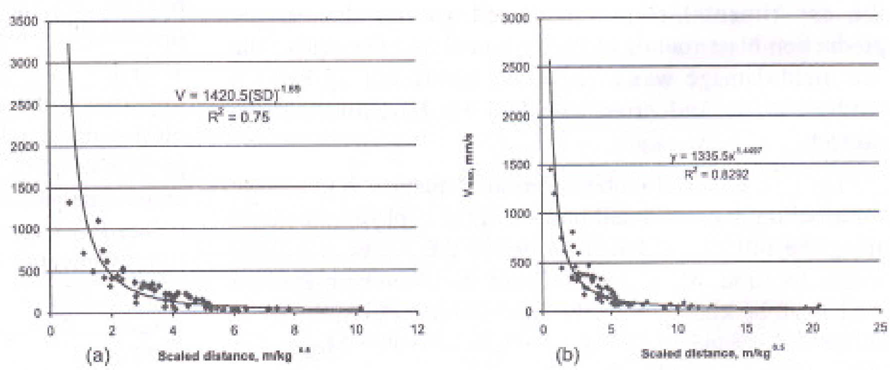

Fig.5 Vibration attenuation curve for (a) Ledge-III and (b) Ledge-IV

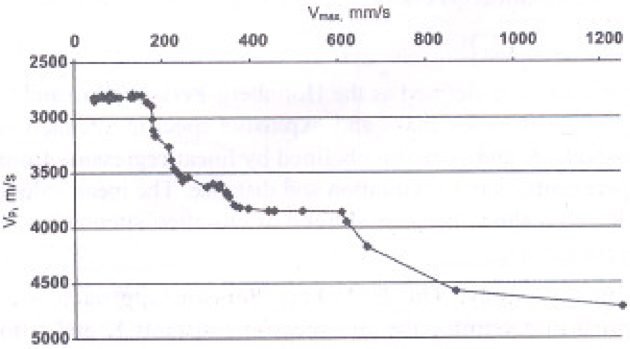

Fig.6 Plot of V_p versus V_{max} for tunnel-I

The borehole camera inspection survey was conducted to determine the exact extent of damage due to repeated loading. The borehole camera captured pictures indicated that the damage was observed only for the near-field blast vibrations and the crack extensions drastically come down and stopped after 6 initial blast rounds. The corresponding vibration level and the depth, where the damage process stopped completely was 595 mm/s and 1.56 m respectively. Again the damage was noticed by the borehole camera at a V_{max} of 315 mm/s after the occurrences of 28 vibration events. The damage and crack extension continued up to V_{max} levels of 130 mm/s and to a depth of 2.86 m. There were no crack extensions noticed by borehole camera inspection surveys below the V_{max} level of 130 mm/s for about 16 cycles of loading. The pre-blast and post-blast images captured by borehole camera at the tunnel-I at a depth of 2.86 m are shown in Fig.7.

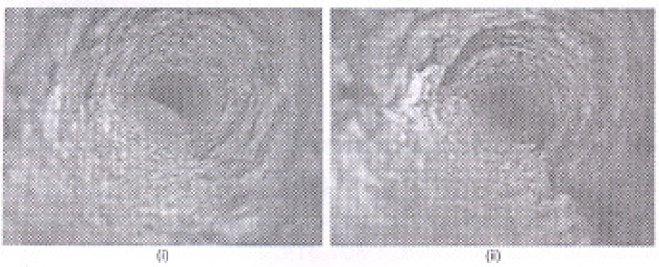

Fig.7 Images of borehole camera (i) before and (ii) after repeated dynamic loading at Tunnel-I

6.2.2 Rock mass damage at tunnel-II

Similar methodology was followed for assessment of rock mass damage due to repeated loading at tunnel-II. The vibration attenuation curve plotted for peak particle velocities versus scaled distance at tunnel-II rock mass is shown in Fig.5. The V_{max} recorded at Tunnel-II were exceeding critical peak particle velocities (V_{cr}) in the initial 7 blast rounds, which were near-field vibrations. The V_p and V_{max} values measured before and after each blast round. The plot of V_p values versus V_{max} is shown in Fig.8. The following observations are made from the plot of V_p versus V_{max}:

(i) There was considerable reduction in V_p above V_{cr} and no reduction in V_p below critical peak particle velocity of near-field vibrations.

(ii) Substantial reduction in V_p was noticed after 7 initial blast rounds.

(iii) The fall in V_p was continued uptill the V_{max} level reached 116 mm/s, which is about 25% of the V_{cr}.

(iv) There was not considerable reduction in V_p below the 25% of V_{cr} irrespective of the number of cycles of blast loading.

The borehole camera survey was conducted to determine the extent of damage due to repeated loading. The borehole camera captured pictures indicated that the damage was observed only for the near-field blast vibrations and the crack extensions drastically come down and stopped after 8 initial

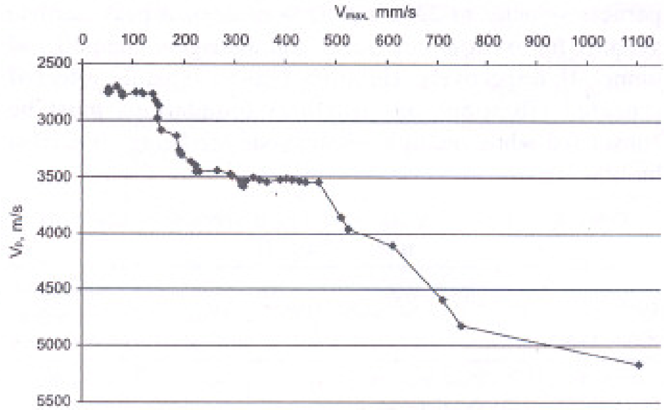

Fig.8 Plot of V_p versus V_{max} for tunnel-II

blast rounds. The corresponding vibration level and the depth, where the near-field damage stopped completely was 460 mm/s and 1.85 m respectively. Again the damage was noticed by the borehole camera at a V_{max} level of 278 mm/s after the occurrences of 31 vibration events. The damage and crack extension continued up to V_{max} levels of 116 mm/s and to a depth of 3.64 m. There were no crack extensions noticed by borehole camera surveys below the Vmax level of 116 mm/s for about 12 cycles of loading. The observations of borehole camera inspection survey are almost in line with the results of ultrasonic testing of rock mass.

Results and conclusions

The near field blast loading due to the production blasts of the utility tunnel and vent blast excavations could generate displacements in the rock mass, only when the peak particle velocity exceeded the critical vibration levels (V_{cr}). After repeated exposures of vibrations from the utility tunnel, vents and tail race channel blast rounds could produce displacements in the rock mass even at vibrations lower than the V_{cr}. The extra damage due to repeated blast loading at tunnels-I and II were 1.25 m and 1.79 m respectively.

After the exposures of about 45 blast rounds with the V_{max} levels ranging from 45 to 1475 mm/s, considerable damage was observed by borehole camera even at the V_{max} level of 130mm/s, which is approximately 22% of V_{cr} at tunnel-I. Similarly, the rock mass damage at tunnel-II was observed even at the V_{max} level of 116.0 mm/s i.e. at approximately 25% of V_{cr} after 45 numbers of exposures of blast loading. The threshold vibration limits with number of cycles of repeated loading for two different rock mass are given in Table 4. The observations also indicate that the repeated dynamic loading resulted in the damage at the vibration levels even at about 22% of V_{cr}. This observation was almost similar to the finding of Adamson and Scherpenisse (1998), which says that of threshold vibration level falls down to 25% of V_{cr} in repeated loading conditions. Almost similar results were achieved by Rajaram (1978). The present study reveals that the repeated dynamic loading resulted in reducing the threshold peak particle velocity to 22% and 25% of critical peak particle velocity for extension of cracks and damage at tunnel-I and tunnel -II, respectively. The study indicates that the effect of repeated vibrations on structural foundations must be considered while multiple excavations are being conducted by blasting method.

TABLE 4. VCR WITH NUMBER OF CYCLES OF REPEATED LOADING FOR TUNNELS-I AND II

	Rock type	No of cycles of blast loading	Threshold vibration limit, mm/s
Tunnel-I	Pink granite (moderately joints)	45	150
Tunnel-II	Pink granite (heavily jointed)	43	106

Acknowledgments

The research work presented in this paper is a part of Ph.D. degree of the first author from Indian Institute of Science and it is also part of CSIR sponsored project No. 70 (0058)/06/ EMR-II-CSIR for which the authors are grateful to CSIR. The authors express their sincere thanks to the Director, CIMFR for permitting to publish this paper. The authors express their gratitude to all the scientists of CIMFR Nagpur Centre, for their help in experimental work. The authors express their sense of gratitude to the executives of JHPP, APGENCO for their cooperation during the field studies.

References

1. Adamson, W. R. and Scherpenisse, C. R. (1998): The measurement and control of blast induced damage of final pit walls in open pit mining, Proc. of 24th Annual Conf. on Explosives and Blasting Research, New Orleans, LA, February 2-5, 1997, pp.539.
2. Andrieux, P., McKenzie, C., Heilig, J. and Drolet, A. (1994): The Impact of Blasting on Excavation Design - A Geomechanics Approach. In: Proceedings of the 10th Symposium on Explosives and Blasting Research.ISEE, Austin, USA, pp. 107-119.
3. Atchison, T. C., and Pugliese, J. M. (1994):. "Comparative Studies of Explosives in Limestone". *BuMines Rept. of Inv.* 6395, 25 pp.
4. Barton, N. and Hansteen, H. (1979): Very large span openings at shallow depth : deformation magnitudes from jointed models and finite element analysis. In proc. 4th rapid Excavation & Tunneling conf. (Atlanta,1979), Vol.2, 1331-1353.
5. Bauer, A. and Calder, P. N. (1978): Openpit and blasting seminar, Mining Engg. Dept. Queens University, Kingston, Ontario.
6. Beyer, R. R. and Jacobs, A. M. (1986): "Borehole television for geotechnical investigations", *Water Power and Dam construction*, 38 (9), pp. 16-28.
7. Brady, B. H. (1990): Dynamic performance and design of underground excavations in jointed rock, Static and dynamic considerations of Rock Engineering, Brummer (ed), Balkema, Rotterdam, pp.1-10.
8. Chakraborty, A. K., Raina, A. K., Ramulu, M. Jethwa, J. L. and Gupta, R. N. (1998): "Lake Tap at Koyna", *World Tunnel. Subsurface Exc.* November: 456-460.
9. Doucet, C., Cameron, A., Lizotte, Y. (1996): The effects of rock mass characteristics on fragmentation in controlled blasting experiments in small development headings, ISEE's 22nd Annual Conference, Orlando, FL, February 4-8, pp.1-11.
10. Holmberg, R., and Persson, P. A. (1978): The Swedish Approach to Contour Blasting Paper presented to Conference on Explosives and Blasting Technique, Society of Explosives Engineers, New Orleans, Louisiana, U.S.A., February 1 to 3, 1978.
11. Jaeger, J. C. and Cook, N. G. W. (1979): Fundementals of rock mechanics, 3rd ed. Chapman & Hall, London.
12. Koltonski and Malecki (1958): "Ultrasonic method for exploration of the properties and structure of mineral layers", *Acustica*, 8:307-314.
13. Kutter, H. K., and Fairhurst, C. (1971): "On the Fracture Process in Blasting," *Int. J. Rock Mech. Min. Sci.*, Vol. 8, pp. 181-202.
14. Langefors, U. and Kihlstrom, B. (1963): The modern techniques of rock blasting. New York: J. Wiley and Sons, Inc.
15. LeBlanc, T., Heilig, J. and Ryan, J. (1995): Predicting the Envelope of Damage from the Detonation of a Confined Charge. In: Proceedings of the Sixth High-Tech Seminar on the State of the Art in Blasting Technology Instrumentation and Explosives Applications, Massachusetts, USA, pp. 225-291.
16. Liu, Q., Tran, Hung, Counter, Dave and Andrieux, Patrick (1998): A case study of blast damage evaluation in open stope mining at Kidd Creek mines, 24th Annual Conference on Explosives and Blasting Research, New Orleans, LA, pp. 323-336.
17. Olson, M. and Bergqvist, I. (1996): Crack lengths from explosives in small diameter boreholes, Rock Fragmentation by Blasting, Rossamanith(ed.) A.A. Balkema, Rotterdam: 193-198.
18. Law, T. M., May, J., Spathis, A. T., Du Plessis, A. T. and Palmer, A. M. (2001): "Blast damage and blast dilution control: The applicaton of bulk emulsion systems at the WMC St Ives junction mine", *Fragblast*, Vol. 5, No. 1-2, pp. 1-20.
19. Oriard, L. L (1989): The Scale of Effects in Evaluating

Vibration Damage Potential", Proc. 15th Annual Conference on Explosives and Blasting Research, International Society of Explosives Engineers, New Orleans, LA, February 5-10, pp.161-176.

20. Oriord, L. L. (1982): Blasting effects and their control, Underground Mining Methods Handbook, SME of AIME, Littleton, Colorado, pp. 1590-1603.

21. Otuonye, F. O. (1997): Dynamic response of a fully grouted resin roof bolt to blast loading, Proc. of Annual Conf. on Explosives and Blasting Research, Explosives Reference Database on CD-ROM, Int. Society of Explosive Engineers, Ohio, USA.

22. Persson, P. A., Holmberg, R. and Lee, J. (1994): Rock Blasting and Explosives Engineering, CRC press Inc., Boca Raton, Florida, 540p.

23. Rajaram V. (1978): Mechanical behaviuor of Berea sandstone and Westerly granite under cyclic compression. Ph.D. dissertation, 89 pp. Univ. Wisconsin-Madison.

24. Rocque, P. (1992): Techniques utilized for the detection and characterization of blast induced damage, M. Sc. Eng. Thesis, Queen's University at Kingston, Ontario, Canada, 250 pages.

25. Rustan, A. (1998): Rock blasting terms and symbols, A.A. Balkema, Rotterdam, Brookfield, 1998, pp. 120.

26. Singh, S. P. (1993): Damage causing potential of different explosives, Proc, 9th annual Symp. on Explosives and Blasting Research, Feb., San Diego, California, pp.325-337.

27. St.John, C. M., and Zahrah, T. F. (1987.): "A seismic design of underground structures". *Tunneling and underground space Technology*, 2(2), 165-197.

28. Stacey, T. R., Cameron-Clarke and Mival, K. (1990): Stabilisation of old workings by blasting: Case study of a failed experiment, Static and dynamic considerations of Rock Engineering, Brummer (ed), Balkema, Rotterdam, pp.317-324.

29. Villaescusa, E., Onederra, I. and Scott, C. (2004): "Blast Induced Damage and Dynamic Behaviour of Hangingwalls in Bench Stoping", *Fragblast*, 8:1, 23 - 40.

30. Wagner, H. (1984): Support requirements for rockburst conditions, In Rockbursts and seismicity in mines, S.Africa Inst. Mining & Metallurgy, Johannesbug, pp.209-218.

31. Yu and Vongpaisal (1996): New blast damage criteria for underground blasting, *CIM Bull*. 89(998): 139-145.