Effect of Repeated Blast Vibrations on Rock Mass Damage at a Penstock Tunnel of a Hydroelectric Power Project – A Case Study

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ABSTRACT

The detonation of explosive charges releases large quantities of energy that can produce rock and soil deformations in the vicinity of blasting site. Extensive data are available on blasting in general and on the behavior of surface structures subjected to blast vibrations. However, only limited information is available on the effect of blast induced dynamic forces on underground structures like tunnels and caverns. This paper deals with the research work carried out at Koldam Hydroelectric Power Construction Project (KHEPP) on the effect of repeated blast vibrations on the jointed rock mass. Multiple rounds of blasts were conducted at the penstock tunnels and at the excavation site for powerhouse foundation. 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 at farther distances. The near-field damage was assessed by Holmberg-Persson model by means of monitoring ground vibrations and by borehole camera inspection surveys. The far-field damage was assessed by measuring deformations of borehole extensometers and by borehole camera inspection surveys. Peak particle velocities (V_{max}) generated by blast rounds were recorded by installing triaxial geophones near the borehole extensometers and borehole camera inspection holes. Damage assessment instrumentation was carried out at both the sides of penstock tunnel wall as another objective of the study was to compare the extent of rock mass damage with different joint orientations. The study reveals that repeated dynamic loading imparted on the jointed rock mass from subsequent blasts, in the vicinity, resulted in damage even at 22-26% of critical peak particle velocity (V_{cr}). The far-field damage due to the repeated blast loading, after 36-42 rounds, was more than 70% of the near-field damage. The results of the study indicate that vibration levels, even at less than critical V_{max}, can cuase safety and stability problems to the structures in jointed rock mass, when exposed to the repeated blast loading. The paper stresses the need for consideration of the effect of repeated blast loading for fixing the threshold limits of V_{max} to avoid the blast induced damage.

Keywords: Tunnelling; Repeated blasting; Rock mass damage; Peak particle velocity

1. INTRODUCTION

Blasting produces seismic waves similar to those produced by earthquakes, but with relatively high frequency and single pulse of low amplitude and the degree of structural damage depends on the total energy of explosion, distance from the source, and the character of the medium. Extensive experimental investigation in this regard was carried out by Thoenen and Windes (1942) of United States Bureau of Mines (USBM), Leet (1946) and Crandell (1949). Based on the experimental results the USBM recommended that no structural damage occurs if the acceleration of vibration is less than 0.1 times the gravity (g) and 0.1 to 1g is caution range; and the acceleration greater than 1g is danger zone. Leet (1946) prefers to limit the size of blasts by the displacements that they produce in the structure and the index of damage is a limiting displacement of 0.03inch (0.75mm). Crandell (1949) proposes to limit the size of the explosion by limiting the kinetic energy delivered to the ground, which is proportional to the quantity of the explosive. A number of studies attempted to correlate ground-motion levels with observed damage to structures. It is generally agreed that the amount of blast damage correlates best to the peak particle velocity (V_{max}). The blasting criteria for residential structures is generally less than 5 cm/s and for massive concrete structures is generally less than 25 cm/s (Charlie, 1985). Oriard (1989) observed that V_{max} of 8-10 ips (20-25cm/sec) normally does not damage the structure, because of the very high frequencies and the rapid, localized attenuation. Tart (1980) observed that at high frequencies the vibration levels of 275 ips (700cm/sec) generate minor cracks in old concrete. Rock mass damage in underground openings occurs mainly due to blast induced forces, stress redistribution and weathering. As underground excavations are carried out, the in-situ stresses redistribute around the boundary of the openings, leading to high stresses on the backs and corners of the excavations and the blasting activity creates initiation and extension of fractures in the surrounding rock mass. Blast damage is defined as either creation or extension of new fracture surfaces or opening of pre-existing geological discontinuities or both in the rock mass (Law et al., 2001). Blast induced damage weakens a rock mass, potentially leading to stability problems in the underground excavations. The stability of the underground structure is dependent upon the integrity of rock immediately surrounding the excavation. The blast damage can easily extend few meters into the rock and the loosened rock can give rise to serious safety and stability problems to the surrounding rock mass of the underground openings. The blast damage problem is more severe and vulnerable for the jointed rock mass in underground excavations (Singh and Xavier, 2005). Inspite of recognition of the importance of duration of ground motion on excavation dynamic response, current engineering practice correlates damage during an episode of dynamic loading to peak ground motion, since it can be related directly to peak transient stress in the ground wave, and the second power of velocity is related to dynamic strain energy (McGarr, 1983). Unfortunately, there are no specific safety guidelines available for the blasted tunnels with regards to the threshold limits of vibrations caused by repeated blasting activity in the close proximity. Many efforts have been made to study blast induced cracking and framing of safety guidelines in residential structures (Langefors and Kihlstrom, 1963; Dowding, 1985; Scott, 1996; Anon, 1997), but less attention was found to study blast induced damage to rock mass in underground openings (Persson et al., 1994). 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; and Zhang and Chang, 1999). In a series of

papers, SveDeFo has investigated the extent of cracking emanating from blastholes in controlled conditions (Olsson and Bergqvist, 1996; Ouchterlony, 1993; Ouchterlony, 1997). Joint orientation with respect to perimeter line of underground opening is one of the influencing parameters of blast induced damage (Connigham and Goetzsche, 1996). Singh and Xavier (2005) observed largest overbreak for the joint orientations of 45^0 and minimum overbreak for parallel and perpendicular joint orientations. Similar results were also obtained by Lewandowski (1996). In view of the large amount of underground excavations like tunnels and caverns it is imperative to develop threshold limits of ground vibrations induced by blasting within or outside the underground opening. 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 a tunneling project.

2. REVIEW OF LITERATURE ON THE ROCK MASS DAMAGE DUE TO REPEATED BLASTING

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, which may lead to achievement of residual strength state in joints, with resultant large displacement at the joint surface (Brady, 1990). The effect of repeated blast loading on jointed rock mass was qualitatively studied by many researchers globally (Atchison and Pugliese, 1964; Oriard, 1989; Otuonye, 1997; Law et al., 2001; Villaescusa, 2004). 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. Brown and Hudson (1974) states that rock mass damage 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. 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 along 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 V_{max} of single blast round. A possible conclusion with regards to dynamic behavior under a range of V_{max} is that repeated dynamic loading may amplify problems of dynamic instability in jointed rock mass in the underground openings like, multiple excavations, tunnels and caverns.

Although far-field damage is not a severe problem at the excavation sites where the blasting faces moves away and vibration gets attenuated substantially, 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. In view of the above arguments it was decided to go for the investigation of both near field and far field damage assessments at the Koldam hydroelectric power project.

3. SITE DESCRIPTION

3.1 General

The study was carried out at Koldam hydroelectric power project (KHEPP) of NTPC, which is located approximately 30km North of Bilaspur in Himachal Pradesh, India. The KHEPP had been constructed to tap the water flow of the Satluj river in Himalayan mountains (Fig. 1). Four penstock tunnels were excavated for hydraulic conductivity of the power house. Each tunnel was of horse shoe shape with 8m diameter, 8.5m height and about 400m running length. Open cut rock excavation was carried out for construction of powerhouse to install the power generation setup of 4x200 MW units. The location of penstock tunnels and the open cut excavation of the power house is shown in Fig. 2. There were six number of benches with height varying from 5-7 m and in each bench there were about 15-20 rounds of blasts conducted. About 150 blast rounds were conducted in the distance range of 5-100m from the monitoring station situated at the penstock tunnel. As there was repeated number of blast loadings on the underground openings, the experimentation was designed to determine the effect of repeated vibrations on rock mass damage, on the basis of previous experience of Ramulu et al. (2008). The damage assessment was carried out at both the sides of the tunnel as the joint orientation with respect to perimeter line was making acute angle at the left side and obtuse angle at the right side of the tunnel. The objective of damage assessment at both sides of the tunnel was to know the effect of joint orientation with respect to the perimeter line of the tunnel. The tunnel profile where the experimentation was carried out is shown in Fig. 3.

3.2 Geological and Geotechnical Information

The rock mass encountered in all the tunnels was Dolomite, which was very heterogeneous, highly weathered, metamorphosed, compact, foliated, sheared and crushed due to the effect of Chamiatar khad fault striking N170^0 E and 45^0 W. It is located around 450m downstream. The present Koldam axis location separates Upper dolomites and phyllites. Foliations are parallel to the Chamiatar khad fault. Rocks are weak and highly jointed with two main joints and few local and minor joints. Joints are open, closely spaced, intersecting, which are having clay fillings due to mechanical and chemical weathering of the rocks. One main joint with angle of N 75^0 E/80^{0}W is running parallel to the axis of the tunnels which is very unfavorable. There is prominent water seepage problem (Dripping conditions) in almost all the tunnels. At some places huge wedges are formed due to the intersection of the joints, which caused excessive overbreaks in the tunnels. The Q values of most of the rock mass of tunnels range from 0.12 to 0.21, which indicates that the rock was very poor. Core samples were collected from both the monitoring locations by underground coring machine. Engineering properties like Rock quality designation (RQD), compressive strength, tensile strength, density and compressional P-wave velocity (Vp) were determined from the core samples. In-situ compressive strengths were also determined by using Schmidt hammer rebound testing. The average in-situ compressive strength of Dolomite rock was 42.4 MPa. The average RQD values of dolomite rock mass ranging from of 40-60%. Water absorption properties measured at the test site was 1.2% at both the sides. The average intact rock properties of the dolomite at the two opposite walls at Chainage 120m are given in Table 1.

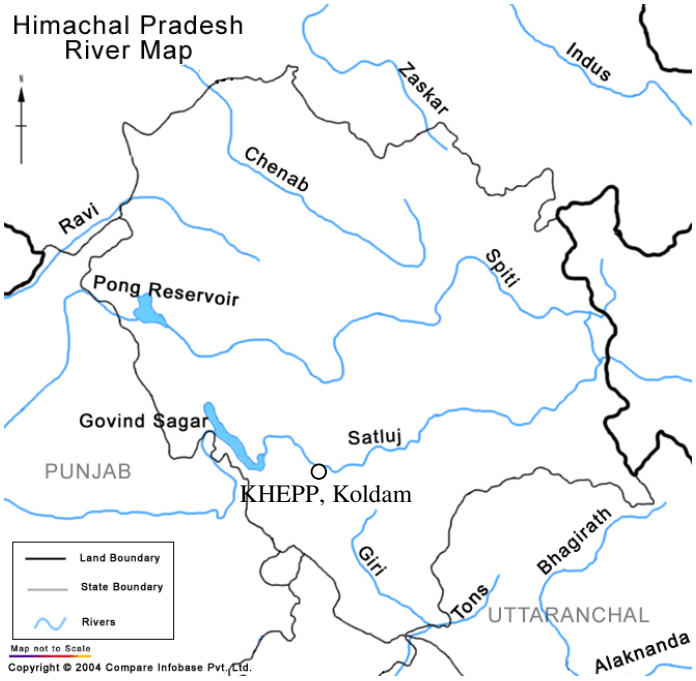

Fig. 1 - Location of the KHEP project site on Satluj River, Himachal Pradesh, India

Table 1 - Average intact rock properties of the Dolomite

Chainage	Rock type	Mass density, kg/m^3	Unconfined compressive strength, MPa	Tensile strength, MPa	Young's modulus , GPa	P-wave velocity, m/s
120m-Left side wall	Dolomite with acute joint orientation	2569	33	4.71	37.5	3201
120m-Right side wall	Dolomite with obtuse joint orientation	2582	32	4.57	37.9	3185

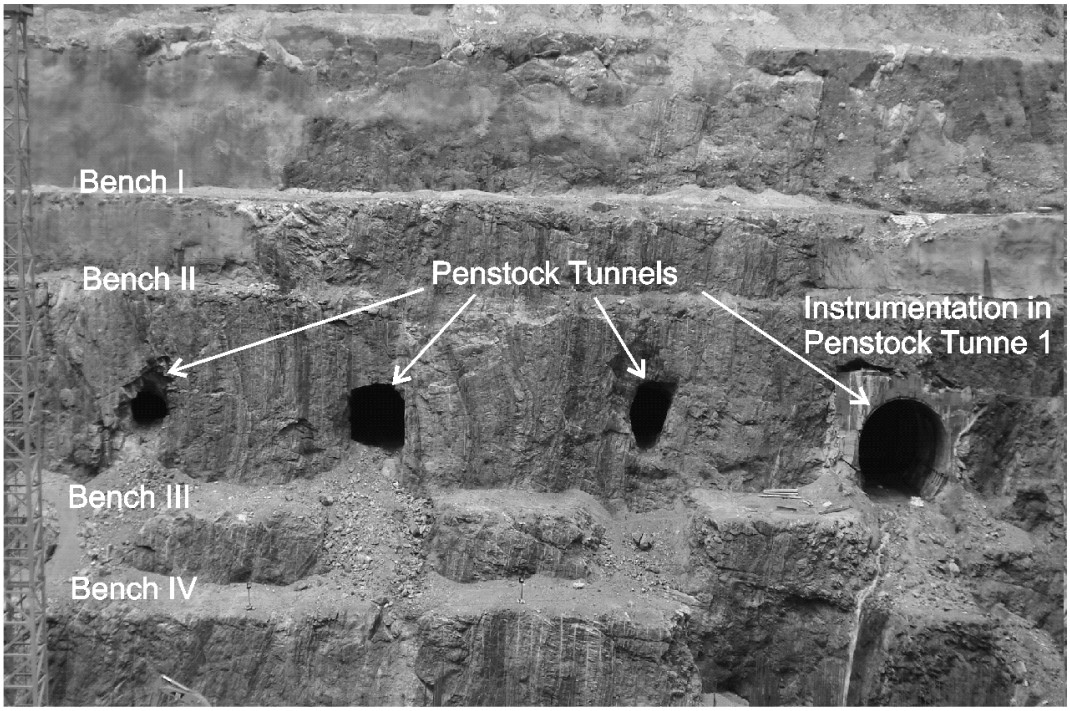

Fig. 2 - Location of penstock tunnels and the opencut excavation of the power house

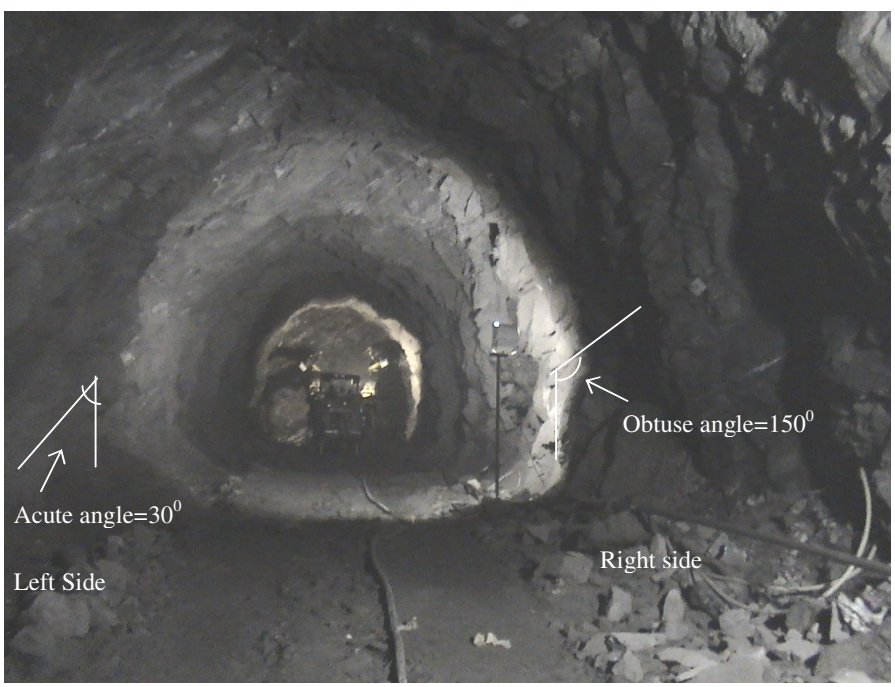

Fig. 3 - Penstock tunnel with acute joint orientation (Left side) and obtuse joint orientation (Right side) with respect to the tunnel profile

4. DETAILS OF THE BLASTING AND INSTRUMENTATION

The production bench blast details for power house site excavation are given in Table 2. Instrumentation carried out in this study include, triaxial geophones for vibration monitoring, borehole camera survey for observing crack extension, loosening of joints and borehole extensometers for measurement of plastic deformations. It has become common practice, recently, to use peak particle velocity (V_{max}) as an indicator of the potential for rock mass damage, as the V_{max} is directly proportional to the dynamic strain (Jaeger and Cook, 1979). Number of authors used V_{max} as criteria for blast damage in rock mass (Langefors and Kihlstrom, 1963; Kutter and Fairhurst, 1971; Holmberg and Persson, 1978; Holmberg and Persson, 1980; Oriard, 1982; Singh, 1993; Yu and Vongpaisal, 1996; Villaescusa et al., 2004). Application of borehole extensometers for blast damage inspections was reported by many researchers globally (Niklasson, 1985; Stacey et al., 1990; Kim et al., 1990; Villaescusa et al., 2004). Application of borehole camera for blast damage inspections was reported by many researchers globally (Niklasson, 1985; 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).

A room of 0.5 m^3 inside the tunnel wall was made for installation of geophones to capture the blast vibrations from the surrounding excavations and for the installation of borehole extensometers. A typical damage monitoring set-up at the tunnel wall is shown in Fig. 4, where the geophones, borehole extensometers and borehole camera survey holes are shown. The details of installation of instrumentation are given in the following sections.

Table 2 - Blast design for the production rounds of powerhouse foundation excavation

S. No.	Blast Parameter	Value
1	Hole diameter	80mm
2	Burden	4.0m
3	Spacing	5m
4	Hole depth	8.0m
5	Charge per hole	50 kg
6	Charge per delay	100 kg
7	Specific charge	0.4 kg/m^3

4.1 Installation of Geophones

It was required to measure the blast vibrations in the near-field as well as far-field zones with respect to blast site to assess the rock mass damage. Therefore a small room of about 0.5 m^3 (1m x 1m x0.5m) was excavated inside the sidewall for installation of geophones at a height of 1m from the bottom. The geophone room excavation was carried out by controlled blasting by using mild explosive charges to avoid disturbance to surrounding rock mass. The geophone room was located at about 2.5m from the initial blasting face and the vibration monitoring was carried out continuously, while the blasting face receded away up to a distance of 40-50m. The geophone sensors of higher frequency and recording equipment with faster sampling rates were used for near-field monitoring and ordinary low frequency geophone sensors were used for far-field vibrations monitoring.

4.2 Installation of Borehole Extensometers

Holes were drilled across the joint planes and foliations of the rock mass in the tunnel wall to install borehole extensometers. Conventional rod type borehole extensometers available in the market were used for measurement of plastic deformations. Two such borehole extensometers were installed at the left side and two were installed at the right side of the tunnel wall, at a height of 1.5m from ground level. Each extensometer consists of two extension rods of varying lengths inserted into the borehole and installed as per the norms prescribed by ISRM (1992). Digital gauges were used to measure the positive or negative extension values of the extensometer at the free end in the mouth piece. The location of borehole extensometers and geophones with respect to blasting location is schematically shown in Fig. 4**.** The lengths of borehole extension rods fixed were 3 and 4m at the left side and 3.5 and 4.5m at the right side of the tunnel wall.

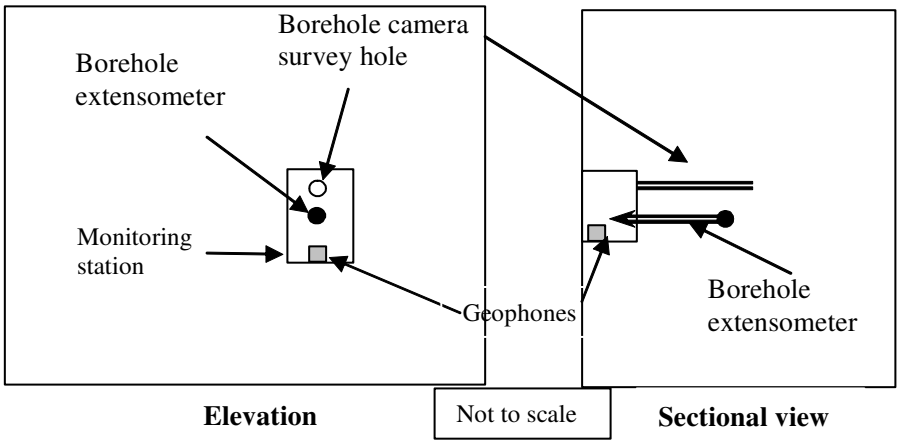

Fig. 4 – Schematic location of geophones and borehole camera survey holes and seismic survey holes

4.3 Installation of Borehole Camera

Borehole camera observation holes of 36mm diameter and 5m depth were drilled across the joint planes of the rock mass in the sidewall, approximately 0.5m away from the geophone location as well as extensometer (Fig. 4). The location and direction of holes were made such that it can intersect maximum joint planes so that the joint displacements and possible new cracks can be detected by borehole cameras. The camera observation holes were made at the close proximity of geophone holes, so that the possible rock mass damage levels can be correlated with the measured peak particle velocity (V_{max}). The monitoring program consisted of surveying the observation hole before and after each blasting event. 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 by 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 analysed on surface to determine the frequency of cracks and crack extensions before and after every blast.

5. EXPERIMENTATION ON THE EFFECT OF REPEATED BLAST LOADING

5.1 Near-Field Blast Damage Assessment

The near-field damage to the rock mass at the experimental station occurred due to the production blast rounds conducted within the tunnel. The near-field damage was assessed by the Holmberg-Persson (H-P)model (1978) as well as borehole camera 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 peak particle velocity (V_{max}) at a fixed sensor location. More details about the Holmberg-Persson (H-P) model can be known from the cited 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,

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

where, *a* isefined 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 on vibrations and distances. 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 paragraphs.

Near-field peak particle velocity (V_{max}) measurements were grouped and analysed separately for two experimental sites. 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, $\log(V_{max}) = \alpha \log(a) + \log(K)$. Results of the analysis are summarized in Table 3. The calculated K and α constants were slightly different for two sides of the tunnel where the monitoring stations were set up.

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),

$$V_{cr} = \frac{\sigma_T V_p}{E} \quad (2)$$

where,

V_{cr} = Critical peak particle velocity before tensile failure (mm/s);
σ_T = Uniaxial tensile strength of rock (Pa);
V_p = Compressional wave velocity in rock mass (mm/s); and
E = Young's Modulus of rock (Pa).

From the properties described in Section 3 (Table 1) and by adopting the above relationship (Eqn. 2), the value of damage threshold, V_{cr}, for the left side rock mass was 402 mm/s and at right was 384 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. Figure 5 shows the results of this analysis. The above analysis indicates that on average the extent of blast induced damage was 1.45m and 1.50m at right side and left side tunnel walls, respectively.

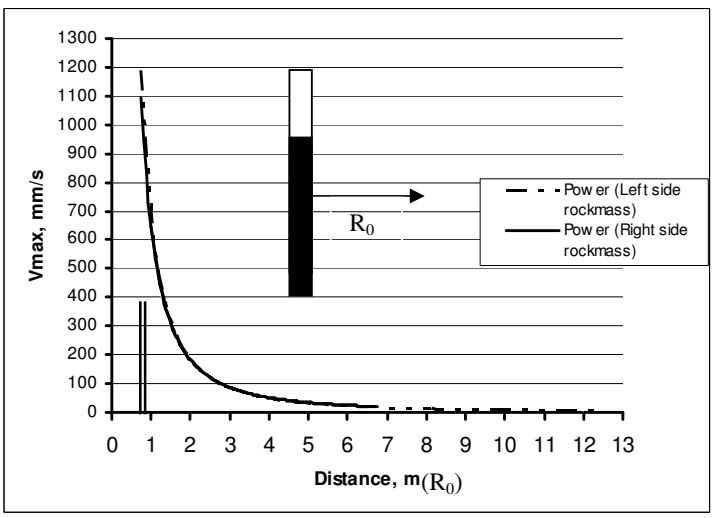

Fig. 5 - Results of the analysis by using H-P model

The damage levels assessed by H-P 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. A total of two borehole camera surveys were taken, one each at right side and left side tunnel walls. All the pictures were sorted out first by editing and matching the same images before and after each blast. Consequently, the pictures, which showed differences in fracture existence, were identified by their position coordinates. The images of observation holes captured by borehole camera clearly indicated that the near-field damage due to production blasts extended up to 1.61m and 2.18m at right side and left side tunnel walls respectively. Near-field damage observations of both the methods are given in Table 3.

Table 3 - Extent of predicted rock mass damage into the tunnel wall

S.No.	Rock type	K	α	V_{cr} from H-P model	Extent of damage from H-P model	Extent of damage from borehole camera survey
1	Dolomite with acute joint orientation	688	0.91	402 mm/s	1.45 m	1.61 m
2	Dolomite with obtuse joint orientation	646	0.83	384 mm/s	1.50 m	2.18 m

5.2 Far-Field Blast Damage due to Repeated Vibrations

The multiple rounds of blasting activity was carried out for the power house room excavation at the downstream side of the penstock tunnels, which was in the close proximity of the tail end of penstock tunnel, where the monitoring instruments were installed. Far-field rock mass damage observations were carried out by using borehole camera and by borehole extensometer. In order to correlate the far field damage with the vibrations, the V_{max} levels were recorded for every blast round till the vibration intensity attenuated to about 45mm/s. The borehole camera observation holes were under continuous monitoring for more than 50 rounds of blasts at the instrument location at both right side and left side of the penstock tunnel. As the blast site is moving away from the monitoring point the vibration intensity, obviously, reduced gradually and vise versa. 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. The blast damage assessment at right side and left side of the penstock tunnel is discussed in the following sections separately.

5.2.1 Damage assessment of rock mass with acute angle joint orientation

Peak particle velocities versus number of occurrences of dynamic loading at dolomite rock mass with acute angle joint orientation is shown in Fig. 6. The vibration intensity recorded was ranging from 50 to 1100 mm/s at the monitoring location. The displacements of extensometer rods of 3 and 4m depth (Fig. 7) at the left side tunnel wall were proportional to the V_{max} for few near-field blast rounds of main tunnel blasts. There was no considerable change in the extensometer readings of these rods for some of the V_{max} levels below V_{cr} (402 mm/s). After 42 rounds of blast occurrences, the displacements were again observed even at the vibration levels 301.54 mm/s, which is below the V_{cr}. There were no displacements observed below the V_{max} level of 106mm/s. This effect gives an indication that the damage zone was below 3m from the tunnel perimeter.

The exact depth of damage zone inside the tunnel walls could not be found with extensometers. Therefore, borehole camera observations were used for determination of exact depth of damage as well as extent of crack network and fracture frequency. As the range of damage extension was already assessed by means of extensometer, the inspection of exact extension of damage by borehole camera had become much easier. Images were captured by an interval of 1cm, within the range of probable damage extension for precise

inspection of rock mass damage. The images of borehole sections which contain the interface of intact and disturbed rock mass of tunnel wall are shown in Fig. 8. The damage depth measured by the borehole camera survey at the left side wall with acute joint orientation was 2.85m.

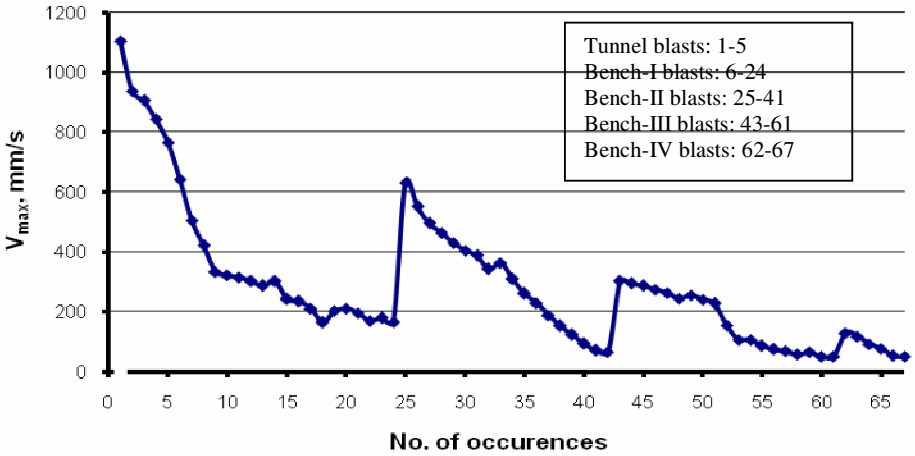

Fig. 6- Peak particle velocities versus no. of occurrences of dynamic loading at dolomite rock mass with acute joint orientation

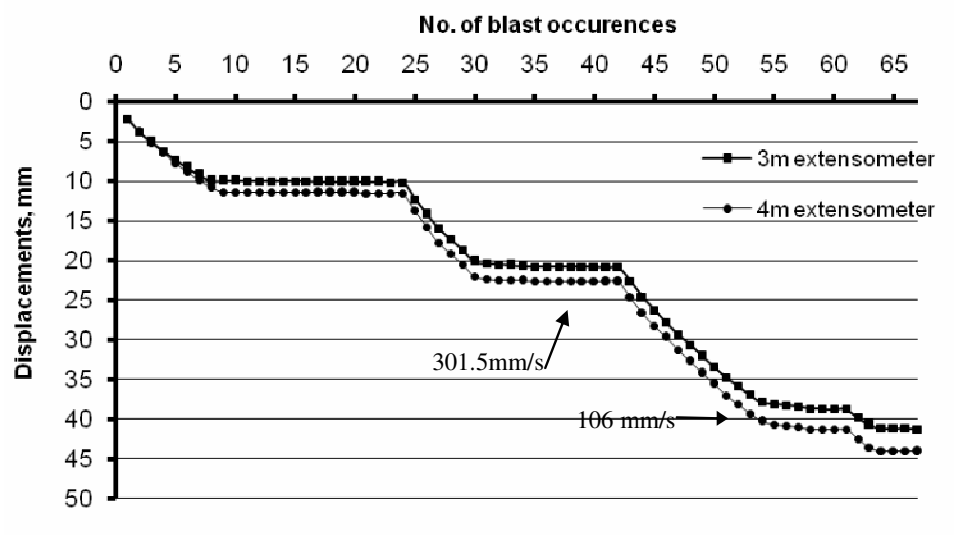

Fig. 7 - Plastic deformations of extensometer rods at dolomite rock mass with acute joint orientation

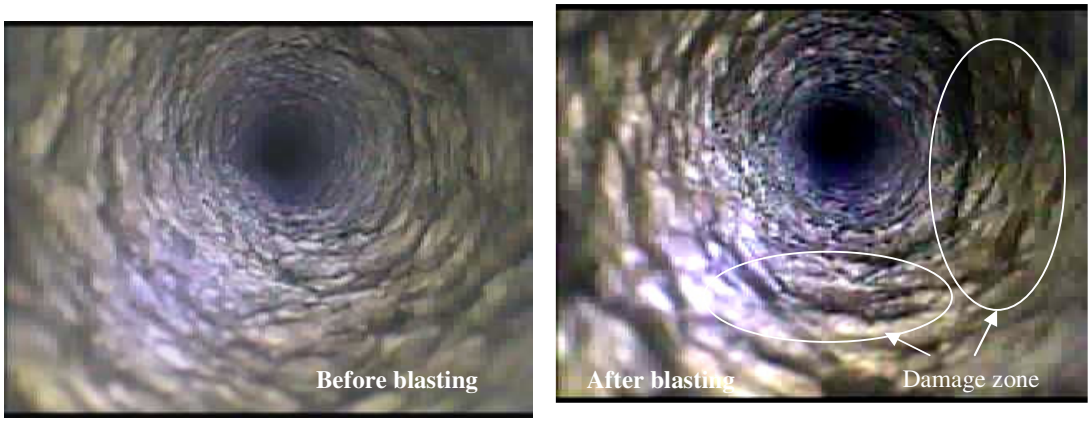

Fig. 8 - Images of intact and disturbed rock mass with acute joint orientations captured by borehole camera

5.2.2 Damage assessment of rock mass with obtuse angle joint orientation

Peak particle velocities versus no. of occurrences of dynamic loading at Right side of the tunnel is shown in Fig. 9**.** The vibration intensity recorded was ranging from 40 to 1014 mm/s at the monitoring location. The displacements indicated by the extensometer rod of 3.5m depth (Fig. 10) at the Right side wall were proportional to the V_{max} for few blast rounds of the tunnel which produced vibrations above V_{cr} (384mm/s). After 5 blast rounds it was observed that there were no displacements observed even though the V_{max} recorded was above V_{cr}. This effect gives an inference that the damage zone extended beyond the anchor point of the 3.5m extensometer rod after few close field blast loadings. This might be the reason why the 3.5m extensometer rod did not respond to even the vibrations levels which are greater than the critical vibration levels. The displacements of the 4.5m extensometer rod (Fig. 10) were proportional to all the V_{max} values of blast rounds which are above V_{cr}. There was no considerable change in the extensometer readings of these rods for some of the V_{max} levels below V_{cr}. After 36 rounds of blast occurrences, the displacements were again observed even at the vibration level of 240.6 mm/s, which is below the V_{cr}. There were no displacements observed below the V_{max} level of 86 mm/s. This phenomena gives an inference that the anchor point of 4.5m rod of the extensometers was in elastic zone and the damage zone was below 3m from the tunnel perimeter.

The borehole inspection survey was conducted for capturing the damaged rock strata due to repeated blasting at Right side of tunnel wall. The images of borehole sections which contain the interface of intact and disturbed rock mass are shown in Fig. 11. The maximum damage depth measured by the borehole camera survey with obtuse joint angle orientation was 3.74m. The far-field damage due to repeated loading at both the sides of tunnel wall is given in Table 4.

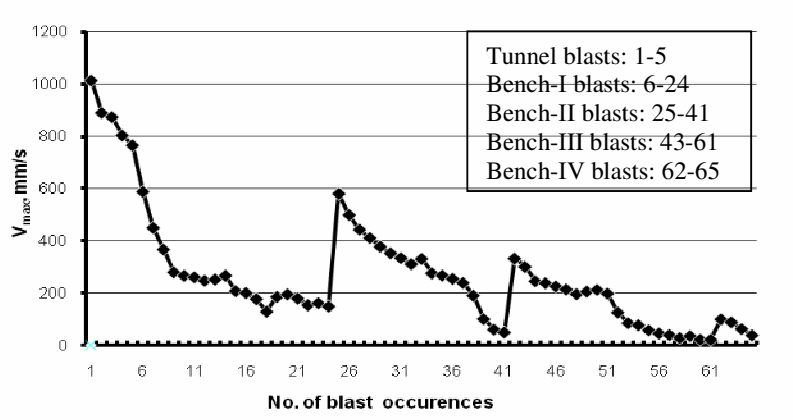

Fig. 9 - Peak particle velocities versus number of occurrences of dynamic loading at dolomite rock mass with obtuse joint orientation

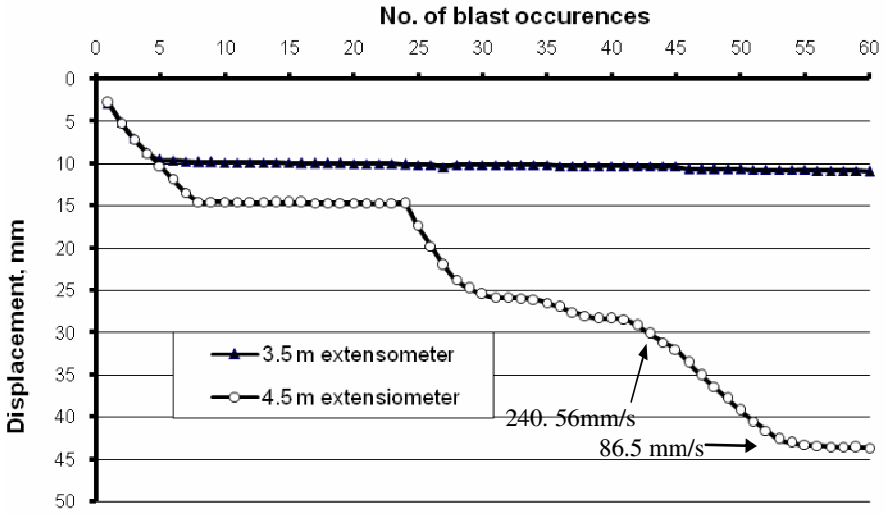

Fig. 10 - Plastic deformations of extensometer rods at dolomite rock mass with obtuse joint orientation

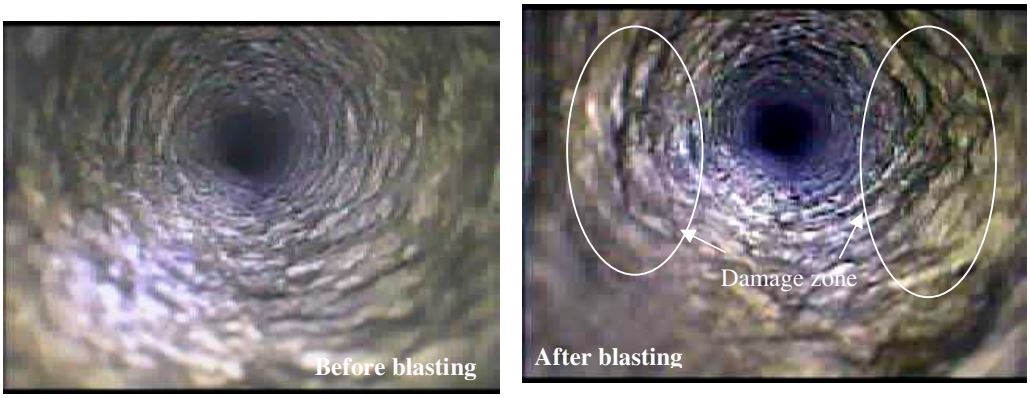

Fig. 11 - Images of intact and disturbed rock mass with obtuse joint orientations captured by borehole camera

6. RESULTS AND DISCUSSIONS

The near field blast loading due to main tunnel 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 due to main tunnel and powerhouse room excavation, blast loading could produce displacements in the rock mass even at lower levels of vibrations than the V_{cr}. The extra damage due to repeated blast loading was 1.24m and 1.56m at the left side and right side of the tunnel walls respectively.

After the occurrences of 48 numbers of blast rounds with the V_{max} levels ranging from 50-1700 mm/s, considerable displacements observed were in the borehole extensometer even at the V_{max} level of 106 mm/s, which is approximately 26.4% of V_{cr} in the dolomite rock mass with acute joint orientation. Similarly, the displacements observed even at the V_{max} level of 86 mm/s i.e. at approximately 22.4% of V_{cr} in the rock mass with obtuse joint orientation, after 41 numbers of occurrences of blast loading. These results are well in line with the observations of Dowding and Rosen (1978). The threshold vibration limits with number of cycles of repeated loading for two different rock mass are given in Table 4.

Table 4 - Far-field damages due to repeated loading at both the sides of tunnel wall

Tunnel Wall	Rock type	No of cycles of blast loading	Threshold vibration limits, mm/s	Maximum extent of damage, m
Left side	Dolomite with acute joint orientation	48	106	2.85
Right side	Dolomite with obtuse joint orientation	41	86	3.74

The study also revealed that the overall damage was about 80% more at tunnel wall with obtuse angle joint orientation in comparison to the acute angle joint orientation. The observations also indicate that the repeated dynamic loading resulted in the damage at the vibration levels even at 22 % of V_{cr}. These observations were almost similar to the findings of Adamson and Scherpenisse (1998), which say that threshold vibration level falls down to 25% of V_{cr} in repeated loading conditions. The findings of the study clearly indicate that the phenomena of repeated blasting with respect to number of cycles of loading should be taken into consideration for proper assessment of comprehensive blast induced damage.

7. CONCLUSIONS

A comprehensive blast monitoring program aimed at the investigation on the effect of repeated blast induced damage due to near-field and far-field vibrations was conducted in a Dolomite rock mass. Estimates of the maximum extent of rock mass damage made through the application of the Holmberg-Persson model compared well with measured results, although the former one was at lower side of damage. Modelling approaches like Holmberg-Persson model provides engineers with practical methods to model peak particle velocity attenuation. The damage levels predicted for near-field blast loading by Holmberg-Persson model were 1.45 m and 1.5m for the left side and right side rock mass, respectively. The damage levels measured for the same rock mass by borehole camera was 1.61m and 2.18m for right side and left side rock mass respectively. There is deviation close matching with both the assessment methods because of the reason that the intact rock properties were used in the equation of tensile failure criteria rather than the rock mass properties.

The study also showed that repeated dynamic loading imparted on the exposed tunnel from subsequent blasts in the vicinity is going to contribute to rock mass weakening and preconditioning. After 48 repeated blast rounds, the threshold vibration level for the dolomite rock was found to be 106 mm/s. The dynamic loading due to repeated blasts resulted in 77.0% and 71.6% of extra damage, in addition to the near-field damage, in Right side wall and Left side wall of the tunnel walls respectively. The repeated dynamic loading also resulted in reduced threshold peak particle velocity to 26 % and 24% of critical peak particle velocity for extension of damage at Right side wall and Left side wall of the tunnel, respectively. The study also revealed that the overall damage was about 80% more at tunnel wall with obtuse angle joint orientation (joint dipping towards the tunnel) in comparison to the acute angle joint orientation (joint dipping away from the tunnel). The findings of the study clearly indicate that the phenomena of repeated blasting with respect to number of cycles of loading should be taken in to consideration for proper assessment of blast induced damage in underground excavations.

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