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Investigations of Rock Mass Damage Induced by Blasting in Tunnelling

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ABSTRACT

Drilling and blasting is preferred method of rock excavation world-wide due to low initial investment, cheap explosive energy, easy acceptability among the blasting engineers, possibility to deal with different shapes and sizes of openings. Although, drill and blast method has witnessed significant technological advancements, it has inherent disadvantage of deteriorating surrounding rock mass due to development of network of fine cracks in it leading to safety and stability problems. The damage in the peripheral rock mass culminates in the form of overbreak and damaged zone. Overbreak increases project cost by more than 15%. The damaged zone extends beyond overbreak. Although significant efforts have been made to assess damage to the surrounding rock mass using different methods, easier solution based on easily available site parameters is still lacking. Authors have carried out field investigations at five different tunnels located in Himalaya, India to formulate an empirical correlation for prediction of blast induced damage for wide range of rock mass quality Q values (0.04 - 17.8). The proposed correlation is based on specific charge, perimeter charge factor, maximum charge per delay, advancement and confinement factor and rock mass quality rating Q. All the parameters used in empirical correlations are readily available to the site engineers and does not require laboratory testing. Data sets of 113 experimental blasts are collected from the five tunnel sites. The proposed empirical correlation has been validated using ultrasonic tests on rock core samples obtained from one of the experimental location.

Keywords: Rock mass damage; Blasting; Overbreak; Maximum charge per delay; Vibration

1. INTRODUCTION

Drill and blast method (DBM) is the most commonly used technique for breaking rocks and minerals in mining, quarrying, tunnelling and other excavation works across the globe. DBM is cheaper than the other available mechanical methods such as Rock Breakers, Tunnel Boring Machines and Road Headers etc. especially with regards to tunnels excavation. DBM offers degree of flexibility over Tunnel Boring Machines (TBM). Low initial investment, cheap explosive energy, easy acceptability among the blasting engineers, possibility to deal with different shapes and sizes of openings and reasonably faster advance rate in a suitable geotechnical mining condition collectively make DBM preferred method of rock excavation (Innaurato et al., 1998).

Although DBM has witnessed significant technological advancements, it has the inherent disadvantage of deteriorating surrounding rock mass due to development of network of fine cracks leading to safety and stability problems. Blasting for underground excavation and tunnelling is a difficult operation compared to open pit excavation due to unavailability of the free face. Rock mass damage is a common problem in tunnelling (Gupta et al., 1988; Kadkade, 1991; Adhikari and Babu, 1994; Roy, 2005). Practicing engineers involved in rock excavation, would try to achieve faster advancement in tunnel and underground excavation by employing drill jumbos which significantly increase drilling accuracy and reduce drilling time. Although, faster advancement rate may be achieved using greater amount of explosives leads to greater extent of blast induced rock mass damage (Murthy and Dey, 2003).

Blast induced rock mass damage has been studied by various researchers such as Langefors and Kihlstrom 1973; Bauer and Calder, 1978; Holmberg and Persson, 1979; Singh, 1993; Scoble et al., 1997; Bäckblom and Martin, 1999; Raina et al., 2000; Singh and Xavier, 2005; Warneke et al., 2007; Ramulu et al., 2009 and Fu et al., 2014. Damage around an opening in underground has been described by using terminology such as blast induced rock mass damage (BIRD), blast induced damage (BID), excavation damage zone (EDZ), rock mass damage zone (RMD) etc. Rock mass damage zone surrounding an underground opening consists of overbreak zone (failed zone), damaged zone and a disturbed zone. The three zones of damage are shown in Fig. 1

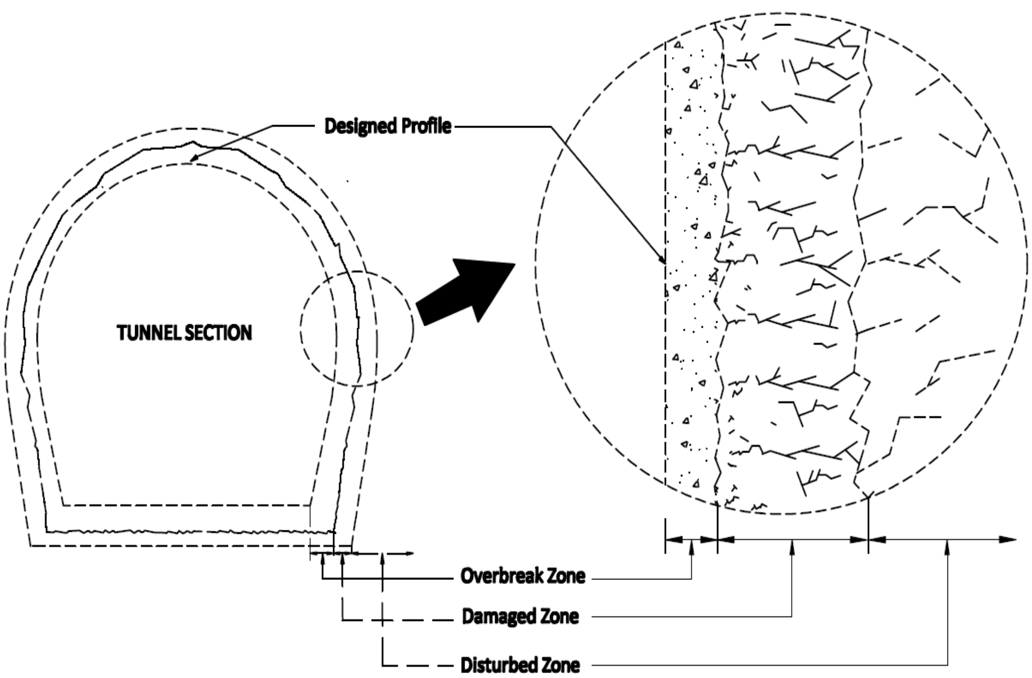

Fig. 1 - Blast induced rock mass damage zone around an underground opening

The overbreak zone represents the zone beyond the minimum excavation line of the designed periphery from where rock blocks/slabs detach completely from the rock mass. It is a measure of difference in excavation between ‘as designed profile’ and ‘as excavated profile’. Overbreak zone is undesirable and leads to cost overrun due to extra excavation and backfilling, shotcrete, concrete or other material as per designed support system. Overbreak varies from 5 % to 30% which incurs significant cost and increases cycle time of the tunnelling operation.

The damaged zone is a zone around tunnel beyond overbreak zone. The irreversible changes in the rock mass properties take place in this zone due to presence of fine networks of micro-cracks and fractures induced by the blasting and excavation process. This zone is characterized by deterioration in mechanical and physical properties and increase in transmissivity properties.

The disturbed zone is a zone in the rock mass immediate beyond the damaged zone where changes in the rock mass properties are insignificant and reversible. This zone is dominated by changes in stresses and hydraulic permeability.

Overbreak as well as damaged zone has significant impact on the project cost, construction period, safety and performance of the underground structures. In the case of the civil construction tunnels, damaged zone can adversely affect the stability of underground openings and hence the need to be accounted for while designing support systems for openings. Enlarged extent of the damage zone endangers safety of the front line workers as it may considerably reduce stand-up time of the rock mass. Functionality and post-construction performance of the structure will also be affected with enlarged extent of the damage zone.

In light of above observation, efforts have been made in this study to develop empirical correlations for estimation of extent of damaged zone based on data obtained from the field investigations.

2. FIELD INVESTIGATIONS

Field experiments have been carried out to gain insight of these influencing parameters at five tunnel construction sites. The sites are integral parts of three major hydro power projects located in Hiamalya. These sites are Access Tunnel AA10R and AA7 from Pump Storage Plant (PSP) Project of THDC India Limited, Tehri; Head Race Tunnel (HRT) of Singoli-Bhatwari Hydroelectric Power Project (SBHEP),Rudraprayag; HRT and Bypass Tunnel (BPT) of Tapovan Visnhnugaad Hydroelectric Power Project (TVHEP), Tapovan. All are located in Uttarakhand state of India. The data was obtained from 113 blasts undertaken at five tunnel construction sites. Rock mass characterisation, blast vibration monitoring, overbreak assessment and estimation of damaged zone was carried out during each blast operation.

All the experimental blasts under observation were closely monitored. The data for drilling patterns including spacing and burden, (with emphasis on holes in perimeter and penultimate row and hole depth) was collected during drilling operations. Parameters relating to blasting such as explosive consumption in a hole as well as total round, initiation system and firing sequence, maximum charge per delay were recorded during charging of holes in a blast round. Record of pull in each round was obtained after surveying of tunnel profile and advancement. Parameters such as total charge used in a blast round (T), maximum charge per delay (W), Pull (l), hole depth (d) were directly available from the records. Other parameters such as such as advancement factor, confinement factor and perimeter charge factor were calculated from the recorded observation for each round of blasts. In the proposed correlation, Specific Charge (q), Maximum Charge per Delay (W), Perimeter Charge Factor (q_p), Advancement Factor (A_f) and Confinement Factor (C_f) have been used to represent the blasting operation in underground excavation. The parameters used are described below.

- Specific Charge (q) (kg/m^3): Specific charge is defined as ratio of total quantity of explosive used and rock volume broken. It is expressed in kg/m^3.
- Maximum Charge per Delay (W) (kg): It is maximum quantity of explosive fired in a delay series. This is obtained from record of delay distribution in a blast round and charging pattern in each hole.
- Perimeter Charge Factor (q_p) (kg/m^3): Similar to specific charge, perimeter powder factor is the quantity of explosive used in perimeter holes and the volume of rock corresponding to burden of the contour holes.
- Advancement Factor (A_f): It is ratio of pull (l) and hole depth (d) in a blast round.
- Confinement Factor (C_f): It is ratio of hole depth (d) and cross-sectional area of tunnel (a).

Rock mass characterisation has been carried out using Barton's rock mass quality, Q. Q-system has been recommended specifically for tunnels and caverns with an arched roof. It is observed that Q-system is preferred method of rock mass classification for civil construction such as tunnels and caverns. A large number of field and design engineers as well as geologists are using the Q-system for various purposes such as support design and engineering analysis of rock mass. Prevailing stress environment influences damage to the surrounding rock mass. In Q-system, stress reduction factor (*SRF*) is one of the parameters which accounts for active stresses during construction of an underground opening. Therefore, Q-system has been selected for rock mass characterisation in the study.

Rock mass of the experimental tunnel sites varies from poor class to good rock class (Q values range from 0.03 to 17.8). This range of rock mass classes represents commonly encountered geo-mining conditions for the civil construction industry. Therefore, the suggested method could be applicable to a wide range of rock mass conditions encountered by tunnel construction industry in the non-squeezing ground condition.

3. ESTIMATION OF DAMAGE DISTANCE (D_d)

During field investigation in each tunnel, vibration monitoring has been carried out at all the sites for determination of attenuation characteristics of blast induced ground vibration. Blast induced ground vibration was measured using three different engineering seismographs namely MinimatePlus (MMP), Minimate Blaster (MMB) and Minimate (MM), (manufactured by Instantel, Canada). Monitoring of the blast induced ground vibration was carried out as per the guidelines given in IS:14881 (2001), ISRM suggested method (ISRM, 1992) and Instantel Minimate user manual (Instantel Manual, 2009). Ambraseys and Hendron (1968) developed a model of blast induced ground vibration attenuation for spherical charge geometry. The suggested blast vibration prediction model is given by Eq. 1.

$$\boldsymbol{V_{ppv}} = \boldsymbol{K} \left(\frac{\boldsymbol{R}}{\sqrt[3]{\boldsymbol{W}}}\right)^{-\boldsymbol{\beta}} \tag{1}$$

where

$\boldsymbol{V_{ppv}}$ = Peak particle velocity, mm/s,
$\boldsymbol{K}, \boldsymbol{\beta}$ = Site constant (function of characteristics of propagating media),
$\boldsymbol{R}$ = Distance of measurement, m and
$\boldsymbol{W}$ = Maximum charge per delay, kg.

The constant $\boldsymbol{K}$ and $\boldsymbol{\beta}$ have been determined using least square regression analysis for all the sites. In the present study, the attenuation characteristic is derived at 95% confidence interval and same is used for prediction of the blast vibration at all the five sites. The attenuation characteristics of the vibration obtained at five sites has been presented in Table 1.

Rock mass damage is a result of the induced dynamic stress during detonation. For an elastic medium, induced dynamic strain can be calculated as a function of peak particle velocity (V_{ppv}) and longitudinal wave velocity ($\boldsymbol{V_P}$). Therefore, blast induced damage arising in the rock mass is widely correlated with peak particle velocity of blast induced vibration. In this study, extent of damage to the surrounding rock mass is calculated using critical peak particle velocity ($\mathbf{v_{cr}}$) (Singh, 1993; Forsyth, 1993; Holmberg and Persson, 1978; Kwon et al., 2009). Critical peak particle velocity for each of experimental predominant rock mass is obtained using Eq. 2.

$$\boldsymbol{V_{cr}} = \frac{\boldsymbol{V_P\,\sigma_t}}{\boldsymbol{E}} \tag{2}$$

where

V_{cr} = Critical peak particle velocity, mm/s
$\boldsymbol{V_P}$ = P-wave velocity of rock, m/s
$\boldsymbol{\sigma_t}$ = Tensile strength of rock, MPa, and
$\boldsymbol{E}$ = Young's modulus of rock, MPa.

Table 1- Geotechnical properties of rock in experimental tunnels
(Source: Detailed project report of respective Projects)

Sl. No	Experimental Tunnel Site	Predominant Rock Type	Vibration Attenuation Equations	σ_t MPa	V_p m/s	E MPa	V_{cr} mm/s
1	HRT SBHEP	Quartz Biotite Schist	$\boldsymbol{V_{ppv} = 1825.1\left(\frac{R}{W^{0.33}}\right)^{-1.20}}$	6.71	3267	12600	1739.8
2	HRT TVHEP	Augen Gneiss	$\boldsymbol{V_{ppv} = 1390.5\left(\frac{R}{W^{0.33}}\right)^{-1.598}}$	8.7	5400	27900	1683.8
3	BPT TVHEP	Quartzite	$\boldsymbol{V_{ppv} = 2079.1\left(\frac{R}{W^{0.33}}\right)^{-1.334}}$	12.4	6200	55500	1754.5
4	AA7 PSP	Phyllitic Quartzite Thinly Bedded (PQT)	$\boldsymbol{V_{ppv} = 441.9\left(\frac{R}{W^{0.33}}\right)^{-1.16}}$	4.3	5400	10500	221.65
5	AA10R PSP	Phyllitic Quartzite Massive (PQM)	$\boldsymbol{V_{ppv} = 576.2\left(\frac{R}{W^{0.33}}\right)^{-1.03}}$	7.2	6000	12700	340.15

Notations: *σ_t - Tensile strength, V_p -P-wave velocity, E- Young Modulus; V_{cr} - Critical peak particle velocity; SBHEP -Singoli-Bhatwari Hydroelectric Project; TVHEP- Tapovan Vishnugaad Hydroelectric Project, Tapovan; PSP- Pump Storage Plant Project, Tehri.*

Geotechnical properties of the predominant rock mass encountered at experimental tunnel site are presented in Table 1. The critical peak particle velocity ($\mathbf{V_{cr}}$) value was obtained using Eq. 2 for each experimental sites. The damage distance from a blast round can be back calculated using Eq. 1. Following this method, damage distance for all the observed blasts has been obtained.

4. ANALYSIS OF THE DATA

The data obtained was analysed to gain insight in to the contribution of the individual parameters in the damage induced by the blasting operation.

Figure 2 shows the variation of average damage distance (Dd) with rock mass quality Q. In general, the damage distance decreases with the increase in Q-values. The average damage distance for rock mass having Q-value less than 1 is greater than 5.0 m. It sharply reduces to approximately 3.0 m for rock mass with Q-value greater than 4. The impact of rock mass quality is significant in lower classes of the rock mass having Q-value less than 4. In higher classes of rock mass, impact of the rock mass quality remains fairly uniform and possibly other parameter of blast design play pivotal role in defining damaged zone around an opening.

In any underground blasting operation, progressive enlargement of the free face shall be achieved by designing the firing sequence of holes using different delay series. Proper distribution of the delay series ensures free face to each hole. The holes fire in the direction of free face thus utilising the explosive energy in breaking and displacing the rock. The maximum charge per delay, *W* depends on the number of delay series used in a blast round. Improper delay distribution gives excessive burden and spacing to the holes which leads to generation of the blast induced ground vibration resulting into greater extent of damaged zone. Thus maximum charge per delay, *W* influences significantly the damage distance among other parameters.

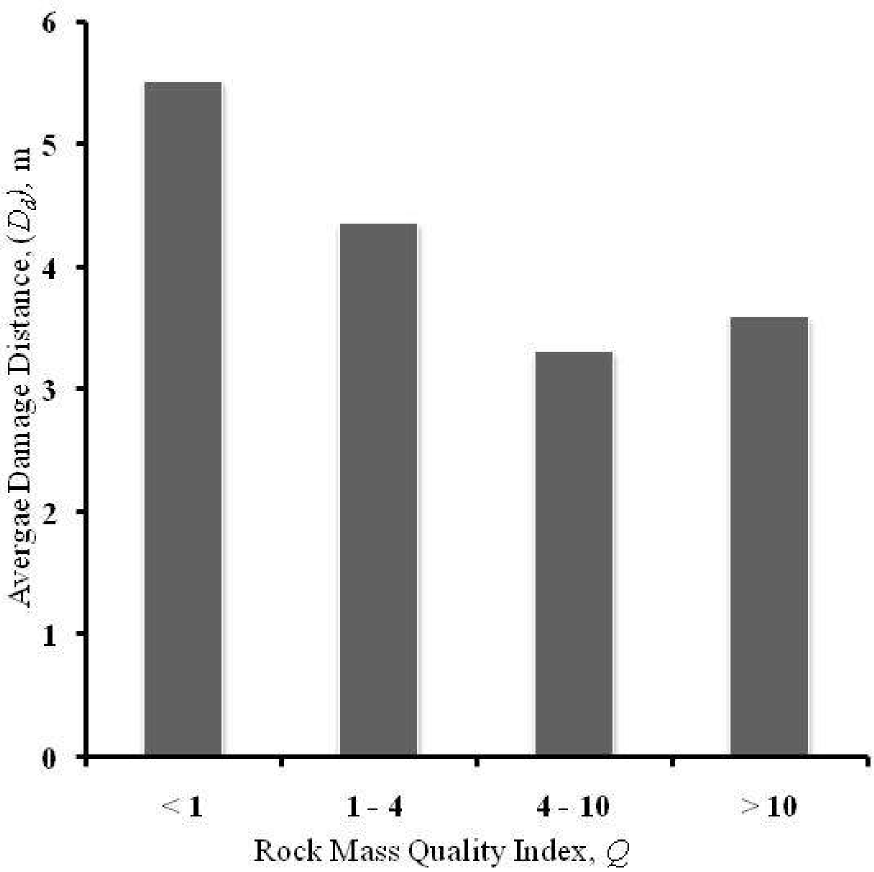

Fig. 2 - Variation of damage distance (Dd) with rock mass quality index, Q

The variation of observed average damage distance with maximum charge per delay is shown in Fig. 3. In experimental blast rounds, range of W varied between 15 kg to 45 kg. Therefore, a range of 15 to 45 kg maximum charge per delay, W has been considered in the analysis. It may be noted from Fig. 3 that the damage distance increases with increase in maximum charge per delay.

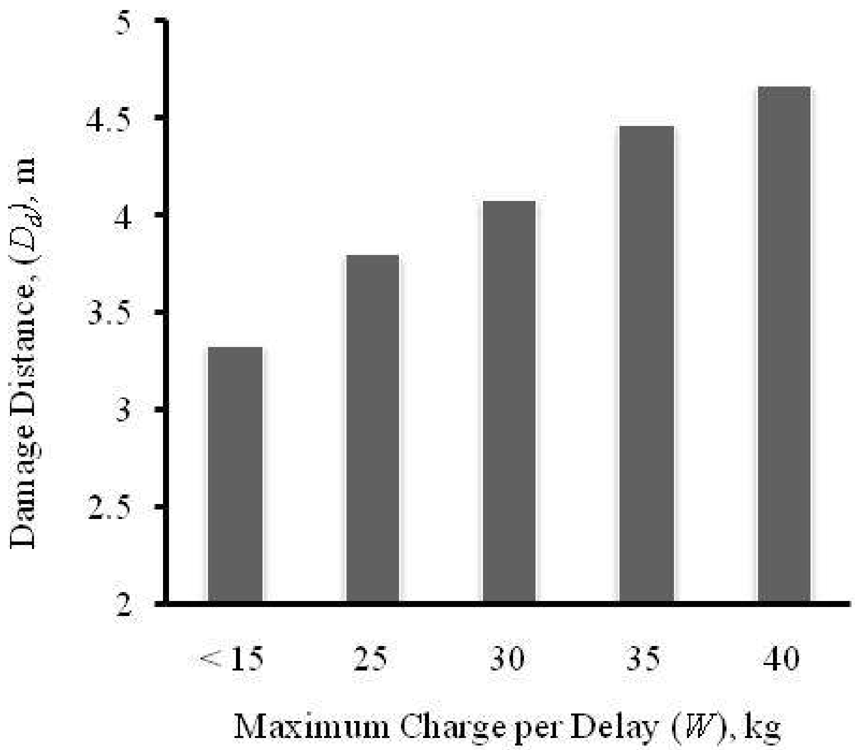

Fig. 3 - Variation damage distance (D_d) with maximum charge per delay (W)

Figure 4 is a plot of damage distance with a normalized advancement factor. The advancement factor is normalized with rock mass quality index to make data comparable for different rock mass conditions. In general, damage distance increases with increase in normalized advancement factor. This can be attributed to the fact that in experimental blasts, higher advancement has been achieved in most cases due to increase in either hole depth or total charge used in blast round. Higher value of total charge in a blast round will increase advancement but will also cause more damage to the rock mass.

Contrary to this observation, an increase in advancement factor by optimizing blast design parameter for a given size of tunnel should ideally decrease the damage distance. The reason behind the decrease in damage distance may be due to the fact that the increase in advancement factor also leads to optimum utilization of the explosive energy which otherwise would have converted in blast induced ground vibration.

In blasting operation, explosive energy appears in three forms, maximum charge per delay (W), specific charge (q) and perimeter charge factor (q_p). Whereas maximum charge per delay is influenced by the initiation and firing sequence of the blast round, perimeter charge factor and specific charge are dependent on advancement of a blast round. For a fixed amount of explosive energy, greater advancement rate reduces perimeter charge factor as well as specific charge.

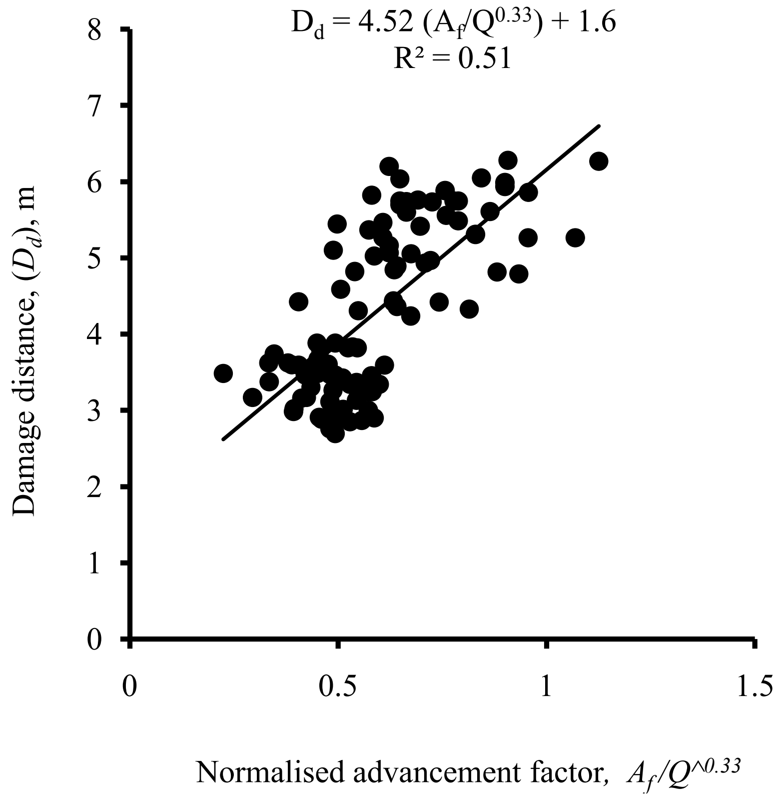

Fig. 4 - Variation of damage distance with normalized advancement factor

All the three explosive energy parameters W, q_p and q are grouped as factor Z as given in Eq. 3

$$Z = q_p^{0.15}\sqrt{W+q} \tag{3}$$

where

q_p = Perimeter charge factor, kg/m^3,
W = Maximum charge per delay, kg, and
Q = Specific charge, kg/m^3.

Figures 5 and 6 are plots of damage distance with factor Z for an experimental tunnel with Q values less than 1.67 and greater than 1.67 respectively.

In case of tunnel AA10R and AA7, rock mass was of lower quality whereas in other three experimental tunnels Q values were higher than 1.67. Data for these two cases are plotted separately due to difference in rock mass classes and also in blasting practices. In experimental tunnel AA7 and AA10R rock mass are of very poor category, cross-sectional area are higher and hole depth is less than 2.5 m. In other tunnels, rock masses are of better quality, tunnel cross-sectional area is less than 40 m^2 and hole depth were greater than 3.0 m.

As shown in Figs. 5 and 6, factor Z is directly proportional to the damage distance. In both the cases a correlation coefficient of 0.83 is obtained. Maximum charge per delay and specific charge are measures of explosive energy in the blast round whereas the perimeter charge factor is parameter introduced to measure the damage created by the explosive energy in the contour hole.

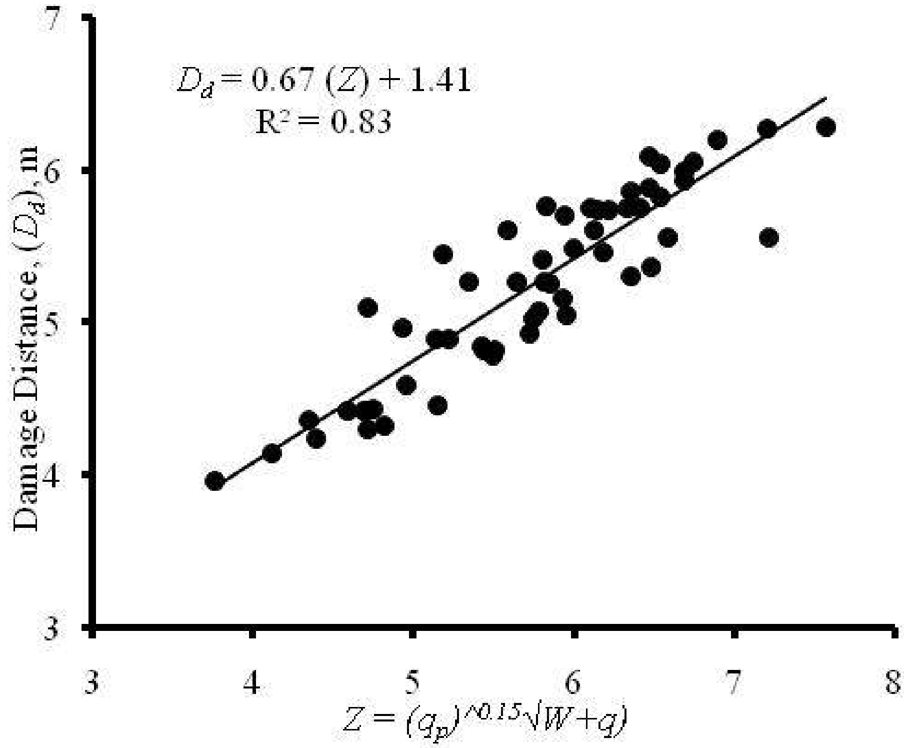

Fig. 5 - Damage distance versus Explosive Energy Parameters for $Q < 1.67$

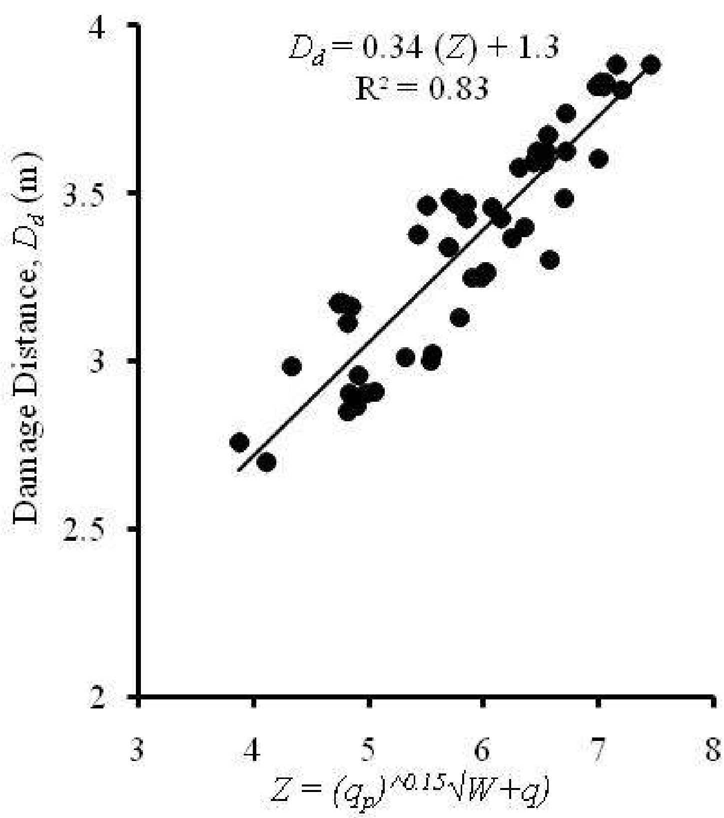

Fig. 6 - Damage distance versus Explosive Energy Parameters for $Q > 1.67$

In a blasting operation in underground excavation, contour holes are fired in the last delay series. All the holes are assigned same delay and spacing in these holes are lesser than the burden of the contour holes. Such firing arrangement creates a fracture line along the final excavation line. Such arrangements of delay for periphery holes, although these holes are lightly charged, contribute as maximum charge per delay due to large number of holes fired in same delay time on several instances. These conditions compound the effect of maximum charge per delay with perimeter powder factor.

Although the periphery holes provide a line of fracture along final line of excavation, due to it initiation in the last delay series, the effect of *W* is not restricted by the lightly charged contour holes. Therefore, the damage distance is enhanced by the perimeter charge factor.

5. EMPIRICAL CORRELATION FOR PREDICTION OF DAMAGE DISTANCE (D_d)

As discussed in the preceding section, damage distance is influenced by the blast design parameters and rock mass quality. A regression analysis of various parameters considering the general trend of the field data and also the damage mechanics was performed to develop empirical correlation for prediction of damage distance, D_d.

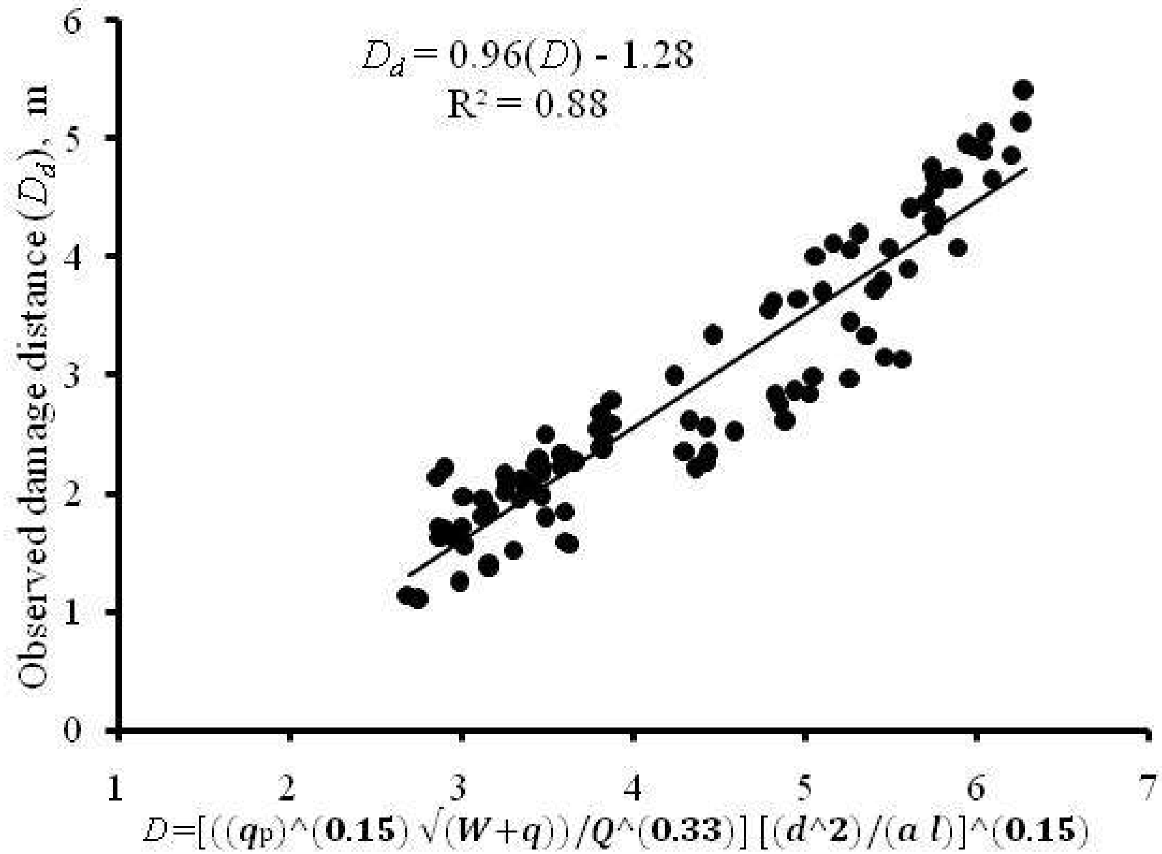

Fig. 7 - Plot of factor D and observed Damage Distance, D_d

Figure 7 shows plot of a factor D and observed damage distance (D_d). It is a plot for all the observed D_d values obtained during field investigation at five tunnel construction sites.

$$\boldsymbol{D} = \frac{q_p^{0.15}\sqrt{W+q}}{Q^{0.33}}\left(\frac{d^2}{a\,l}\right)^{0.15} \tag{4}$$

or

$$\boldsymbol{D_d} = \mathbf{0.96}\left[\frac{q_p^{0.15}\sqrt{W+q}}{Q^{0.33}}\left(\frac{d^2}{a\,l}\right)^{0.15}\right] - \mathbf{1.28}\ (R^2 = 0.88) \tag{5}$$

where
$\boldsymbol{D_d}$ = Damage distance, m,
$\boldsymbol{q_p}$ = Perimeter charge factor, kg/m^3,
W = Maximum charge per delay, kg,
q = Specific charge, kg/m^3,
Q = Rock mass quality index (Barton' Q-system)
d = Hole depth, m,
l = Pull of blast round, m, and
a = Tunnel cross-sectional area, m^2.

Equation 5 can be further simplified as Eq. 6 and 7

$$\boldsymbol{D_d} = \mathbf{0.96}\left[\frac{q_p^{0.15}\sqrt{W+q}}{Q^{0.33}}\left(\frac{\frac{d}{a}}{\frac{l}{d}}\right)^{0.15}\right] - \mathbf{1.28} \tag{6}$$

$$\boldsymbol{D_d} = \mathbf{0.96}\left[\frac{q_p^{0.15}\sqrt{W+q}}{Q^{0.33}}\left(\frac{C_f}{A_f}\right)^{0.15}\right] - \mathbf{1.28} \tag{7}$$

Confinement factor, $\boldsymbol{C_f} = \frac{d}{a}$

Advancement factor, $\boldsymbol{A_f} = \frac{l}{d}$

where
d = Depth of drill hole, m,
a = Tunnel cross-sectional area, m^2, and
l = Pull of blast round, m.

Equation 7 can be used for prediction of the damage distance induced by blasting in underground excavation.

In Eq. 7, explosive energy parameters, blast design parameters and rock mass quality are included. Equation 7 gives impression that energy parameters are used repetitively in the recommend correlation. It may be noted that all these three parameters are mutually exclusive. In a same blast design, values of W, q_p and q can be altered without changing other parameters. In blast round, having same drill hole depth and total charge, arrangement of firing sequence will change the values of W. Pull of the blast from such changed configuration will alter values of q_p as well as q. Inclusion of these three parameters will therefore be able to assess their effect on blast induced damage distance.

The correlation for assessment of damaged zone has been validated from ultrasonic test data of rock core samples obtained from ten locations from HRT of SBHEP project. Percentage reduction of P-wave velocity with depth was computed from the ultrasonic test data. As per Liu et al. (2009) and Fu et al. (2014), the threshold of damage is defined as a 10% reduction in the P-wave velocity as compared to the P-wave velocity of the undisturbed rock mass. The damage distance obtained from the ultrasonic test data were in close agreement with the predicted damage distance obtained using Eq. 7.

6. CONCLUSIONS

A comprehensive field investigation have been carried out at five tunnel construction sites to evolve empirical correlation for estimation of damage distance using readily available site parameter. Observations of 113 blasting experiment have been taken in different rock mass from extremely poor to good rock mass class. An empirical correlation has been suggested using Specific Charge (q), Maximum Charge per Delay (W), Perimeter Charge Factor (q_p), Advancement Factor (A_f) and Confinement Factor (C_f) and rock mass quality index Q.

It is suggested that the proposed correlation for estimation of damage distance may be used to fix a range of allowable blasting parameters based on the anticipated rock mass characteristics. Using proposed correlation, the extent of damaged zone may be computed for known values of Q and blast design parameters in the non-squeezing ground conditions. The rock mass ground support could also be designed by considering damaged zone.

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