Deterioration in Rock Mass Properties due to Blasting Operationsin Underground Excavations

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1.0 INTRODUCTION

Drill and blast method (DBM) is commonly used method of rock excavation world-wide due to lower capital requirement, ability to adjust with any shape and size of excavation and flexibility of the DBM system to deal with changing rock mass conditions. Although DBM has witnessed significant technological advancements, it has inherent disadvantage of deteriorating surrounding rock mass due to development of network of fine cracks leading to safety and stability problems. Rock mass damage zone surrounding an underground opening consists of overbreak zone (failed zone), damaged zone and a disturbed zone. The overbreak zone represents the zone beyond minimum excavation line of the designed periphery from where rock blocks/slabs detach completely from the rock mass. The damaged zone is a zone around the opening beyond overbreak zone where irreversible changes in the rock mass properties take place due to presence of fine networks of micro-cracks and fractures induced by the blasting and excavation process. Disturbed zone is a zone in the rock mass immediately beyond the damaged zone where changes in the rock mass properties are insignificant and reversible.

The damaged zone extends beyond overbreak zone and adversely affects safety of front line workers as well long term stability of the underground structures. The immediate effect of damage may not be visible but the problems may appear later and add to post construction maintenance. The service life of an underground structure will be also shortened due to blast induced damage. Therefore, support design engineers shall consider deterioration in rock mass quality with respect to type of blasting, for an effective control of overbreak and enhance safety and stability of underground structures. Field experiments have been undertaken at Head Race Tunnel (HRT) of L&T Singoli-Bhatwari hydroelectric power project, Rudraprayag to evaluate the effect of blast design parameters on the shrouding rock mass. Detail of the experiments, discussion of the results and important conclusion are presented in this paper.

2.0 DETAILS OF THE PROJECT SITE

Singoli-Bhatwari Hydroelectric Power (SBHEP) Project is a 99 MW (3 x 33MW) run-off-river scheme located in Rudraprayag district of Uttarakhand state in north India. General layout of the SBHEP project is shown in Fig 1. SBHEP project envisages construction of barrage for diversion of about 59.6 m^3/s of water, conveying it through desilting chamber, head race tunnel (HRT), surface powerhouse and then back to the riverthrough tail race channel (TRC). The project will have three units of Francis turbines, each of 33 MW giving a total installed capacity of 99 MW. Major components of the SBHEP project include 54 m wide and 20 m high barrage and intake structure, 57 m X 17.6 m surface power house, 168 m long surge shaft with 10.58 m diameter and 11.87 km of head race tunnel having finished diameter meter of 4.9 m. Entire rock excavation is planned to be carried out using drill and blast method. Experimental investigations for assessment of blast induced damage have been carried out at Head Race Tunnel (HRT upstream from Adit-III) of the project.HRT of the SBHEP project is D-shaped shaped, 5.6 m in size and 11.87 km in length.

Excavation of HRT is being carried out through five adits.Layout of Singoli-Bhatwari Hydroelectric Power Project is presented as Fig. 1. Amongst the four physiographic divisions of the Himalaya namely viz. Sub-Himalaya, LesserHimalaya (Outer and Inner), Great Himalaya and Trans-Himalaya, the area of SBHEP project, falls in theInner Lesser Himalaya division. The rock belonging to Garhwal Group, trending in N-S directionwith easterly dips,are exposedin the SBHEP project area. These exposed rocks abut against the NW-SE trending northeastdipping rocks of the Central Crystalline along a major tectonic plane - the MainCentral Thrust, in the north. In the south, the rock also abuts against E-W trending majorfault-the Alaknanda Fault (Kumar and Agarwal, 1975) which brings the rocks of theAgastmuni Formation in contact with the younger sequence of the Garhwal Group -the Rautgara Formation. The project area is mostly covered with colluvium and fluvioglacial material. A fewoutcrops are visible along the road cut section as well as along river and nalasections.

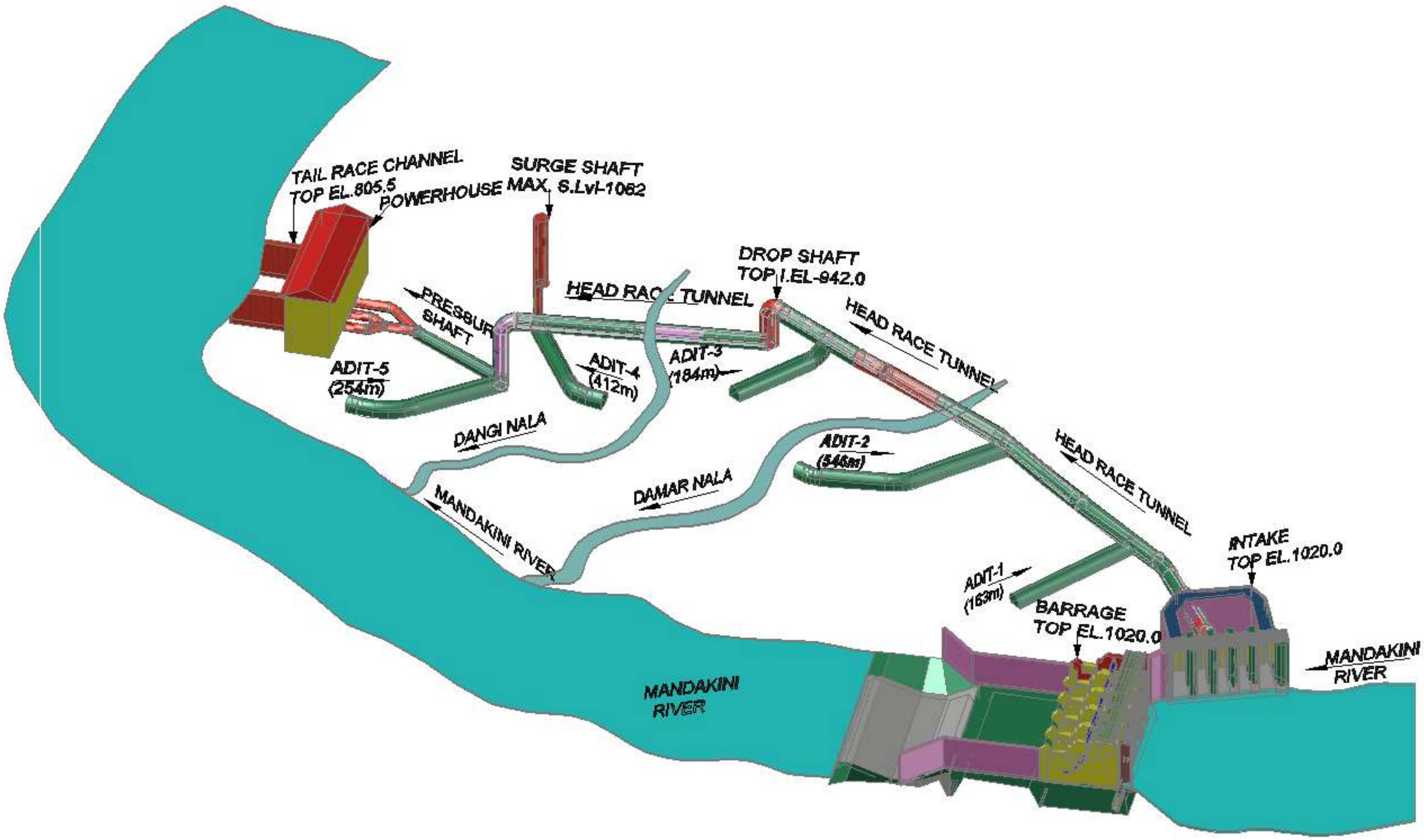

Fig.1: Layout of Singoli-Bhatwari Hydroelectric Power Project (L&T-SBHEP Detailed Project Report Vol. 4, 2007)

3.0 EXPERIMENTAL INVESTIGATIONS

Experimental investigations have been carried out in in Head Race Tunnel (HRT) between RD 368.0 to RD 628.3 m. The observations on blast rounds and associated rock mass parameters have been collected for 20 blasting rounds. The *Q*-values of the rock mass, encountered in the tunnel, are found to vary from 2.6 to 10.6. Rock mass cover of tunnel was less than 200 m in the experimental tunnel section.

Characterisation of rock mass at head race tunnel of the SBHEP project site has been carried out using Barton's *Q*-system (Barton et al., 1974). After each blast, the face mapping was done by evaluating Rock Quality Designation (*RQD*), Joint number (J_n), Joint roughness number (J_r), Joint alteration number (J_a), Joint water condition (J_w) and Stress Reduction factor (*SRF*). Rock mass quality parameter *Q* is calculated as per formulas suggested by (Barton et al., 1974). Various rock mass parameters, their rating and *Q*-values observed at experimental blast locations are presented in Table 1. The predominant rock mass encountered in HRT of the SBHEP project site is quartz biotite schist. The rock mass encountered at experimental blast locations are classified in fair and good rock mass category as per Barton et al. (1974).

Table 1: Parameters of Q-System observed at HRT of SBHEP Tunnel

Blast RD (m)	*RQD*	J_n	J_r	J_a	J_w	*SRF*	*Q*
22.0	58.9	12	2.0	2	1	1	4.9
25.0	58.9	12	2.0	2	1	1	4.9
350.4	92.0	12	1.5	1	1	1	11.5
353.3	92.0	12	1.5	1	1	1	11.5
359.0	65.0	12	1.5	1	1	1	8.12
424.8	62.2	12	1.5	3	1	1	2.6
431.4	75.0	12	1.5	1	1	1	9.4
472.1	85.0	12	1.5	1	1	1	10.6
474.7	80.0	9	1.5	1	0.66	1	8.8
478.0	80.0	9	1.5	1	1	1	13.3

3.1 Observations on Blasting Practices

The burn-cut blast design, having 45 mm hole diameter and 3.2 to 4.0 m, is used in the excavation of HRT in experimental blast rounds. Diameter of the large diameter Reamer holes is 76 mm diameter. Total number of holes in a blast round varied from 59 to 71. Emulsion explosive cartridges with 40 mm diameter and 90% strength were used in all the experimental blasts rounds. The length and weight of the emulsion cartridges were 200 mm and 390 gm respectively. Shock tube initiation system of long delay series were used in the blast rounds. Nominal delay time between successive delay series were 500 millisecond. Indicative drilling and blasting pattern used in excavation of HRT of SBHEP is shown in Fig2.

In a typical blast pattern, four square section burn cut pattern with four large diameter empty holes are drilled in all the blast rounds. In the contour area, alternate charged and dummy holes are provided to control overbreak and restrict growth of cracks beyond the excavation lines. These holes are spaced at a distance of 250 mm to 300 mm so that the charged holes are 500 mm apart. In a typical blast pattern, ten delay series were used. All the contour holes were fired in the last delay. All the experimental blasts under observation were closely monitored to obtain various influencing parameters. Parameters such as total charge used in 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, confinement and perimeter charge factors were calculated from the recorded observations for each round of blast. In the present study, Specific Charge (q), Maximum Charge per Delay (W), Perimeter Charge Factor (q_p), Advancement (A_f) and Confinement (C_f) Factors have been used to represent the blasting operation in underground excavation. The observed data pertaining to the blast design parameter are presented in Table 2. The measurement of overbreak has been carried out in HRT after each blast round. The minimum and maximum overbreak values observed at HRT were 6.8 % and 17.1 % respectively.

Table 2:Details of the Observed Drilling and Blasting Parameters

Blast RD (m)	Q	a (m^2)	d (m)	W (kg)	l (m)	q_p (kg/m^3)	q (kg/m^3)	OB (%)
22.0	4.9	27.77	3.2	38.1	2.8	1.83	2.83	31.34
25.0	4.9	27.77	3.2	33.2	2.6	1.81	2.42	28.21
350.4	11.5	27.77	3.2	35.1	2.9	1.43	2.30	14.10
353.3	11.5	27.77	3.2	35.1	3.0	1.05	2.27	11.62
359.0	8.12	27.77	3.2	28.3	2.7	0.94	2.27	11.64
424.8	2.6	27.77	3.2	42.8	3.0	1.24	2.31	18.71
431.4	9.4	27.77	3.2	30.1	2.9	1.05	2.30	10.70
472.1	10.6	27.77	3.2	35.1	2.8	1.12	2.15	11.95
474.7	8.8	27.77	3.2	31.2	2.9	1.16	1.68	8.74
478.0	13.33	27.77	3.2	36.35	3	1.01	1.33	5.85

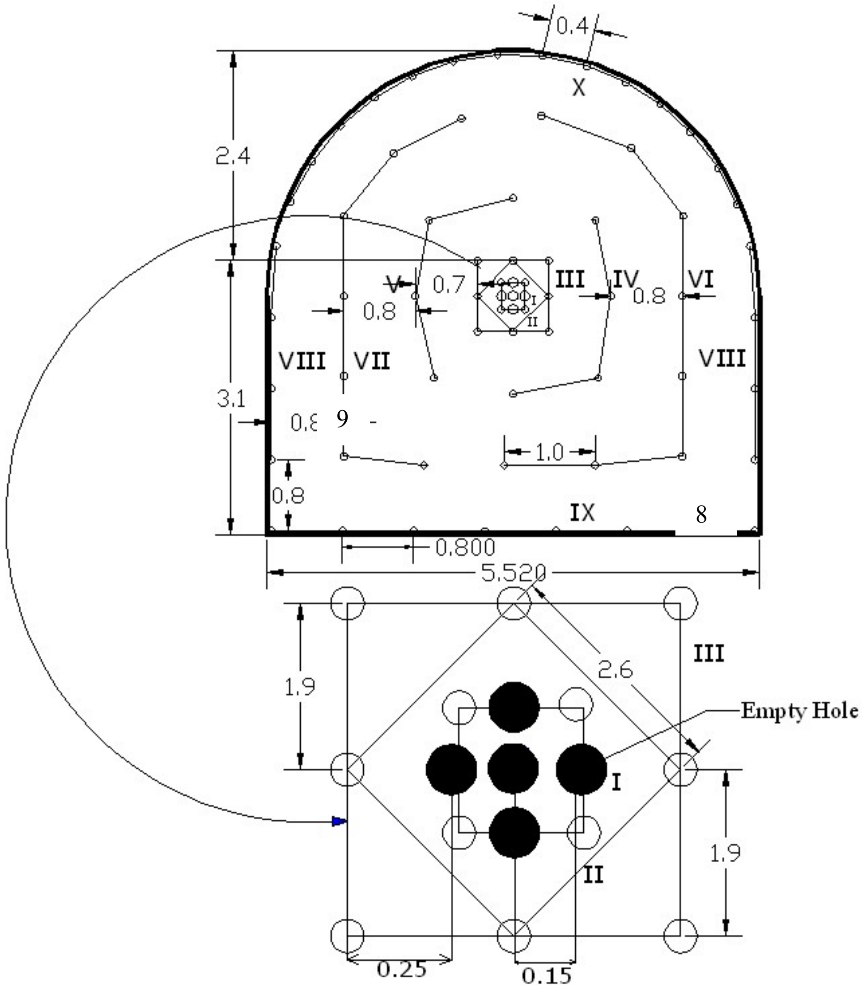

Fig.2: Blasting Pattern used in HRT Excavation

All dimensions in meter. Roman numeral denotes delay series.

3.2 Laboratory Experiments

Rock core sample were obtained by core drillingat the ten experimental blasting locations of the HRT of SBHEP project. The details of the core drilling locations and drill depth etc. have been presented in Table 3. The rock core samples have been analysed in laboratoryby computing *RQD* and *CR*and maximum core length as per IS: 6926 (1973). *RQD* and Core Recovery (*CR*), maximum core length for each meter of drill run are also given in Table 5.3.

Rock core samples were tested for variation in strength along the depth usingProceq make NR type Schmidt hammer (Fig. 3) as per suggested guidelines in ISRM (1981). The Schmidt hammer used in this study has additional feature of recording the strength values in bar chart attached with the Schmidt hammer. For a better specimen–steel base–ground interface, rock specimen was securely clamped to a steel base having an arc-shaped machined slot. Mean of 10 values measured in all the direction was taken as final value for a particular core specimen.

Fig.3: Schmidt Hammer used for Testing of Rock Core Samples

Results of *RQD*, *CR* and maximum core length, for each location, indicate extent of damage due to blasting operation on the surrounding rock mass. Variation of the strength values also corroborate extent of the damaged zone obtained using other methods of testing. A discussion on the results of geological analysis, Schmidt hammer are presented for all the ten experimental blast location.

4.0 DISCUSSION ON RESULTS

As discussed earlier, core drill at ten locations drilled at spring level gave useful information about variation in rock mass properties in the form of *RQD*, Core recovery (*CR*) and maximum core length along depth. The variation in rock mass properties in the form of *RQD*, Core recovery (*CR*) and maximum core length along depth are presented in Figs 4- 13. At RD. 22.0, effect of the blast induced damage in the initial 2.0 m of drill run is predominant. Originally, the *RQD* of the rock mass at this location was found to be 60 % (Table 3). The *RQD* and maximum core length reduced to less than 40 % and 40 cm respectively in the initial 1.0 m. The *RQD* and maximum core length increase to 58% and 52 cm respectively at 2.0 m depth. It may also be noted that the core recovery is not affected by the blast induced damage. Reduced *RQD* and maximum core length indicates that the blast induced vibration further weakens the pre-existing joints which culminate in the form of reduced *RQD* and maximum core length.

At RD 25.0 m, Variation in rock mass properties in terms *RQD*, *CR* and maximum core length with depth (Fig 5) shows impact of blast induced damage in the initial 2.0 m. Maximum core length and *RQD* for the initial one meter has been found to be less than 20 cm and 20% respectively which increases moderately to 40 cm and 59 % respectively at 2.0 m depth. Rock mass properties exhibit insignificant variation beyond 2.0 m depth.The RQD and maximum core length has been found to be less than 50% and 50 cm respectively in the initial 1.0 m at RD 350.4 m indicating impact of the blasting operation. In the remaining 3.0 m of drill run, variation in rock mass properties has been found to be comparable. It is observed at RD 353.3 m that the variation in maximum core length, *RQD* and core recovery are insignificant. *RQD* and *CR* remained equal in all the cases except at 2.0 m depth due to opening of joint along the biotite intrusion at this location. The intactness of rock core samples, marginal variation in rock mass properties indicate smaller extent blast induced damaged zone.

Figure 8 shows the variation of *RQD*, *CR* and maximum length of rock core samples obtained from RD 359.3 m. The variation in rock mass properties has been found to be insignificant except in the initial 1.0 m. The maximum core length is 22 cm in the initial 1.0 m as compared to more than 40 cm length at remaining 4.0 m of drill run. *RQD* and core length at 3.0 m and 4.0 m depth have been observed to be less than values at 2.0 m depth. Variation at RD 424.8 m (Fig 9) shows that upto initial 3.0 m depth, difference in maximum core length and *RQD* is significantly higher than the remaining 2.0 m of drill run. Influence of the blast damage is clearly delineated upto 3.0 m. Maximum core length in the initial 3.0 m drill run remained less than 40 cm as compared to more than 70 cm of maximum core length in the remaining 2.0 m drill run. The reducing gap between *RQD* and core recovery with depth indicate gradual reduction in the intensity of the blast induced damage on the surrounding rock mass.

Figure 10 shows impact of blast induced damage in the initial 2.0 m drill run at RD 431.4 m. The *RQD* and maximum core length in the initial 1.0 m drill run was found to be less than 26 cm and 53% which moderately increases to 44.6 cm and 81.5% respectively for 2.0 m of drill run. The variation in rock mass properties beyond a depth of 3.0 m has been found to be comparable. It may also be noted from the Fig 11 that gaps between *RQD*, maximum core length and core recovery is higher in the initial 2.0 m of drill run as compared to remaining 3.0 m of drill run. Impact of blasting on rock mass at RD 472.1 m is shown in Figure 12. The values of *RQD*, *CR* as well as maximum core length has been found to be low as compared to values beyond 2.0 m depth. The maximum core length and *RQD* in the initial 1.0 m drill run is 35 cm and 61.2 % respectively. The gap between *RQD* and *CR* is significantly higher for the initial 1.0 m drill run. The RQD and CR values corresponding to 2.0 m to 5.0 m of drill run is comparable.

The rock core samples, obtained at RD 478.7 were highly fractured and foliated due to instruction of muscovite and biotite minerals in the rock mass. Although, core drilling has been done upto 5.0 m depth at this location, the rock core samples corresponding to 5.0 m drill run were available in thin disks, open along the foliation. The large difference in *RQD* and core recovery reveals that the rock core samples obtained were highly fractured. Maximum core length has also been found to be significantly low for the entire length of drill run. The variations in the rock mass properties at RD 474.7 m has been found to be insignificant in the entire depth of the drill run. The limited effect of the blast damage on the rock mass properties can be seen only in the initial 1.0 m depth where the maximum core length has been found to be less than 40 cm which increase to more than 60 cm beyond 3.0 m depth.

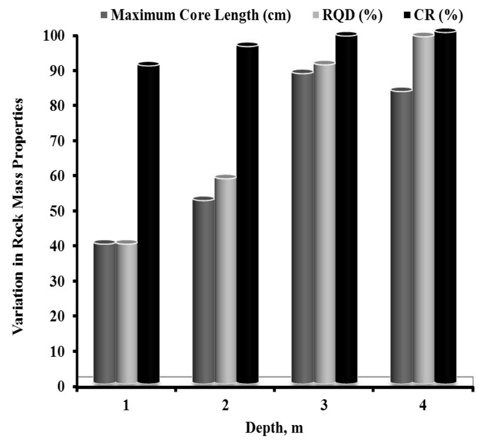

Fig 4: Variation in RQD, CR and Maximum Core Length at RD 22.0 m

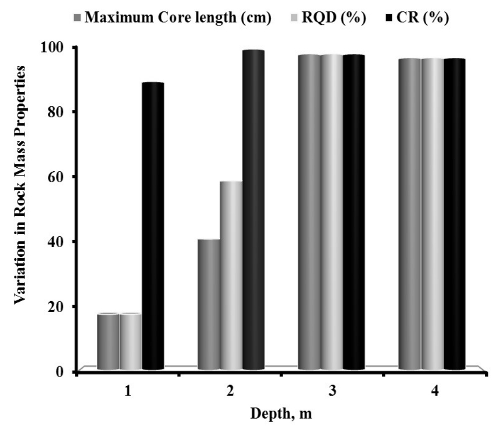

Fig 5:Variation in RQD, CR and Maximum Core Length at RD 25.0 m

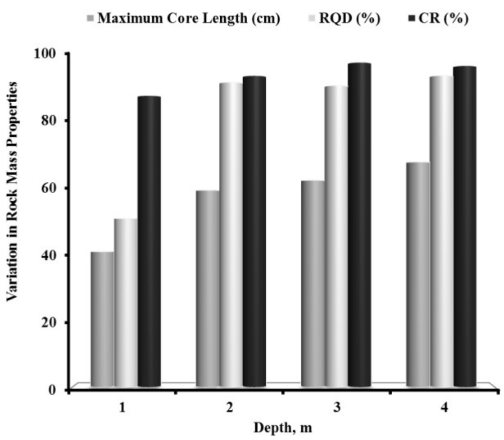

Fig 6: Variation in RQD, CR and Maximum Core Length at RD 350.4 m

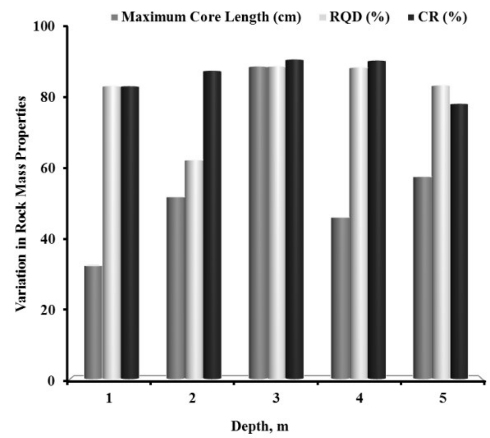

Fig 7: Variation in RQD, CR and Maximum Core Length at RD 353.3 m

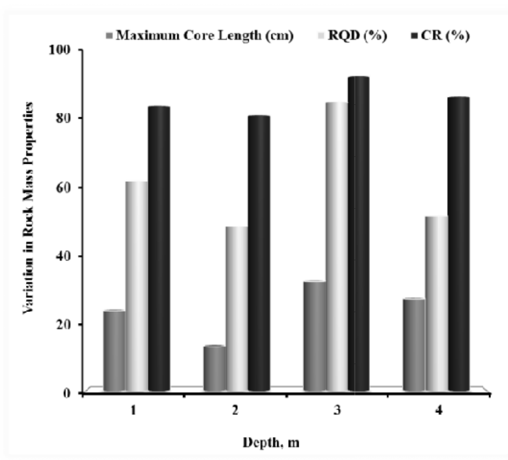

Fig 8: Variation in RQD, CR and Maximum Core Length at at RD 359.0 m

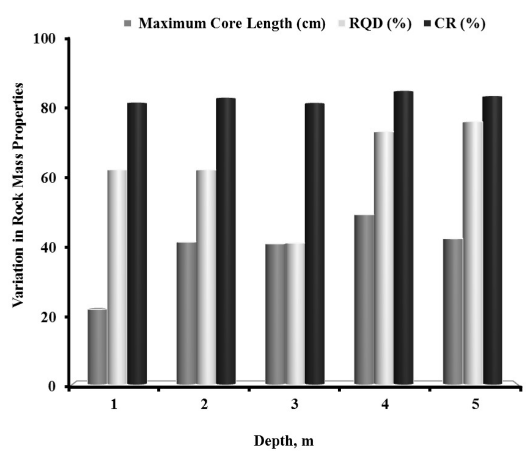

Fig 9: Variation in RQD, CR and Maximum Core Length at RD 424.8 m

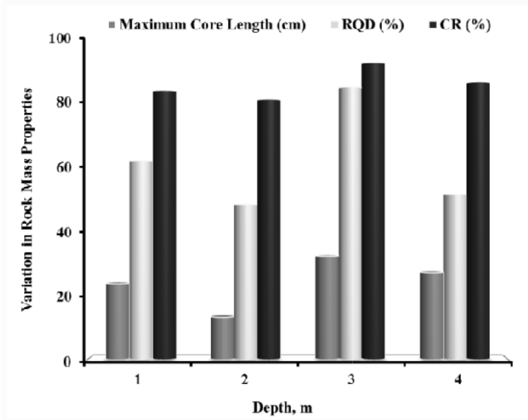

Fig 10: Variation in RQD, CR and Maximum Core Length at at RD 431.4 m

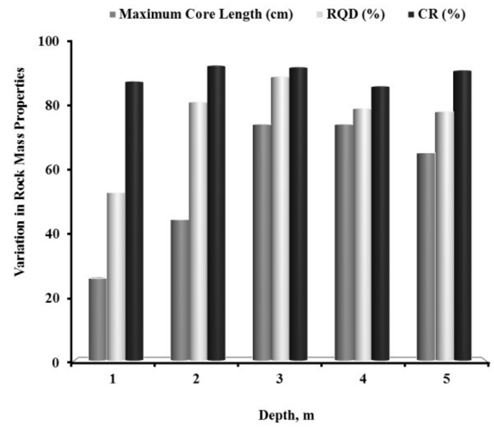

Fig 11: Variation in RQD, CR and Maximum Core Length at at RD 472.1 m

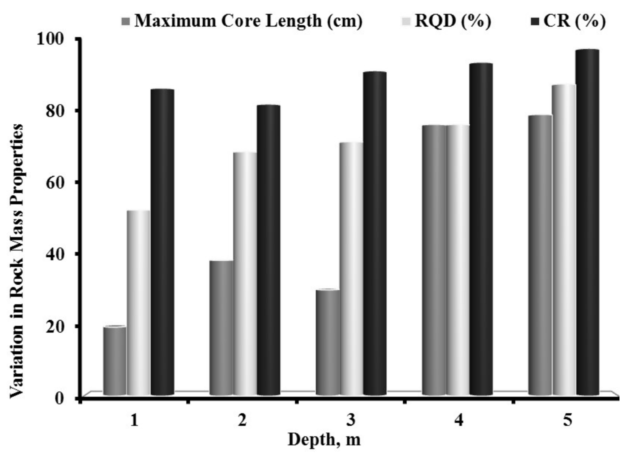

Fig 12: Variation in RQD, CR and Maximum Core Length at at RD 478.0 m

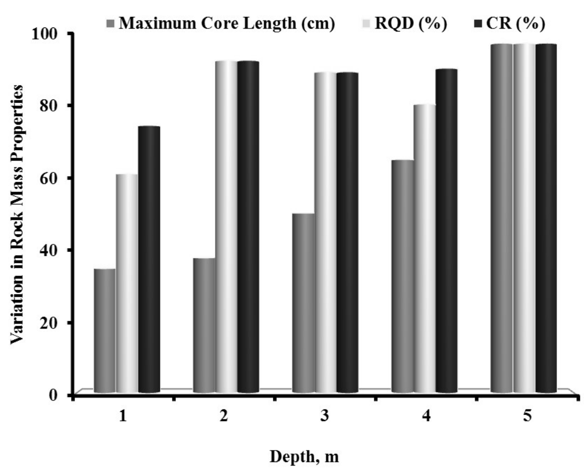

Fig 13: Variation in RQD, CR and Maximum Core Length at RD 474.7 m

Results of Schmidt hammer tests showing variation of average strength with depth are presented in Figs 14 to 23. These figures exhibit significant effect of blast damage in the initial 2.0 m at RD 22.0. The average strength was found to be below 50 MPa in the initial 2.0 m and thereafter, it attains value of approximately 60 MPa beyond a depth of 3.0 m.

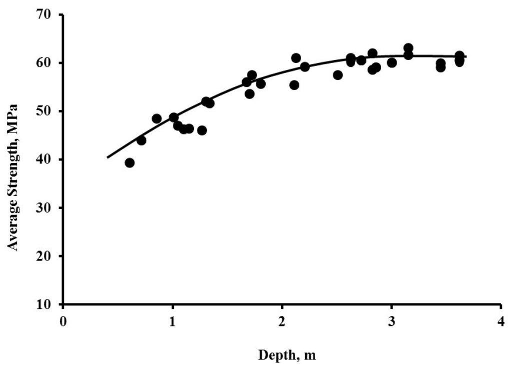

Fig 14: Variation of Strength along Depth at RD 22.0 m

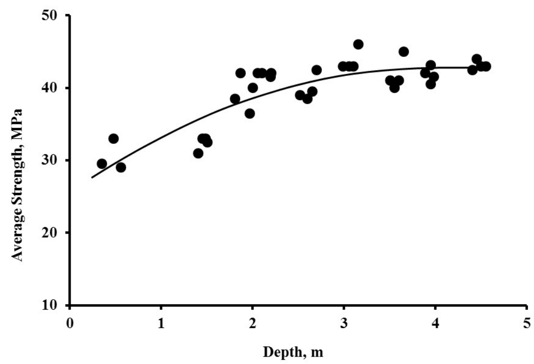

Fig 15: Variation of Strength along Depth at RD 25.0 m

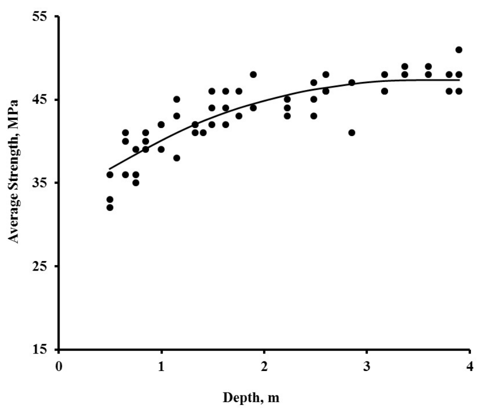

Fig 16: Variation of Strength along Depth at RD RD 350.4 m

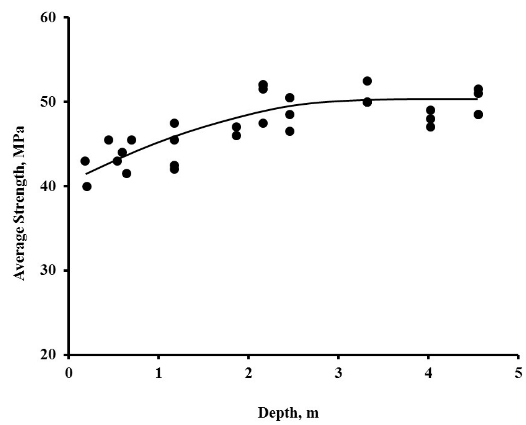

Fig 17: Variation of Strength along Depth at RD 353.3 m

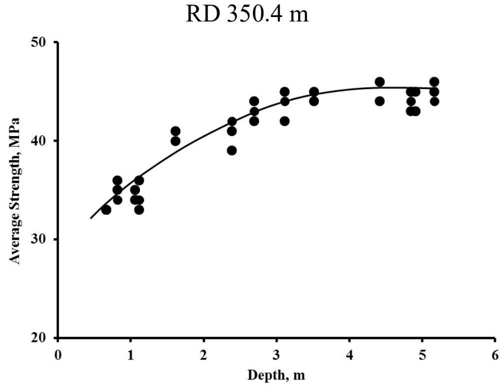

Fig 18: Variation of Strength along Depth at RD at RD 359.0 m

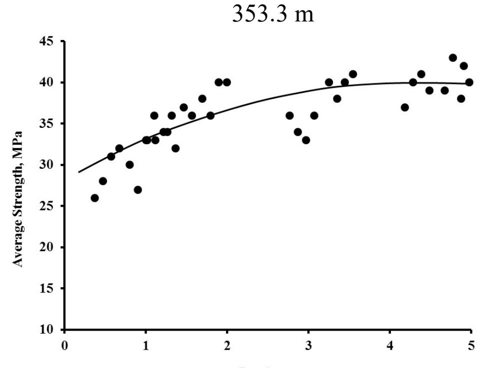

Fig 19: Variation of Strength along Depth at RD at RD 424.8 m

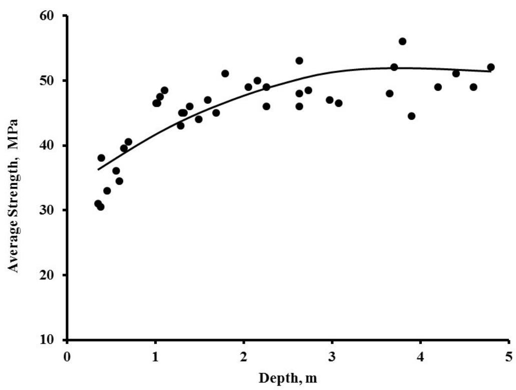

Fig 20: Variation of Strength along Depth at RD at RD 431.4 m

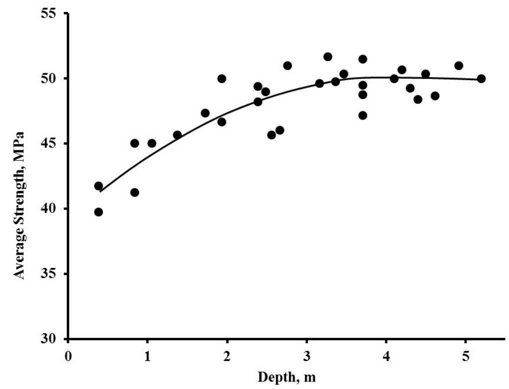

Fig 21: Variation of Strength along Depth at RD at RD 472.1 m

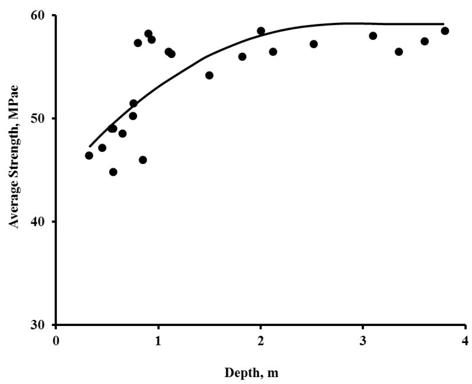

Fig 22: Variation of Strength along Depth at RD at RD 478.0 m

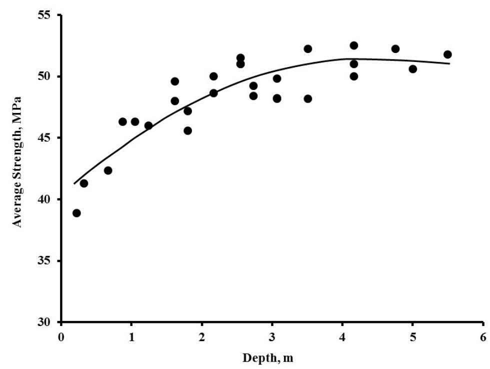

Fig 23: Variation of Strength along Depth at RD RD 474.7 m

Figure 15 reveal that the average strength of the rock remained lower than 35 MPa in the initial 2.0 m depth which increases to greater than 43 MPa beyond a depth of 3.0 m. The rate of decrease in strength in the initial 2.0 m depth is higher than the rate of change of strength beyond 3.0 m depth indicating impact of the blasting**.** The results of Schmidt hammer tests on rock core samples obtained at RD 350.4 m show that the average strength of rock samples for the initial 1.2 m is less than 40 MPa (Fig 16). The average strength of rock samples increases to greater than 45 MPa beyond 2.5 m depth. It signifies that the influence of the blasting operation is predominant upto a distance 1.2 m.

Presence of smaller extent of blast induced damaged zone at RD 353.3 is also revealed by the results of the Schmidt hammer test as shown in the Fig 17. Impact of the blasting operation has been found to be localized in the initial 1.5 m where the strength is reduced to 40 MPa compared to 50 MPa strength exhibited beyond a depth of 2.5 m. Variation in the average strength in 5.0 drill run have been found to be in the small range of 40 - 50 MPa.

It may be noted from Fig 18 that the average strength of rock mass close to the periphery of the tunnel (values corresponding to 1.0 m depth) are in the range of 30-35MPa, whereas average strength at 5.0 m depth has been found to be greater than 45 MPa. The trend line becomes asymptotic beyond 3.5 m depth at RD 359.3 m. Analysis of variation of average strength of rock core samples obtained from RD 424.8 reveals that the average strength in the initial 1.0 m depth is 30.0 m MPa. The average strength of rock samples increases to greater than 42 MPa and remained constant beyond 3.5 m depth (Fig 19). Figure 20 shows results of Schmidt hammer tests on the rock core samples obtained from RD 431.4 m. The average strength of rock core samples is approximately 30 MPa in the initial 1.0 m depth showing effect of blast induced damage. Average strength increases to greater than 45 MPa beyond 3.0 m depth. The rate ofchange strengthis higher in the initial 1.5 m depth and then it becomes moderate beyond 2.5 m depth. Asymptotic curve towards end shows insignificant impact of the blast on surrounding rock mass.

It may be observed from the analysis of results of Schmidt hammer tests that the average strength in the initial 1.5 m is 40 MPa and it increase to approximately 50 MPa beyond 3.0 depth (Fig 21). The curve becomes asymptotic beyond depth 2.0 m. Reduced average strength for initial 1.5 m shows impact of blast induced damage. As the depth increases, the intensity of the blast induced damage decreases. The average strength remained constant after a depth of 3.0 m.

At RD 478.0 m (Fig. 22), the rock core samples at depth of 1.0 m suffered maximum damage as indicated by the reduced strength (40MPa) near 1.0 m depth as compared to more than 55 MPa strength observed beyond a depth of 2.5 m. Average strength of rock samples beyond 3.0 m depth remained fairly constant at 57 MPa. It may be noted from Fig 23.0 that the average strength of rock in the initial 1.0 m is approximately 40 MPa and strength remained constant at approximately 50 MPa beyond 3.0 m depth.

5.0 CONCLUSION

In civil construction projects, blasting inevitably causes damage to the peripheral rock mass (Paventi et al., 1995a). However, the extent of the damage should be within the limit and should not pose threat to the safety and stability of the underground structures. In many construction projects, the dispute between contractor and client arises at later stage due to difference in suggested and implemented blasting practices.

Cost escalation of project due to significant overbreak and damage to rock mass requiring additional supports is major cause of dispute.The damage extends beyond overbreak. The immediate effect of damage may not be visible immediately but the problems may appear later and add to post construction maintenance cost. The service life of an underground structure will be shortened due to blast induced damage.

Results of Schmidt hammer tests on rock core samples obtained from ten different locations reveal that the strength of rock mass in the tunnel periphery is significantly reduced. Reduction in strength upto 20% as compared to the undisturbed rock mass is observed when damaged zone is greater than 2.5 m.

Smooth wall blasting technique is effective in achieving smoother tunnel profile but the extent of damaged zone is governed not only perimeter charge factor (q_p) but it is an outcome of complex interaction between maximum charge per delay and rock mass quality and other blast design parameters.Analysis of variation of the rock mass properties in terms of *RQD*, Core recovery (CR) and maximum core length revealed that the blast induced damage significantly reduces RQD leading to deterioration in rock mass quality, particularly in the immediate vicinity of the underground openings.

In general, minimum reduction in *RQD* of the rock mass in the immediate vicinity has been found to be 15% leading to approximately 15% (2 to 3 unit) reduction in *Q*-rating of the rock mass. It has been observed that the deteriorating effect of the blasting has been enhanced in places where the Joint alternation number (J_a) is higher. The higher extent of damaged zone significantly reduces RQD and damage is exhibited by the large difference between *RQD* and Core recovery. Core recovery is not affected by the blast induced damage.

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