Transaction Paper

Impact of deep-hole opencast blasting on the stability of water dams of a close-by underground coal mine

by C. Sawmliana*, R.K. Singh*, P. Pal Roy*, and P. Basu*

Synopsis

Deep-hole blasting operations in opencast mines are always associated with some annoyance to the surrounding areas in terms of ground vibration, noise, flyrock, etc. These become more severe when underground mines run parallel to them. Blasting in opencast mines may pose danger to the underground mines in terms of roof/side failures, damage of water dams, isolation/ventilation stoppings and other underground installations. The paper discusses a case study in India on the impact of deep-hole blasting in Ramagundam Opencast Project – III on the stability of various water dams of GDK-6B Incline coal mine operating in close proximity. The stability of underground water dams of GDK-6B Incline mine was endangered due to deep-hole blasting at OCP-III. There were 15 water dams in seam 3 and 16 dams in seam 4 of the underground mine. The physical strength tests using a Schmidt Hammer instrument were carried out in all the 15 water dams of No. 3 seam and 16 water dams of No. 4 seam. The test revealed that there is a decrease in compressive strength values of three dams of seam 3, namely 6A, 6B and 7. Theose three dams were also directly connected to the water source. Water seepage was also observed from the surfaces of the three dams. However, they were found to be stable in accordance with their design parameters and damage point of view while considering a threshold value of ground vibration as 25 mm/s. It was found that the value of tensile stress generated by a vibration of 25 mm/s was much lower (i.e. 0.171 N/mm^2) than the tensile strength of the two weakest concrete dams (1.85 N/mm^2). Ten experimental blasts were conducted at different working benches of the opencast mine and the ground vibration data were recorded in roof and pillars near various underground water dams. The maximum value of vibration data recorded was 5.9 mm/s in roof near dam no.1 in seam 3 with the dominant peak frequency of 22 Hz. No adverse effect on either of the dams was observed after that blast. During the period of study, no deterioration/adverse impact was found in any dam of the mine. Based on the results of the study and the analyses of the data, optimized blast design parameters and explosives weight per delay were suggested for the opencast mine to maintain the safety of the underground water dams.

Keywords: Opencast blasting, damage, water dams, underground mine, ground vibration

Introduction

Blasting with commercial explosives is the cheapest and easiest method of breaking the rocks in mines and quarries throughout the world. Mine operators attempt to break a large volume of rock using a huge amount of explosives in a round of blasts. These blasts generally include a large number of holes with greater depths. These deep-hole blasts in surface mines often pose a threat to the nearby structures, especially underground installations, when there is an underground mine operating side-by-side. The damage may be due to the higher magnitude of blast-induced ground vibrations. Roof, pillars, isolation/ventilation stoppings, water dams etc. of the underground mine may be damaged due to the disturbances created by seismic waves. This, in turn, may cause a huge loss of men and machinery. It may induce the opening of cracks in the underground strata, rendering them unstable (Singh *et al.*[1]).

In India, there are number of opencast mines in close proximity to operating underground mines. One such was Godavarikhani (GDK)-6B Incline Mine and Ramagundam Opencast (OCP)-III, Ramagundam Area of The Singareni Collieries Company Limited (SCCL), Andhra Pradesh, India.

Ramagundam OCP-III is a reconstruction project of the coal left after the erstwhile underground mines of GDK-7 and GDK-7A inclines were developed/depillared. Due to an up-throw fault of about 100 m, No. 3 and 4 seam workings of GDK-6 and GDK-6B inclines have moved almost in line with No. 1 seam of the GDK-7A incline (Figure 1). There were 15 water dams in No. 3 seam and 16 water dams in No. 4 seam. These water dams were constructed in 1992 and they were designed to withstand 120 m water head. The OCP-III working was advancing towards the dip side where these water dams were located. The distance between the blasting site in the opencast mine and the water dams varied

* *Central Institute of Mining and Fuel Research, Jharkhand, India.*

 Paper received Oct. 2008; revised paper received Jun. 2009.

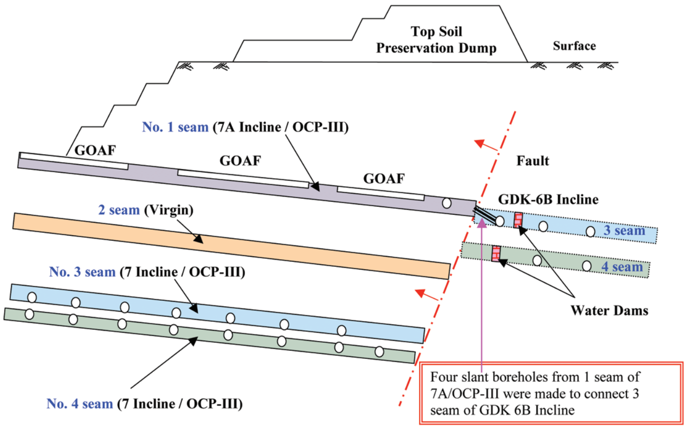

Figure 1—Layout of various workings of different mines with the location of water dams of GDK-6B incline

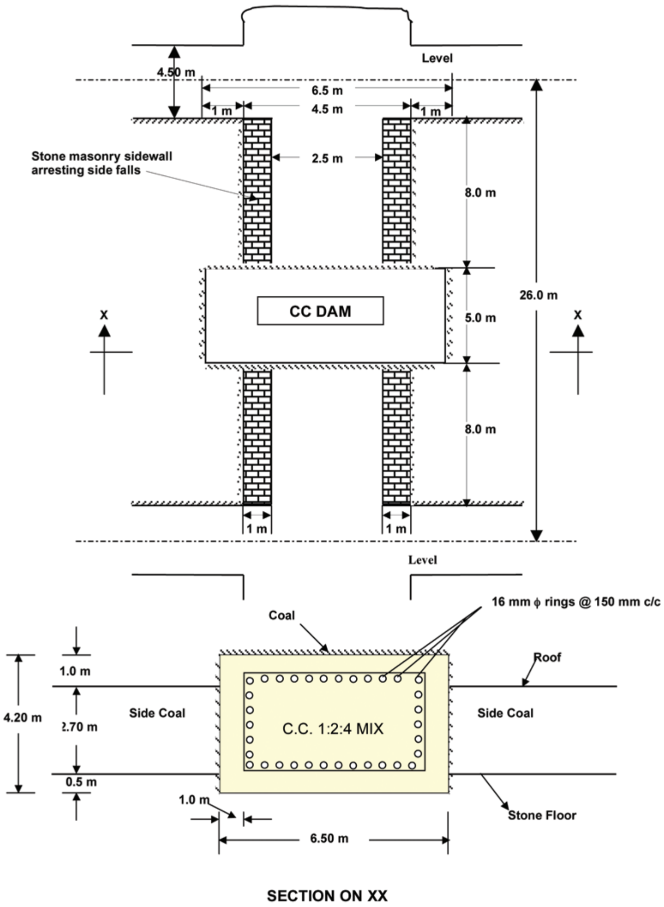

Figure 2—Construction details of water dams

between 200 and 300 m. This paper discusses the scientific study carried out at the mines to assess the damage threat to the water dams due to deep-hole blasting carried out in OCP-III.

Details of water dams and their strength tests

There were 15 dams in No. 3 seam and 16 dams in No. 4 seam of GDK-6B underground mine. During initial inspection of those dams, the presence of water inside the dams as well as the condition of the outer surface of the dams were assessed. Cracks on the outer portion of the walls/plasters were also observed. The constructional details of the water dams are shown in Figure 2. The details of the exposed dimensions of the dams are given in Tables I and II.

The compressive strengths of the retaining walls of the dams were evaluated using a Schmidt Hammer instrument. The calculated compressive strengths of different dams using rebound hardness values obtained by the Schmidt Hammer are given in Tables III and IV.

Table I

Construction-details of the dams in No. 3 seam

Dam no.	Location of the dam in No. 3 seam	Exposed dimension			Remarks
		Length (m)	Height (m)	Length of stone masonry (m)	
1	55R/15L	2.44	2.2	3.5 and 4.2	Wetting on all the front wall up to 0.65 m from bottom
2	15L/55R	2.10	2.00	6.80	No damage observed visually
3	56R/16L	2.10	3.00	5.90	No visible crack observed
4	16L/S/57R	2.65	2.60	6.90	No visible crack
5	58R/17L-S	2.60	2.23	7.65	No visible crack
6A	20L-S/63D	2.50	2.25	6.20	Water flow from the top-right side of stone masonry
6B	64R/21L	2.68	2.43	6.10	Water seepage from the top portion of the dam as well as from the side lining of the stone masonry
7	65R/21L	3.05	2.14	5.26	Water seepage from the dam
8	70R/23L	2.83	2.20	6.60	No visible crack and no sign of water inside the dam
9	23L/70R	3.30	1.80	6.75	No visible crack and no sign of water inside the dam
10	71R/24L	3.28	2.50	7.00	No visible crack and no sign of water inside the dam
11	72R/24L	3.55	2.50	7.85	No visible crack and no sign of water inside the dam
12	24L/72R	3.12	2.90	6.58	No visible crack and no sign of water inside the dam
13	73R/25L	3.10	2.00	6.90	No visible crack and no sign of water inside the dam
14	74R/25L	2.93	2.20	6.80	No visible crack and no sign of water inside the dam

Table II

Construction-details of the dams in No. 4 seam

Dam no.	Location of the dam in No. 4 seam	Exposed dimension			Remarks
		Length (m)	Height (m)	Length of stone masonry (m)	
1	6L/48D	3.60	2.10	4.60	No visible crack observed
2	41R/2LR	3.70	4.00	5.00	No visible crack observed
3	14L-S/58R	2.30	1.70	4.90	No visible damage and no sign of water inside the dam
4	15L-S/58R	3.35	2.70	5.30	No visible damage and no sign of water inside the dam
5	59R/16L-S	4.00	2.60	5.40	No visible damage and no sign of water inside the dam
6	16L-S/60D	3.05	3.35	6.50	No visible damage and no sign of water inside the dam
7	17L-S/61D	3.20	3.00	4.40	No visible damage and no sign of water inside the dam
8	62R/18L-S	2.30	2.80	5.30	No visible damage and no sign of water inside the dam
9	18L-S/62R	3.40	2.60	5.50	No visible damage and no sign of water inside the dam
10	63R/19L-S	4.00	2.90	3.90	At the centre of the dam, slight seepage was observed
11	67R/21L-S	3.40	2.80	4.60	Slight water seepage through the drainage pipe
12	21L-S/67R	3.80	2.90	4.40	No visible damage and no sign of water inside the dam
13	68R/22L	3.70	3.00	6.70	Small seepage of water through the drainage pipe
14	69R/225L-S	3.60	2.70	5.05	No visible damage and no sign of water inside the dam
15	22L-S/69R	3.40	2.50	5.85	No visible damage and no sign of water inside the dam
16	70R/23L	2.70	3.00	5.40	No visible damage and no sign of water inside the dam

Rebound hardness values for stone masonry, used for supporting the concrete dam, were also measured in some of the dams located in No. 3 and 4 seams. The results of the rebound hardness values as well as their corresponding compressive strengths are given in Table V.

The site inspection and results of the rebound hardness values of various dams indicated that the dam nos. 6A, 6B and 7 in No. 3 seam were most sensitive as they were directly connected to the water pressure through four boreholes. The hardness values in those three dams were also lower than that of other dams located in No. 3 and 4 seams.

Assessment of the stability of the dams

At the macro-level, any dam of reinforced cement concrete (RCC) consists of coarse aggregate particles surrounded by a mortar matrix. However, a mortar itself consists of sand grains and hydrated cement paste where the paste is virtually or never fully hydrated so that the products of hydration are intermingled with remnants of hydrated cements. The products of hydration consist of particles of different orders of magnitude, notably gel and crystalline particles such as calcium hydroxide. The large surface volume means that

Table III

Rebound hardness values and their corresponding compressive strengths for different dams in No. 3 seam

Dam no.	Location of the dam	Rebound hardness values	Average hardness values	Corresponding compressive strength (N/mm^2)
1	55R/15L	25, 35, 31, 41, 40, 27, 31, 24	31.75	22.21
2	15L/55R	29, 31, 30, 32, 29, 35, 31, 30	30.87	20.63
3	56R/16L	29, 29, 37, 24, 29, 36, 22, 24	28.75	16.81
4	16L/S/57R	32, 22, 45, 12, 40, 35, 40, 27	33.37	25.13
5	58R/17L-S	48, 49, 47, 24, 36, 42, 34, 32	39.00	35.28
6A	20L-S/63D	12, 22, 25, 18, 18, 20, 20, 26	20.12	<10.00
6B	64R/21L	30, 25, 24, 22, 32, 28, 29, 24	26.75	13.21
7	65R/21L	24, 11, 19, 27, 18, 14, 32,14	18.62	<10.00
8	70R/23L	30, 27, 37, 40, 29, 41, 28, 42	34.25	26.72
9	-	39, 34, 27, 28, 42, 33, 26, 23	31.50	21.77
10	71R/24L	32, 26, 46, 35, 31, 40, 42, 28	35.00	28.07
11	72R/24L	30, 33, 32, 32, 32, 39, 30, 36	33.00	24.47a
12	24L/72R	27, 28, 33, 23, 25, 22, 23, 24	25.62	10.06
13	73R/25L	39, 35, 40, 39, 33, 34, 38, 40	37.25	32.12
14	74R/25L	46, 49, 38, 51, 46, 47, 47, 42	45.75	47.43

Table IV

Rebound hardness values and their corresponding compressive strengths for different dams in No. 4 seam

Dam no.	Location of the dam	Rebound hardness values	Average hardness value	Corresponding compressive strength (N/mm^2)
1	6L/48D	41, 33, 22, 25, 28, 34, 24, 31, 35, 39	31.20	21.23
2	41R/7L	24, 42, 29, 35, 29, 28, 19, 32, 16, 16	27.00	13.66
3	14L-S/58R	39, 34, 36, 37, 33, 37, 38, 37, 30, 47	36.80	31.31
4	15L-S/58R	38, 48, 37, 41, 42, 37, 23, 37, 30, 30	36.50	30.77
5	59R/16L-S	32, 21, 30, 26, 24, 24, 38, 39, 36, 32	30.20	19.43
6	16L-S/60D	36, 38, 44, 46, 35, 35, 46, 21, 38, 45	38.40	34.20
7	17L-S/61D	37, 32, 41, 38, 33, 28, 26, 19, 24, 25	30.30	19.61
8	62R/18L-S	35, 40, 37, 19, 22, 28, 11, 12, 19, 15	23.80	14.56
9	18L-S/62R	25, 30, 31, 22, 26, 35, 29, 32, 30, 26	32.90	24.29
10	63R/19L-S	32, 33, 25, 25, 25, 25, 26, 35, 22, 33	28.10	15.64
11	67R/21L-S	21, 43, 37, 21, 38, 39, 32, 46, 22, 31	33.00	24.47
12	21L-S/67R	17, 18, 17, 33, 32, 19, 30, 31, 18, 38	25.30	10.60
13	68R/22L	40, 39, 37, 42, 32, 32, 48, 29, 43, 41	38.30	33.97
14	69R/22L-S	26, 29, 45, 40, 42, 47, 38, 39, 43, 29	37.80	33.11
15	22L-S/69R	41, 36, 42, 45, 43, 40, 33, 42, 45, 43	41.00	38.88
16	70R/23L	30, 37, 29, 35, 24, 24, 25, 36, 30, 33	30.30	19.60

Table V

Rebound hardness values and their corresponding compressive strengths for stone masonry walls of some of the dams

Dam no.	Location of the dam	Rebound hardness values		Average hardness value	Corresponding compressive strength (N/mm^2)
1	55R/15L	N-side	30,42,47,41,41,30,31,36,38,40	34.50	27.17
	3-seam	S-side	30,42,40,53,40,43,38,50,55,35	42.60	41.76
2	15L/55R	R-side	52,50,55,49,42,49,35,43,49,42	46.60	48.96
	3-seam	D-side	38,50,38,36,45,40,42,41,40,34	41.30	39.42
6A	20LS/63D	R-side*	08,14,19,20,20,21,23,17,23,20	18.60	<10.00
	3-seam	D-side*	25,37,24,28,25,31,34,28,26,21	27.90	15.25
6B	64R/21L	N-side*	19,17,24,23,29	22.40	<10.00
	3-seam	S-side*	32,27,17,35,38,37,12,20	27.25	14.11
7	65R/21L	N-side*	29,33,24,21,17,29,21,18,39,25	25.60	11.14
	3-seam	S-side*	9,7,8,22,8,16,13,39,24,22,27	19.50	<10.00
1	6L/48D	R-side	39,47,46,56,31,45	44.00	44.28
	4-seam	D-side	49,39,43,41,46,48	44.33	44.88
2	41R/7L	N-side	43,49,40,48,48,45	45.50	46.98
	4-seam	S-side	54,59,30,44,45,43	45.83	47.58

*Measurements were taken on cement plaster surface forces are large compared to gravity and other forces. The bonds between the particles are relatively weak and physical in nature and are referred to as van der-Waals forces. That is why the compressive strength produced by the structure is large compared to the tensile strength.

The maximum tensile stress reached in the concrete is referred to as the modulus of rupture. This stress overestimates the tensile strength of concrete for three reasons: (1) the calculation of rupture is based on a linear stress distribution system, (2) crack propagation from the extreme fibre is blocked by the less stressed adjacent fibres so that a higher stress may be reached prior to the collapse of the test specimen than would occur in axial tension, and (3) only extreme fibre is subjected to the maximum stress so that the modulus of rupture is on an average 50% higher than the direct tensile strength produced by the RCC dam.

Tensile strength produced by the concrete at the middle of the dam to withstand vibration (as per ASTM C496-71 and IS 516, 5816 standards) is:

$$\text{Tensile strength} = f_{cr} = k\sqrt{f_{ck}}\left(N/mm^2\right) \qquad [1]$$

where,

f_{cr} = tensile strength of concrete (N/mm2)
f_{ck} = characteristic compressive strength of concrete at the centre of the dam (N/mm2)
k = constant.

The European Concrete Committee recommended the value of 'k' for reinforced concrete dam as 0.7.

Schmidt Hammer test results for all the dams showed that the compressive strengths of dam Nos. 6A and 7 in No. 3 seam (at the exposed cement concrete wall) were approximately 7 N/mm2. However, the compressive strengths of other dams varied between 10.60 and 47.43 N/mm2. So, for better safety, the lowest value of compressive strength (i.e. 7 N/mm2) was considered for the determination of tensile strength in order to assess the stability of all the dams.

Using Equation [1], the minimum tensile strength of the underground water dam was calculated as:

$$f_{cr} = 0.7\sqrt{7} = 1.85\ N/mm^2 \qquad [2]$$

The ultimate shearing strength of rock or the plug material can be determined by the relationship between tensile strength and compressive strength as:

$$\sigma_s = 0.5(\sigma_c \times \sigma_t)^{1/2} \qquad [3]$$

where,

σ_s = shear strength of rock or plug material
σ_c = compressive strength of concrete and
σ_t = tensile strength of concrete.

For coal pillars, the safe permissible shear strength can be estimated by taking a factor of safety of 1.5 on the ultimate strength determined in the laboratory. According to the weakest link theory, the strength of coal pillars is about 7.5 times less than that of the laboratory strength. However, in all the underground dams under study, two sides of the plug were encased in sandstone (one in the roof portion and the other in the floor portion of the gallery). The other two sides of the dams were encased in the coal pillar. As such, the reduction factor was taken as nearly the half of 7.5 i.e. four times less than that of the laboratory strength. There could be further reduction in strength in due course. Considering the weakest link as being in the coal portion, the safe permissible shear strength of coal can be calculated as:

$$\sigma_s = 0.015 \times (\sigma_c \times \sigma_t)^{1/2} = 31.681\ t/m^2 \qquad [4]$$

In order to check the stability of all the dams, the resistance to sliding of the plug against the normal water pressure from the dimension of all the dams is calculated. The maximum water pressure trying to dislodge the dam can be determined as:

$$P_{water} = L \times W \times WH \qquad [5]$$

where,

P_{water} = maximum water pressure acting on the dam
L = length of the dam exposed to the water
WH = maximum water head acting on the dam (120 m)

The resistance area along the periphery of the dam is:

$$R_{periphery} = (6.5 + 6.5 + 5.0 + 5.0) \times 4.2 = 95.34\ m^2$$

Hence, shearing force acting on the dam to dislodge it can be obtained from:

$$P_{shear} = P_{water}/R_{periphery}\left(t/m^2\right) \qquad [6]$$

From the above Equations [4], [5] and [6], the stability of all the dams can be checked on the basis of their shearing forces. The calculated shearing forces of different dams in No. 3 and 4 seams are given in Table VI.

From Table VI, it is clear that the shearing forces varied between 5.29 and 18.63 t/m2. It was maximum (i.e.18.63 t/m2) in the case of Dam No. 2 in 4 seam. All the calculated values of shearing force were less than the shearing strength at the weakest link in coal portion (i.e. 31.68 t/m2). Therefore, as per the design, all the dams in No. 3 and 4 seams were found to be stable in respect of their resistances in sliding against the normal water pressure acting on them. The shearing forces calculated for dam nos. 6A, 6B and 7 in No. 3 seam area were 7.08 t/m2, 8.20 t/m2 and 8.22 t/m2 respectively.

Influence of geological features on wave propagations

The transfer of seismic energy from an open pit blasting to a nearby underground working is accomplished predominantly by primary waves (Tunstall[2]). As the seismic waves move through the crown pillar from an open pit, they are attenuated according to the nature of rock in which they are travelling. The rate of seismic wave attenuation is lower in hard and massive rock formation than in highly jointed rock and softer formations. In a jointed rock formation, the waves will be attenuated to a greater degree as a result of frictional loses at joint surfaces, and less vibration will reach the underground excavation.

In the area studied, the parting between the opencast workings and underground water dams consisted predominantly of sandstone beds, as given in Figure 3. These sandstone beds were mainly medium grained and massive formation, which predominantly supported the higher

Table VI

Calculated values of shearing force (P_{shear}) acting on the dams

No. 3 seam			No. 4 seam		
Dam no.	Location	P_{shear} (t/m^3)	Dam no.	Location	P_{shear} (t/m^3)
1	55R/15L	6.76	1	6L/48D	9.52
2	15L/55R	5.29	2	41R/2L	18.63
3	56R/16L	7.93	3	14L-S/58R	4.92
4	16L/S/57R	8.67	4	15L-S/58R	11.38
5	58R/17L-S	7.30	5	59R/16L-S	13.09
6A	20L-S/63D	7.08	6	16L-S/60D	12.86
6B	64R/21L	8.20	7	17L-S/61D	12.08
7	65R/21L	8.22	8	62R/18L-S	8.11
8	70R/23L	7.84	9	18L-S/62R	11.13
9	23L/70R	7.48	10	63R/19L-S	14.60
10	71R/24L	10.32	11	67R/21L-S	11.98
11	72R/24L	11.17	12	21L-S/67R	13.87
12	24L/72R	11.39	13	68R/22L	13.97
13	73R/25L	7.80	14	69R/22L-S	12.23
14	74R/25L	8.11	15	22L-S/69R	10.70
			16	70R/23L	10.20

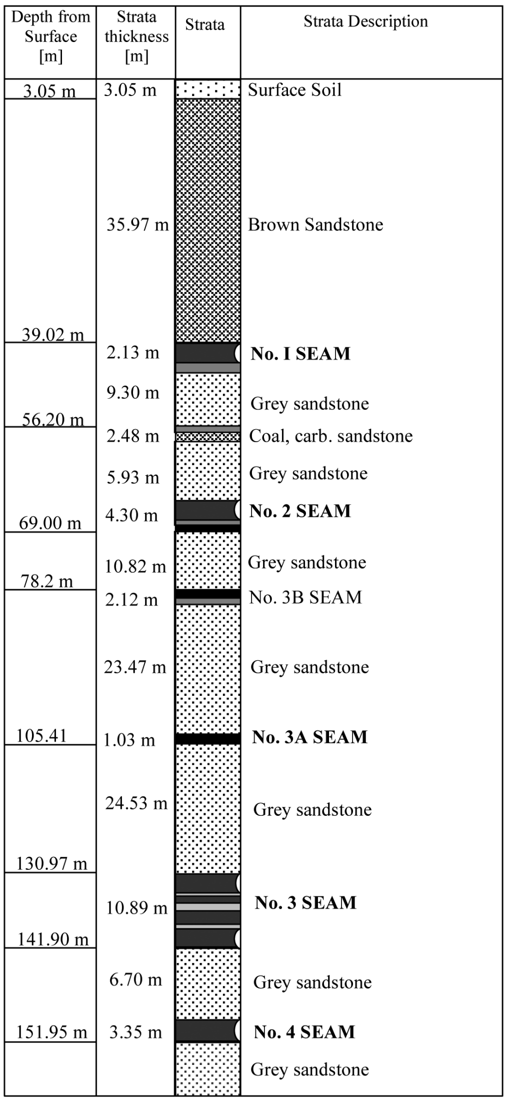

Figure 3—Borehole section (BH No. 79) showing different strata at GDK-6B Incline

transfer of the propagating wave energy from opencast blasting to the underground water dams. However, the presence of a decoaled area (goaf area) in No. 1 seam between the opencast working and underground workings, attenuated the seismic waves whenever a blasting operation was conducted above those seams. The presence of an up-throw fault (100 m) between the opencast working and underground water dams would also affect the attenuation of seismic waves from the opencast blasting. Therefore, lower values of frequency waves (5–10 Hz) were recorded in some of the blasts irrespective of their distances from the monitoring points.

Threshold value of vibration for safety of dams

The permissible levels of vibration for different surface structures at different frequency levels have been suggested by the mine regulatory agency in India. But, for underground structures especially water bodies, no threshold value or limit has been suggested till date. Various researchers throughout the world have suggested different levels of vibration in terms of peak particle velocity (PPV) in relation to the damage to the underground workings (underground roof and pillars) due to blasting in close-by open-pit mines. Rupert and Clark[3] concluded in their study that only minor damage in the form of localized thin spalls and collapse of previously fractured coal ribs resulted from blasts having an associated PPV of more than 50 mm/s. Jensen *et al.*[4] reported no roof failure even at vibration levels of 445 mm/s in roof and detachment of few loose stones only at 127 mm/s. Kidybinski[5] reported that damage to underground coal mine openings in the form of small roof falls or floor heave may occur when the PPV lies in the range of 50–100 mm/s and large roof falls at 100-200 mm/s. Fadeev *et al.*[6] suggested a PPV of 120 mm/s for one fold blasting in the case of primary mine openings (service life up to 10 years) namely pit bottom, main cross entries and drifts, and 60 mm/s for repeated blasting. For secondary mine openings (service life up to 3 years), namely haulage breakthrough and drifts, the allowable values of PPV were 240 mm/s for repeated blasting and 480 mm/s for one fold blasting. Fourie and Green[7] concluded in their studies that the PPV of 110 mm/s produced only minor damage, and serious extensive damage resulted when PPV was as much as 390 mm/s.

Tunstall[2] suggested that a PPV of 175 mm/s would not contribute any damage to underground opening where very good quality rocks (RMR = 85) were encountered. On the other hand, the poor quality rock (RMR = 49), which had been loosened by previous open-pit blast vibrations, sustained minor visible damage at a PPV of 46 mm/s and major damage at 379 mm/s. Lewandowski *et al.*[8] set a conservative criteria of targeted maximum PPV of 50 mm/s for the safety of coal underground headings. They further clarified that this conservative value of PPV was decided after investigations indicated a possible limit of 250 mm/s. Based on the R&D contributions of CIMFR, the Directorate General of Mines Safety (DGMS) in their Technical Circular No. 06 of 2007 has stipulated the limiting values of peak particle velocity (mm/s) for different RMR (rock mass rating) of roof rock for the safety of underground coal mine workings (Pal Roy[9]). The PPV values range between 50 and 120 mm/s (measurement made in roof) for different RMR values varying between 20 and 80.

Assessment of stability of dams considering vibrations from opencast blasting

Siskind[10] derived a relationship between peak particle velocity (PPV, in/s) and tensile strength of a concrete (lb/in^2) that can withstand the level of ground vibration. The relationship is a unit-adjusted equation for peak particle velocity (PPV) and tensile strength of concrete, by taking into consideration the specific gravity of concrete and its P-wave velocity as:

$$PPV = S / \left(9.25 \times 10^{-5} \times \sigma \times C_p\right) \qquad [7]$$

where,

S = tensile strength of the concrete (in/lb^2)
σ = specific gravity of concrete
C_p = P-wave propagation velocity (in/s)
PPV = peak particle velocity (in/s)

Considering the safe value of vibration as 25 mm/s (0.984 in/s), the primary wave velocity (P-wave) of the concrete dam as 3 500 m/s (137 795.26 in/s) and its specific gravity as 2.0, the induced tensile stress can be determined as:

$$\begin{aligned} S &= PPV \times (9.25 \times 10^{-5} \times \sigma \times C_p) \\ &= 0.984 \times (9.25 \times 10^{-5} \times 2.0 \times 137795.26) \\ &= 25.084 \text{ lb/in}^2 \\ &= 0.171 \text{ N/mm}^2. \end{aligned}$$

It is clear that the value of tensile stress generated by peak particle velocity of 25 mm/s is much lower than the observed tensile strength of the two weakest concrete dams i.e. 1.85 N/mm^2 (for dam nos. 6A and 6B in No. 3 seam). Based on equation [7], the critical value of peak particle velocity for the dam having a tensile strength of 1.85 N/mm^2 is 267.97 mm/s.

Water head acting on the dams could also affect their stabilities. The water head coupled with a higher magnitude of ground vibrations from opencast blasting could destabilize the dams. The maximum water head recorded during peak rainy season in all the dams of the mine was 24 m only (i.e. in dam no. 6A of Seam 3). But the dams were designed to withstand a maximum water head of 120 m. The recorded peak water head in the rainy season was much less compared to the designed water head of the dam. Therefore, considering the age of the dams, level of water heads acting continuously on the dams, sensitivity of the dams, as well as their water seepage problems, a peak particle velocity of 25 mm/s was considered as the safe vibration limit for the water dams.

Experimental work

Three-directional transducers of seismographs (frequency range: 2–300 Hz; seismic range: 0.25–254 mm/s) were used to monitor the blast-induced ground vibrations in Seam 3. Some of the dams of Seam 3 were directly connected to the water head. They were nearer to the opencast blasting locations also in comparison to the dams of Seam 4. Hence, it was decided to monitor the ground vibrations near the water-connected dams of seam 3. Special arrangements using non-ferric attachments of sensors were made to mount the transducers of seismographs in the roof near the underground water dams. Monitoring points were fixed in the roof of the galleries at a distance of 1–1.5 m from the concrete dams. Due to presence of stone masonry walls at the pillar sides, it was difficult to make monitoring points at the pillar side, nearby the dam. Notches were made in the coal pillars near the stone masonry walls. The points in the pillars were 1.0–1.2 m below the roof having a depth of 0.5–0.6 m inside the pillar.

Ten blasts were performed at different working benches of the opencast mine with the existing and modified designs suggested by CIMFR. The system of mining in the opencast mine was drilling and blasting for dragline as well as shovel and dumper combinations. The diameter of the holes were 250 mm in both shovel as well as dragline benches. In shovel bench blasting, the depth of holes varied between 6.1 and 21.0 m, whereas in dragline blasting, hole depth varied between 22.8 and 26.0 m. The average burden and spacing in dragline blasting were 8 m x 10 m, while in the case of shovel bench blasting, it was 6 m x 8 m. The initiation system included the conventional detonating cord (D-cord) in nine blasts and Nonel devices in one blast. The total explosive weight in a blasting round varied between 3 206 and 21 942 kg in the case of shovel bench blasting and 54 606 and 66 932 kg in dragline bench blasting. The maximum explosive weight per delay varied between 214 and 1 380 kg in shovel bench blasting whereas it varied between 828 and 1 812 kg in dragline bench blasting.

Results and discussion

Blast-induced ground vibrations were recorded in terms of peak particle velocity (PPV) in mm/s. Depending upon the distance of monitoring points near the particular water dam in the underground mine from the blasting face in the Opencast mine and the amount of explosives detonated in the blasting round, the vibration data varied from 0.50 mm/s to 5.9 mm/s. The maximum magnitude of vibration recorded during shovel bench blasting was 2.70 mm/s in the roof near dam no. 6A, Seam 3 with a dominant peak frequency of

21 Hz. The corresponding vibration in the pillar was 2.10 mm/s with associated frequency of 20 Hz. The blast wave signatures (at 2 048 samples per second) recorded in roof and pillar are shown in Figures 4 and 5. The blast was performed at 727 m away from the above mentioned dam. The total amount of explosives detonated in the blasting round was 9 020 kg whereas the maximum explosive charge detonated in a delay was 675 kg. No detachment of any loosened chip was observed at this level of vibration.

In dragline bench blasting, the maximum magnitude of vibration recorded was 5.9 mm/s near dam no. 1 in Seam 3 with the dominant peak frequency of 22 Hz. The distance from the blasting site to the monitoring point was 781 m. The total amount of explosive and maximum explosive charges fired in a delay were 66 932 kg and 1 812 kg respectively. This level of vibration did not cause any adverse impact on the dam in the form of detachment of any loosened chips either from the roof near the dam or from the dam structure itself. Summarized blast details and recorded vibration data at different dams are given in Table VII.

Analyses of vibration data

Regression analyses of vibration data collected near various underground water dams were carried out to derive a propagation equation for computing the explosive weight per delay to be fired in the opencast mine considering the safety and stability of the water dams of the underground mine. The established empirical equation has the correlation between maximum explosive weight per delay (Q_{max}, in kg), radial distance between the seismograph transducer in the underground mine and the blasting patch in the opencast mine (*R*, in m) and recorded peak particle velocity near the water dams (PPV, in mm/s). The propagation equation derived at 95% confidence level is as follows:

$$PPV = 87103 \times \left[R / \sqrt{Q_{max}}\right]^{-2.89} \qquad [8]$$
$$Correlation\ coefficient = 0.706$$

The regression plot of vibration data is shown in Figure 6.

Recommendation of safe explosive charges for the safety of the dam

The maximum amount of explosive weight to be detonated in a delay for opencast blasting at OCP-III considering the safety of underground water dams has been calculated based on the regression Equation [8]. The recommended charges per delay to contain ground vibration within the safe and permissible limit (25 mm/s) are given in Figure 7.

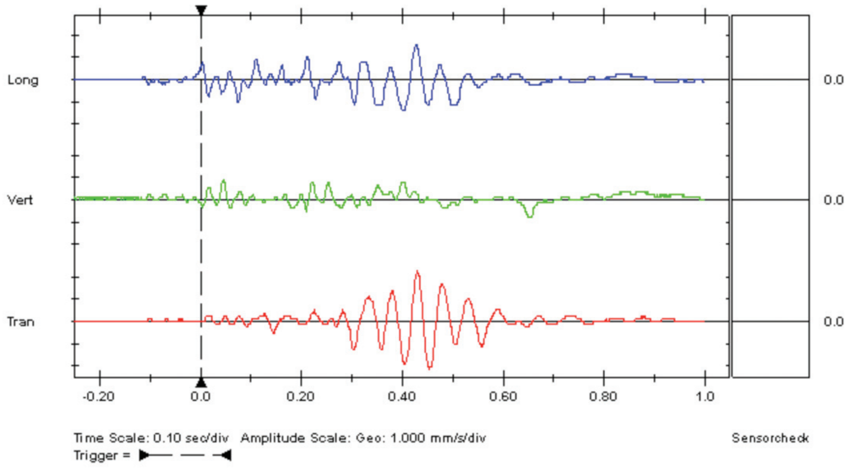

Figure 4—Blast wave signature recorded in roof near dam 6A

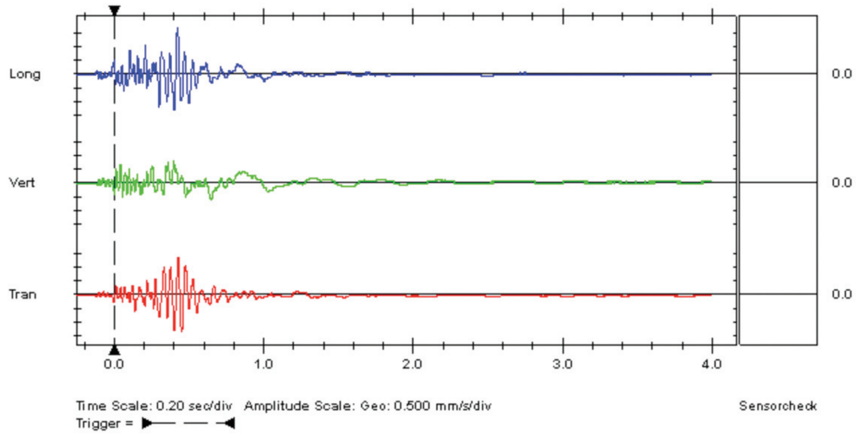

Figure 5—Blast wave signature recorded in pillar near dam 6A

Impact of deep-hole opencast blasting on the stability of water dams

Table VII

Details of blast design parameters in OCP-III and vibrations recorded near various underground dams

Sl. no.	Location of blast	No. of holes	Avg. hole depth (m)	Total explosive in a blast (kg)	Explosive in a delay (kg)	Vibration monitored				Explosive and initiation system	Remarks
						Location	Distance (m)	PPV (mm/s)	Freq. (Hz)		
1	Shovel bench (DM-15)	11	21.0	6 513	1 380	Dam 6A-roof Dam 6A-pillar Dam 6B-roof Dam 6B-pillar	1461 1461 1479 1479	<0.50 <0.50 <0.50 <0.50	- - - -	SMS, DF with cord relay	No detachment of any loose chips
2	Shovel bench (DM-2)	46 + 28*	11.5 & 7.0	9 020	675	Dam 6A-roof Dam 6A-pillar Dam 6B-roof	727 727 739	2.70 2.10 2.24	21 20 20	SMS, DF with cord relay	No detachment of any loose chips
3	Shovel bench [DM-6]	48 + 30*	11.8	11 826	430	Dam 6A-roof Dam 6A-pillar Dam 6B-roof Dam 6B-pillar	1018 1018 1024 1024	<0.50 <0.50 <0.50 <0.50	- - - -	SMS, Nonel with TLD	No detachment of any loose chips
4	Shovel bench [DM-8]	25 + 13*	13.5	6 012	214	Dam 6A-roof Dam 6A-pillar Dam 6B-roof Dam 6B-pillar	829 829 839 839	1.54 1.30 1.40 1.27	21 20 22 23	SMS, DF with cord relay	No detachment of any loose chips
5	Shovel bench [DM-9]	75	6.10	3 206	392	Dam 6A-roof Dam 6A-pillar Dam 6B-roof Dam 6B-pillar	882 882 884 884	<0.50 <0.50 <0.50 <0.50	- - - -	SMS, DF with cord relay	No detachment of any loose chips
6	Shovel bench [DM-14]	40	21.0	21 942	1 097	Dam 1-roof Dam 6A-pillar Dam 7-roof Dam 7-pillar	1310 1537 1574 1574	1.22 <0.50 0.524 <0.50	12 - 9 -	SMS, DF with cord relay	No detachment of any loose chips
7	Shovel bench [DM-10]	86	11.0	9 532	440	Dam 1-roof Dam 6A-pillar Dam 7-roof Dam 7-pillar	512 732 767 767	1.57 0.587 0.619 0.500	17 18 15 19	SMS, DF with cord relay	No detachment of any loose chips
8	Dragline bench	80 + 48*	22.8	66 932	1 812	Dam 1-roof	781	5.90	22	SMS, DF with cord relay	No detachment of any loose chips
9	Dragline bench	63 + 40*	25.8	54 606	828	Dam 1-roof	801	5.46	16	SMS, DF with cord relay	No detachment of any loose chips
10	Dragline bench	60 + 42*	26.0	61 219	985	Dam 1-pillar Dam 6A-roof Dam 6B-pillar	813 1001 1017	3.49 1.77 1.62	23 16 9	SMS, DF with cord relay	No detachment of any loose chips

* Satellite hole; SMS—site-mixed slurry; DF—detonating fuse; TLD—trunkline delay

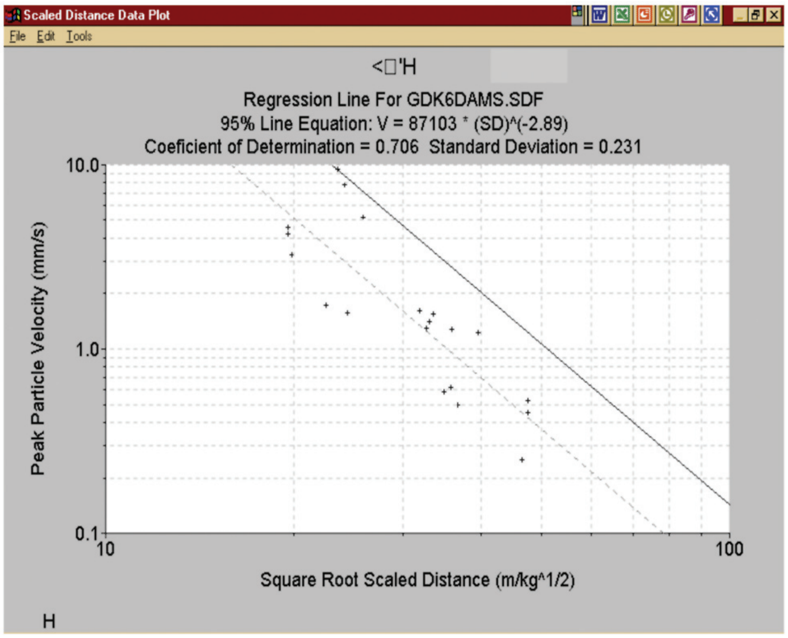

Figure 6—Regression plot of vibration data recorded near various water dams

Impact of deep-hole opencast blasting on the stability of water dams

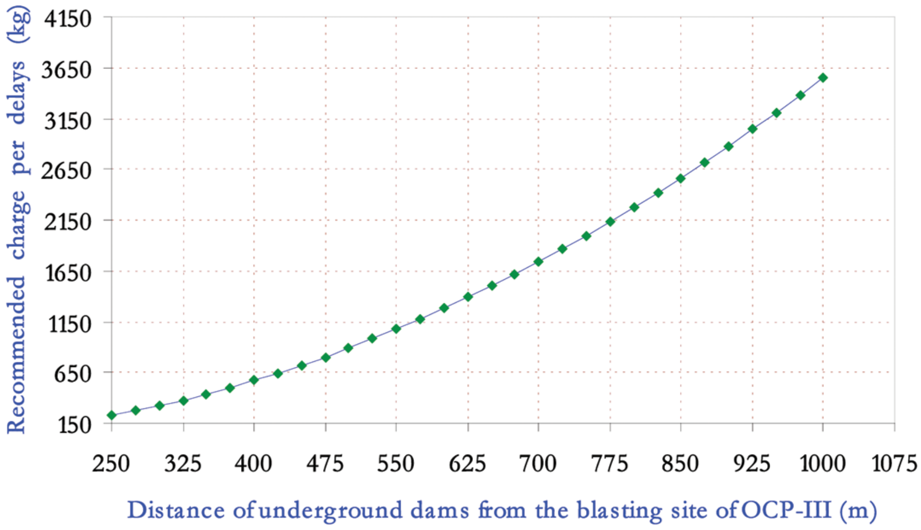

Figure 7—Recommended charge per delay to be fired in a round of blasting at OCP-III to contain vibration within safe limits for the safety of water dams

Conclusions

The physical inspection of all the dams and their strengths clearly indicated that three dams in No. 3 seam i.e. dam nos. 6A, 6B and 7 were in deteriorating conditions in their outer lining walls. They were also connected directly to the water pressure through four boreholes. Schmidt Hammer tests indicated that their compressive strength values were also less than 10 N/mm^2. But, as the maximum magnitude of vibration monitored during the period of study was only 5.90 mm/s (in blast no. 8), no damage or any adverse impact was created on these dams as well as on the surrounding roof and pillars.

The dam stability was evaluated as per the design and measurement of the exposed dimensions. It was found that the dams are stable as far as the shearing force of the water head is concerned. The water head coupled with a higher magnitude of ground vibrations could destabilize the dams. The maximum water head recorded during the peak rainy season was 24 m only. But, the dams were designed to withstand a maximum water head of 120 m. The recorded peak water head in the rainy season was much less compared to the designed water head of the dam. The threshold level of ground vibration for underground water dams was kept as 25 mm/s. This level of vibration would produce much less tensile stress on the concrete dams than their actual tensile strengths. The recorded ground vibration data varied between 0.50 and 5.9 mm/s. An empirical equation based on the regression analyses of the vibration data was suggested to predict the magnitude of vibration near the underground water dams due to opencast blasting. The maximum explosive charges per delay for different distances for the safety of dams have been recommended.

Acknowledgements

The authors are thankful to Dr. Amalendu Sinha, Director, Central Institute of Mining and Fuel Research, Dhanbad, for his kind permission to publish this scientific paper. The ideas and findings mentioned in the paper are those of the authors and not necessarily of the organizations to which they belong.

References

1. Singh, P.K., Roy, M.P., and Singh, R.K. Responses of roof and pillars of underground coal mines to vibration induced by adjacent open-pit blasting. *Environmental Geology*, 2005, pp. 205–214.
2. Tunstall, A.M. Damage to underground excavations from open-pit blasting. *The Transaction of Institute of Mining and Metallurgy* (Sec. A: Mining. Industry), vol. 106, 1997. pp. A19–A24.
3. Rupert, G.B. and Clark, G.B. Criteria for the proximity of surface blasting to underground coal mines. *Proceedings of Rock Mechanics Symposium*, Colorado School of Mines, 1977, pp. 3C31–3C310.
4. Jensen, D.E., Munson, R.D., Oriard, L.L., Reitman, J.D., and Wright, R.S. Underground vibration from surface blasting at Jenny Mine. Final contract RPT J0275030 for the US Bureau of Mines. 1979, p. 99.
5. Kidybinski, A. Design criteria for roadway supports to resist dynamic loads. *International Journal of Mining Engineering*, vol. 4, 1986. pp. 91–109.
6. Fadeev, A. B., Glosman, L. V., and Sofonov, L. V. Seismic control of mine and quarry blasting in the USSR. *Proceedings of International Congress on Rock Mechanics*, Montreal, Canada. International Society of Rock Mechanics, 1987. pp. 617-619.
7. Fourie, A.B. and Green, R.W. Damage to underground coal mine caused by surface blasting. *International Journal of Surface Mining and Reclamation*, vol. 7, 1993. pp. 11–16.
8. Lewandowski, T., Keith, G., Croucher, M., and Richards, A. The impact of surface blasting on underground opening—Geotechnical Measurement. *Proceedings of 6th International Symposium on Rock Fragmentation by Blasting, Fragblast-6*. The South African Institute of Mining and Metallurgy. 1999. 131 p.
9. Pal Roy, P. *Rock blasting effects and operations*. Oxford and IBH Publishing Company Pvt. Ltd., New Delhi and Taylor & Francis Group plc., U.K., 2005, 345 p.
10. Siskind, D.E. *Vibrations from blasting*. International Society of Explosives Engineers (ISEE), USA, 120 p. ◆