Final Report

Role of blast design parameters on ground vibration and correlation of vibration level to blasting damage to surface structures

S&T Project: MT/134/02

Project Leader	**Dr. G.R. Adhikari**
Co-investigators	**Dr. H.S. Venkatesh**
	A.I. Theresraj
	Surendra Roy
	R. Balachander
	Nitin Kumar Jain
Project Advisor	**Prof. R.N. Gupta**

Implementing Agency	**National Institute of Rock Mechanics**
Collaborating Agencies	**Western Coalfields Limited**
	Singareni Collieries Company Limited

September 2005

ABSTRACT

Ground vibration induced by blasting is a serious environmental issue in Indian mines. With the increasing production targets from surface mining, it is likely to be compounded in future unless pro-active measures are taken to mitigate the problem. In response to the need of the mining industry, an S&T (Coal) project was undertaken by National Institute of Rock Mechanics (NIRM) in collaboration with Western Coalfields Limited (WCL) and Singareni Collieries Company Limited (SCCL). The main objectives of this project were: 1) to establish a rational damage criterion for surface structures with reference to Indian conditions, and 2) to suggest measures for effective control of ground vibration due to blasting.

The first step in this study involved analysis of the data available with NIRM on ground vibration due to blasting at different surface mines. The analysis revealed that the dominant frequency in coal bearing strata was low (< 8 Hz) and hence the permissible peak particle velocity as per the current DGMS standard is 5 mm/s. In complying with such a low statutory limit, coal mines located close to surface structures are struggling for their survival. Other conditions being similar, non-coal mines are in a better position because frequencies are relatively high for which permissible levels are also high.

Though ground vibrations have been monitored for several decades, there is no universally accepted method of transducer mounting. The influence of different methods of transducer mounting on vibration measurements was therefore conducted at Kamptee OCP of WCL. For this purpose, the first transducer was placed freely on a horizontal surface, the second one was 'sandbagged', the third one was 'spiked' and the last one was completely buried in soil. These transducers were mounted side by side and 14 blasts were monitored. The results indicate that decoupling was most likely with the surface transducer. However, the sandbagged and spiked transducers were also prone to decoupling. Decoupling could result in higher or lower ground vibration. Therefore, burial should be the preferred method for mounting of transducers in soil.

Having established a suitable procedure for mounting of transducers, a comprehensive monitoring programme was designed and implemented at Kamptee OCP of WCL and OC-2 of SCCL. It included direct measurement of structure response and assessment of damage to four existing structures at Kamptee OCP and to three test structures that were constructed at

OC-2 exclusively for this purpose.

Measurement of structure response to ground vibrations reaffirmed the role of frequency of ground vibration in causing structure response and thereby increasing the damage potential of the structures. More importantly, it provided a basis for categorisation of frequencies that could be used in a vibration standard. Depending on amplification (response) factor, frequencies have been categorised into low (<20 Hz), medium (20-50 Hz) and high (>50 Hz).

Pre- and post blast survey of the structures was systematically carried out along with vibration monitoring for a large number of blasts. No visible damage to these structures was observed even at vibration levels more than four times the current permissible limits. Therefore, the permissible peak particle velocity (PPV) of 5 mm/s in the low frequency range can be safely increased to 10 mm/s. For higher frequencies, PPV can be still higher.

Another important field programme included monitoring in and around the mines to study the influence of blast design parameters on ground vibration. For both the mines, maximum charge per delay and the delay interval were found to be the most important design parameters that can control ground vibration. For Kamptee OCP, the availability of free faces and the slurry explosives used also had significant influence whereas the total charge had an insignificant influence on ground vibration. On the other hand, it was observed that the frequencies of ground vibration were confined to certain limits that could not be altered by modifying the blast design parameters.

When predicted or monitored vibrations exceed the statutory limits, ground vibrations are to be controlled by modifying the blast design parameters. In critical situations, digging a trench between the blast and the structure can further reduce ground vibration. The extent to which it can reduce ground vibration has been examined by numerical modelling. The results show that the percentage of reduction depends on the trench depth to blasthole depth ratio. At a ratio between 1.0 and 1.5, which seems to be feasible, vibration was reduced by 55 per cent.

Apart from suggesting the methods for monitoring and control of ground vibration, this study sets the stage for revision of the current DGMS standard which will help the mining industry.

Contents **Page**

Chapter 1
INTRODUCTION

1.1 Indian Mining Industry

The Indian mining industry, which exploits non-renewable resources for meeting the material needs of the society, makes valuable contributions to society and the progress of the nation. This industry contributes over 3.5 per cent of the gross domestic product. Besides sizeable direct contribution to Government revenue and significant export earnings, it also provides direct employment to over 2.5 million persons. The mining industry has contributed significantly to the development of infrastructure in the nation and catalysed extensive economic development of remote and backward regions.

Ranked third in the world, India's coal production has crossed 324 million tonnes per annum. Iron ore production at 120 million tonnes occupies the fourth rank in the world. Limestone production has increased to 154 million tonnes and bauxite to 10.95 million tonnes. Most non-metallic mines have also increased production. Over 70 percent of coal production comes from surface mining whereas iron ore, limestone, bauxite and most non-metallic minerals are produced by surface mining alone. Ever growing demand for coal and minerals and the pressure for cost reduction has compelled the mining industry to increase the scale of operations requiring large blasts to feed their high capacity earth moving equipment. This in turn has caused adverse impacts on environment in the form of ground vibration due to blasting, which are by and large controllable.

1.2 Environmental Impacts of Blasting in Mining

Blasting is the principal method of rock breakage in mining and construction projects throughout the world. This may probably be due to its distinct advantages like economy, efficiency, convenience and ability to break the hardest of rocks. However, only a portion of the total energy of the explosives used in blasting is consumed in breaking rocks while the rest is dissipated. The dissipated energy creates environmental problems in the form of ground vibration, air overpressure and flyrock. With increasing mining and construction activities in areas close to human settlements, ground vibration has become a critical environmental issue as it can cause human annoyance and structural damage.

1.3 Statement of the Problem

In order to protect surface structures from the deleterious effect of ground vibration, regulations have been formulated in different countries. These regulations vary from country to country depending on the type and the construction materials used. In India, the Director General of Mines Safety (DGMS) through its Circular No. 7 of 1997 specified the permissible limits of ground vibration for different types of structure (Table 1.1). The DGMS Circular has categorised surface structures into two categories based on the ownership. For each category, there are three types of structure for which permissible peak particle velocity (PPV) has been specified depending on the frequency. This circular does not make reference to any scientific study conducted in India or abroad. Perhaps it was based on the experience of the DGMS on controlled blasting close to surface structures.

Table 1.1 Permissible PPV (mm/s) as per DGMS (Tech)(S&T) Circular No. 7 of 1997

Type of structure	Dominant excitation frequency, Hz		
	< 8 Hz	8 – 25 Hz	> 25 Hz
A) Buildings/ structures not belonging to the owner			
Domestic houses/ structures (Kuchha brick and cement)	5	10	15
Industrial Buildings (RCC and framed structures)	10	20	25
Objects of historical importance and sensitive structures	2	5	10
B. Buildings belonging to owner with limited span of life			
Domestic houses/ structures (Kuchha brick and cement)	10	15	25
Industrial buildings (RCC & framed structures)	15	25	50

The mining industry has been implementing the DGMS standard over the last eight years. Due to the stringent vibration levels, charge per delay was very low and in many cases lower than the charge per hole. In order to comply with permissible limits, mines decreased the size of blasts, resorted to the use of smaller blasthole diameter and/or bench height. All these measures reduced production and productivity and increased the cost of production. Compared to the permissible levels adopted in other countries, the vibration levels in India, particularly at frequencies below 8 Hz, appear to be conservative. With due emphasis on the safety of surface structures, it has become necessary to look into the current vibration standard.

Though exhaustive studies related to ground vibration and structure damage have been conducted abroad (Siskind et al, 1980), their findings may not be directly applicable for surface structures that are normally found in mining areas in India. Some work has been conducted in India (Singh et al, 1993) but they are not conclusive enough to determine threshold values of damage vis-à-vis permissible levels of ground vibration. Further studies are needed to establish maximum permissible levels for Indian conditions.

With the development of mini-seismographs and analysis software that are available at an affordable cost, most of the mining companies have started regular monitoring of ground vibration. Since most of the standards including the DGMS one do not suggest methods for transducer mounting, different methods followed may influence vibration measurements. Studies in this area are needed to formulate a procedure for vibration monitoring.

If ground vibration at a point of concern is greater than the permissible level, it has to be controlled. It is usually the maximum charge per delay that is restricted for this purpose. In reality, a large number of variables influence ground vibration. The degree to which each variable influences ground vibration is to be established and suitably incorporated in blast designs for effective control of ground vibration.

1.4 Objectives of the Study

1. To establish a procedure for monitoring ground vibration at opencast mines.
2. To study the influence of blast design parameters on ground vibration.
3. To study the influence of delay interval and delay sequence on ground vibration.
4. To study the influence of explosive type on ground vibration.
5. To study the efficacy of vibration isolators through numerical models.
6. To correlate vibration level with damage to surface structure.

1.5 Scope of the Work

This study deals with ground vibration due to blasting at surface mines with a focus on the current Indian vibration standard and the proposed one. A large body of data available with National Institute of Rock Mechanics (NIRM) at different surface mines has been analysed keeping the objectives in view. Further field investigations were carried out at two surface coal mines in collaboration with Western Coalfields Limited (WCL) and Singareni Collieries Company Limited (SCCL) under a Coal (S&T) project from October 2002 to September 2005.

Chapter 2

PARAMETERS OF GROUND VIBRATION AT SURFACE MINES - AN OVERVIEW

2.1 Introduction

National Institute of Rock Mechanics (NIRM) had carried out several vibration studies earlier at different mines to assess and control ground vibration. Besides peak particle velocity and frequency, the data generated included blast design parameters, maximum charge per delay, distance between the blast and the transducer. A cursory look at the data indicated that the permissible levels for a given type of structure at a given distance varied across the mining industry. A detailed analysis was carried out in this chapter to understand the implications of the DGMS permissible levels for the mining industry. The data from following mines/projects were selected for back analysis:

- Three opencast projects (OC-1, OC-2 and OC-3) of Godavarikhani area, SCCL
- Three opencast projects (OC-1, OC-2, and Centenary) of Manuguru area, SCCL
- Neyveli lignite mines of Neveli Lignite Corporation Limited (NLC)
- Donimalai and Bailadila iron ore mines of National Mineral Development Corporation
- Kudremukh Iron Ore mine of Kudremukh Iron Ore Company Limited (KIOCL)
- Malanjkhand copper project of Hindustan Copper Limited
- Rampura Agucha mine of Hindustan Zinc Limited
- Panna diamond mining project of National Mineral Development Corporation (NMDC)
- Several limestone mines
- Several construction projects

Among the selected mines, coal and iron ore mines represent large mechanised opencast operations in India, where large diameter deep hole blasting has been in practice. Unlike these mines, no fragmentation or displacement of rock is required at Neyveli lignite mines where the purpose of blasting is only to loosen the rock for cost-effective operation of bucket wheel excavators.

Limestone quarries under the reference were developed in two or three benches of varying heights. The drillhole diameter was 100-115 mm, rarely up to 165 mm. Blasting was predominantly carried out using ANFO (ammonium nitrate mixed with fuel oil), primed with cap-sensitive slurries. Slurries were used in case of wet holes. Blastholes were initiated on a V-type or diagonal patterns using various types of initiation systems.

Malanjkhand copper project of HCL and Rampura Agucha mine of HZL are large mines in hard rock mining while Panna diamond mine of NMDC is the largest diamond mine in India.

Unlike mining blasts, the size of blasts in construction projects was small. Construction blasts usually employed small diameter holes (32-36 mm, rarely up to 100 mm) and used relatively small quantities of explosives in shallow holes up to a depth of 2.5 m for jack hammer drill and up to 6 m for wagon drill of 100 mm diameter. The holes were charged with explosives (nitroglycerine based, ANFO, slurry or emulsion) and initiated with various types of initiation systems.

2.2 Peak Particle Velocity

At a given location, peak particle velocity (PPV) depends on the distance from the blast and the maximum charge per delay. The DGMS Circular requires that square root scaling shall be used when blasting is carried out on the surface and vibrations are also monitored on the surface. The square root scaling to estimate PPV is given by (ISEE, 1998):

$$V = K\,(D/\sqrt{Q}\,)^{b} \qquad (1)$$

where V is the peak particle velocity (mm/s), D is the distance between the blast and the monitoring station (m), Q is the maximum charge per delay (kg), and K and 'b' are the site constants. Conventionally, $D/\sqrt{Q}$ is called scaled distance.

Peak particle velocity is plotted against scaled distance on logarithmic scales. The site constants for a mine can be determined by regression analysis of the data sets. The site constants for various surface mines with correlation coefficients between PPV and scaled distance are given in Table 2.1.

Table 2.1 Observed frequencies, site constants with correlation coefficients for different surface mines/projects

Industry	Mine	Number of blasts	Number of data	K	b	r	Frequency (Hz)
Coal	GDK OC-1	13	35	159.17	1.40	0.74	5 - 20
	GDK OC-2	11	43	119.11	1.30	0.85	5 - 40
	GDK OC-3	13	42	185.65	1.33	0.82	5 - 20
	All GDK data	37	120	146.89	1.30	0.84	5 - 20
	Manuguru	12	53	534.31	1.63	0.93	5 - 27
Lignite	Mine I and Mine II	22	68	858.90	1.58	0.86	5 - 27
Limestone	27 mines	_	740	320.81	1.30	0.77	10 - 60
Iron ore	Deposit 5, NMDC	4	16	66.44	1.17	0.79	3 - 14
	Deposit 11C, NMDC	6	15	100.00	1.40	0.96	2 - 15
	Deposit 14, NMDC	3	10	48.60	0.80	0.72	2 - 16
	Donimalai, NMDC	13	38	69.30	1.16	0.87	2 - 20
	All NMDC data	26	79	70.30	1.16	0.85	2 – 20
	Kudremukh mine	260	260	65.35	1.15	0.66	2 – 30
Hard rock mines	Malanjkhand Copper Project, HCL	21	24	303.75	1.54	0.75	5 - 20
	Rampura Agucha mine, HZL	10	31	211.82	1.42	0.86	11 - 75
Precious stone	Panna diamond mine, NMDC	6	25	501.29	1.56	0.94	10 - 70
Construction projects	13 different sites	_	356	67.85	0.85	0.58	11 - 200

Note: r = correlation coefficient, K and b are site constants of Equation (1)

GDK = Godavarikhani area

As seen in Table 2.1, constants K and 'b' vary for different mines. However, the variation of K is much more pronounced than that of 'b'. It implies that K is very sensitive compared to 'b'. The constants K is affected by different factors depending on the various conditions prevailing at the site.

The DGMS Circular mentions that the coefficient of correlation should be high but it does not state what is high or low. It is important that the coefficient should be statistically significant for the number of data used in the analysis. The lowest correlation coefficient was obtained for construction blasts due to wide variations in rock and the highest for coal mines of Manuguru area where the geological and mining conditions were similar. The correlation coefficient for KIOCL is relatively low as the data pertain to a period of over five years during which the rock being excavated might have varied. Moreover, this was the period when KIOCL was evolving optimum blast design by conducting field trials.

Fig. 2.1 shows peak particle velocity against scaled distance for three coal mines (OC-1, OC-2 and OC-3) of Godavarikhani area. It reveals that the data of the individual mines in the same area follow a similar trend. This is important when vibrations induced by blasting at a new project in the same area are to be estimated. The impact of blasting at OC-4 of Manuguru area, which was then at the proposal stage, was assessed using the data from three adjoining mines, namely OC-1, OC-2 and Centenary (Fig. 2.2).

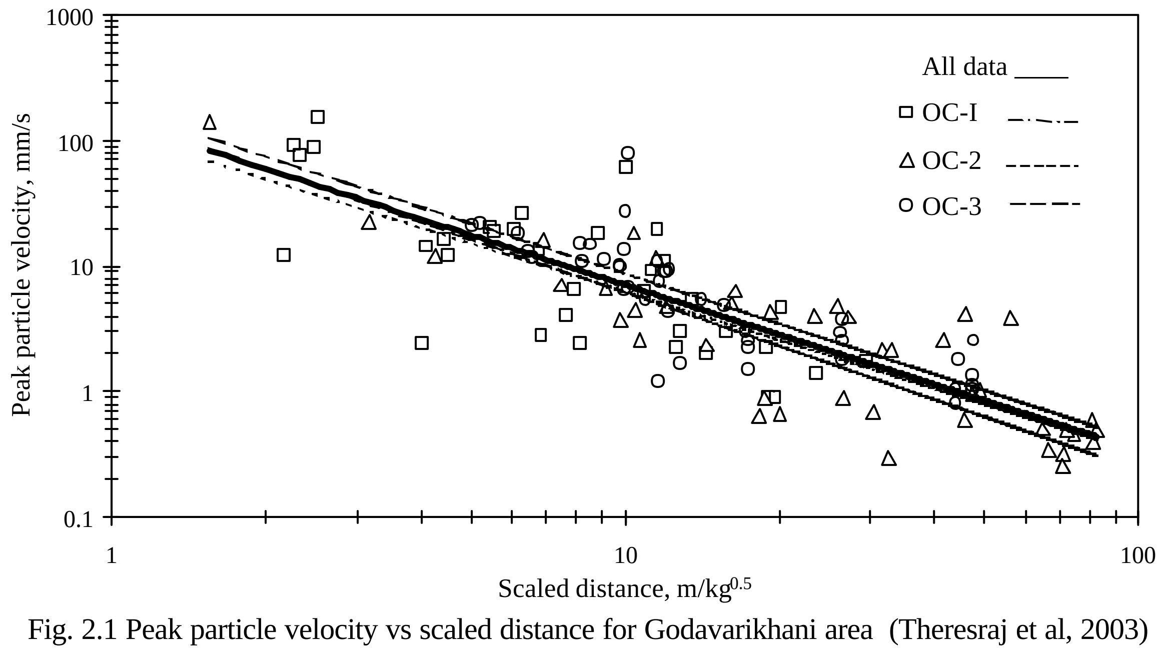


Fig. 2.1 Peak particle velocity vs scaled distance for Godavarikhani area (Theresraj et al, 2003)

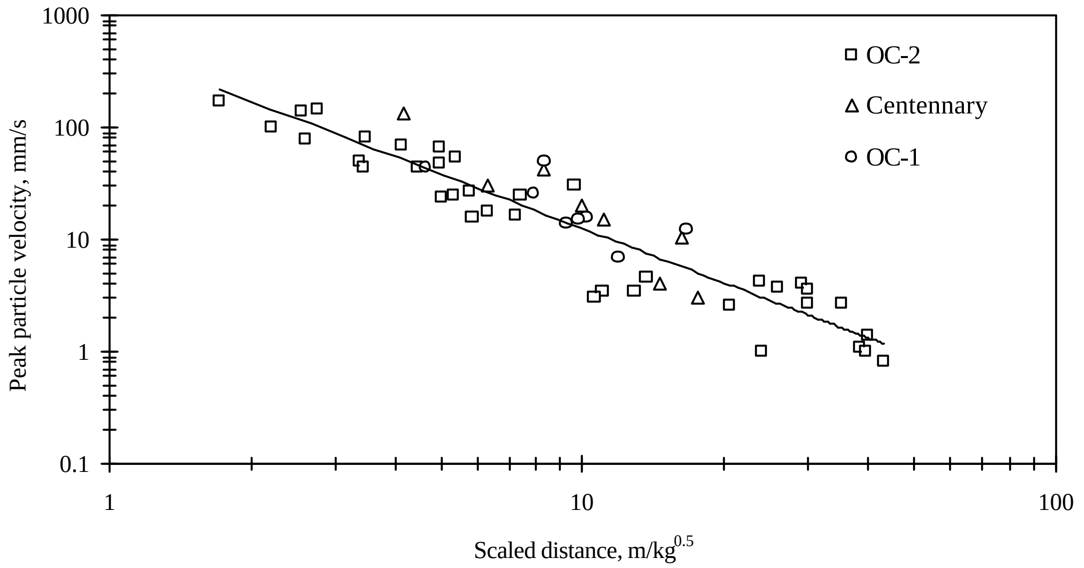


Fig. 2.2 Peak particle velocity vs scaled distance for Manuguru area, SCCL

For Neyveli Lignite mines, 84 sets of data collected from Mine II and Mine I were used for regression analysis (Fig. 2.3). Although most of the data are from Mine II, the result is valid for other mines of NLC as long as the geology of the area and blast design remains the same.

The blast vibration data generated from different iron ore mines of NMDC were plotted individually and also as a single group. Fig. 2.4 reveals that the data of the individual mines in the same group follow a similar trend with an exception of deposit 14, probably due to insufficient number of data.

The attenuation pattern for KIOCL using the data collected by the mine over a period of five years (Vidyarthi, 2004) is shown in Fig. 2.5. There is large scatter of data, possibly due to the variation in rock types and blast design patterns over the corresponding period.

The vibration data generated from a number of limestone quarries were combined and plotted as a single group (Fig. 2.6). It may be noted that the data follow a trend, which can be represented by a generalised equation.

Data generated from 13 construction sites were also compiled and analysed to predict PPV from construction blasts (Fig. 2.7). Due to variations in blasting techniques and the site conditions, the correlation coefficient is rather low.

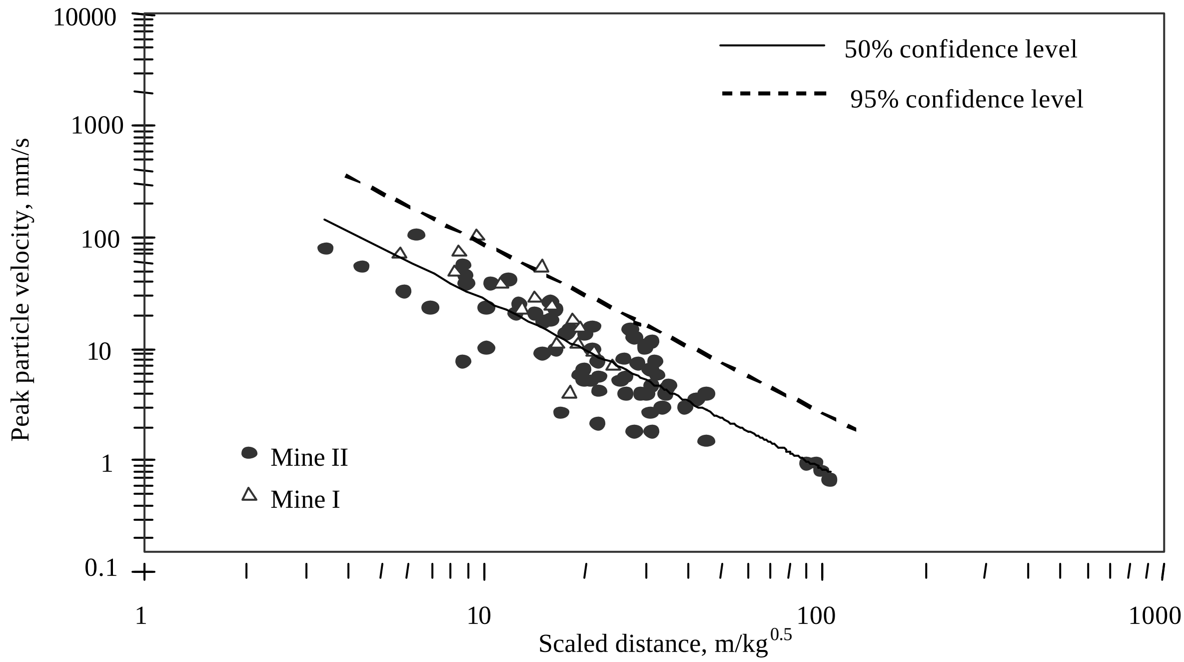


Fig. 2.3 Peak particle velocity vs scaled distance for Neyveli mines, NLC (Theresraj et al, 2004)

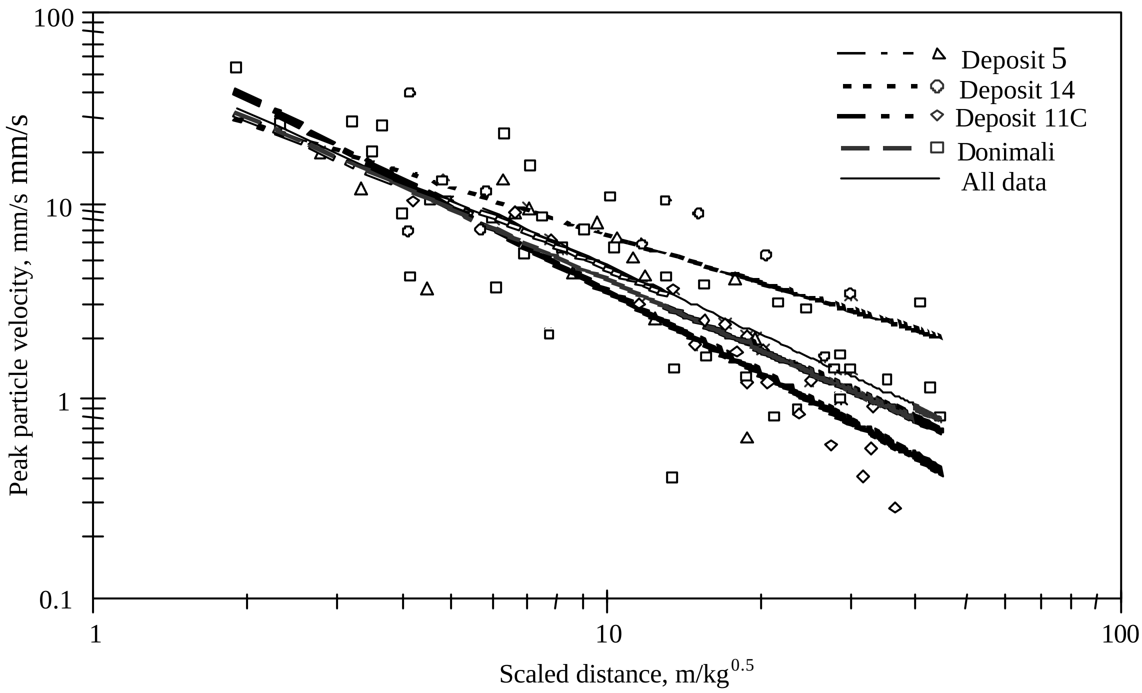


Fig. 2.4 Peak particle velocity vs. scaled distance for iron ore mines of NMDC (Theresraj et al, 2003)

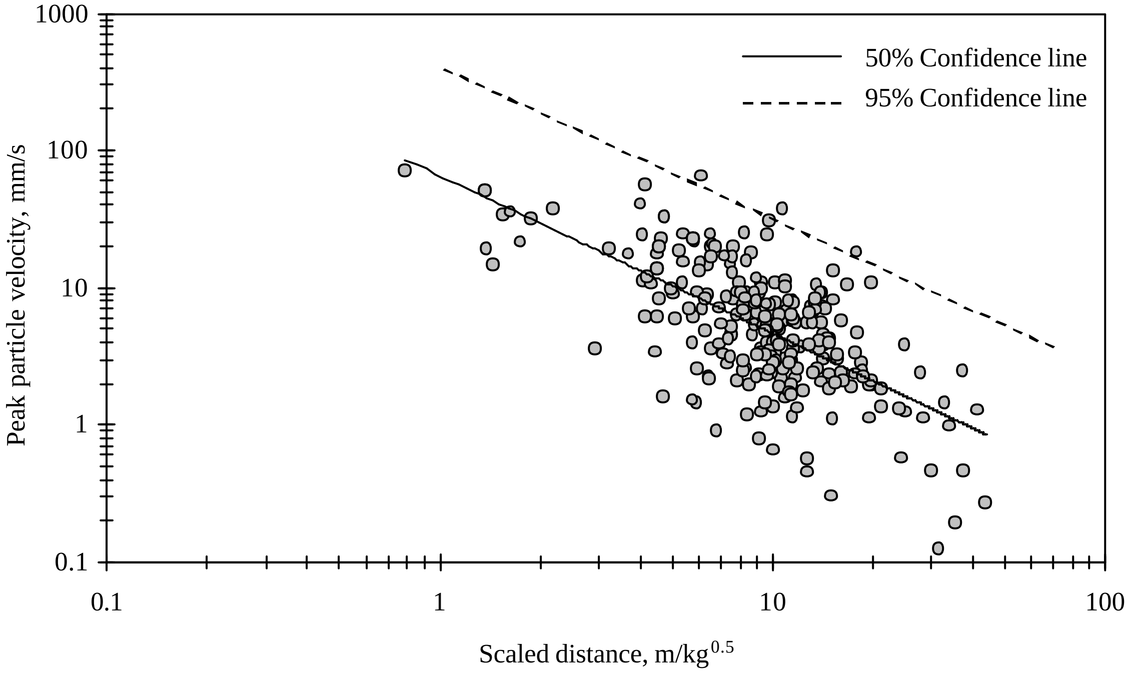


Fig. 2.5 Peak particle velocity vs scaled distance for Kudremukh iron ore mine

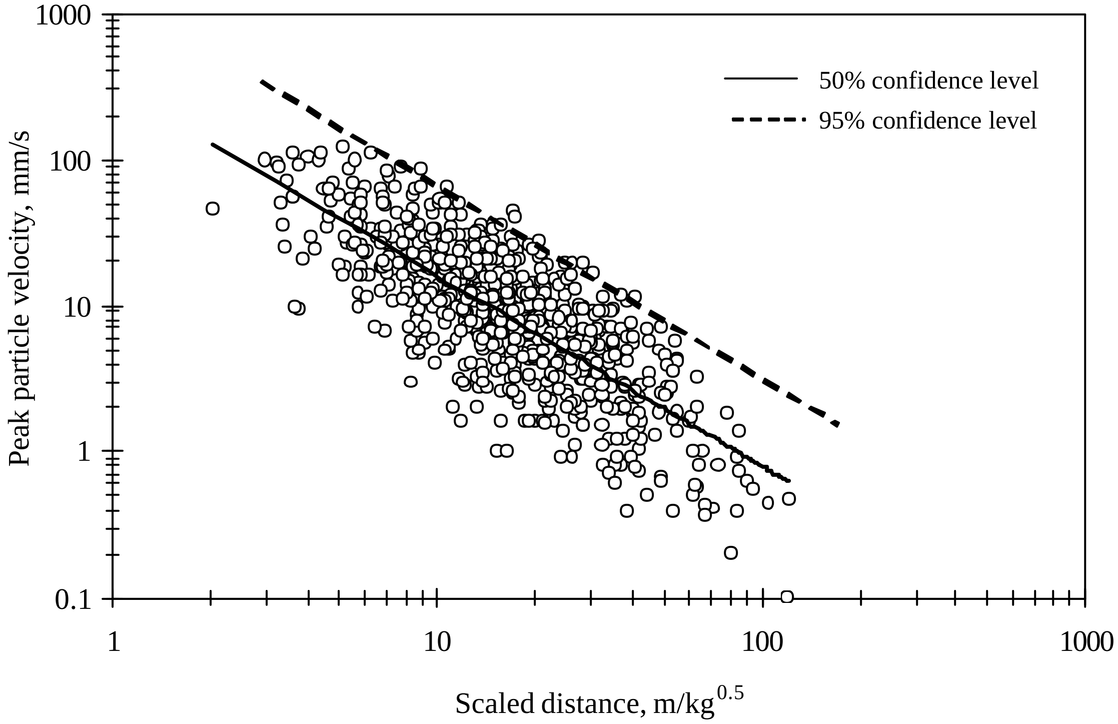


Fig. 2.6 Peak particle velocity vs scaled distance for limestone quarries (Adhikari et al, 2004)

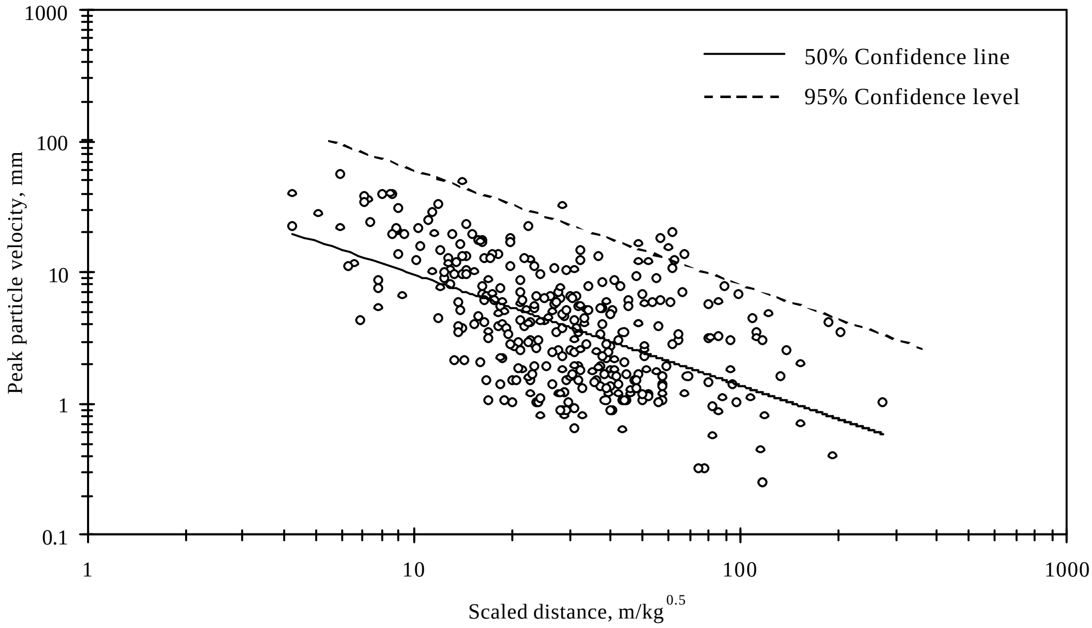


Fig. 2.7 Peak particle velocity vs scaled distance for construction projects (Adhikari et al, 2005)

For a given mine, PPV-scaled distance relation shows a large scattering of data about their mean which might be due to variation in delay interval, initiation sequence and other blast design parameters.

Substituting the values of K and 'b' for different mines from Table 2.1in Equation (1), the mean attenuation lines for different mines were plotted (Fig. 2.8). Peak particle velocity at a given scaled distance is highest for Neyveli, though the attenuation lines for Malanjkhand copper project, Rampura Agucha mine and Panna diamond project are not shown in this figure. The possible reason for the highest PPV at Neyveli could be the low specific charge followed at the mine (Jemino et al, 1995) and/or the higher water table and wet ground condition (Beattie, 1992). The lowest PPV is noted for iron ore mines. Despite the fact that NMDC exploits haematite ore and KIOCL banded magnetite quartzite, the attenuation lines for these mines are comparable. The generalised attenuation lines for coal mines of Manuguru and Godavarikhani area are not comparable.

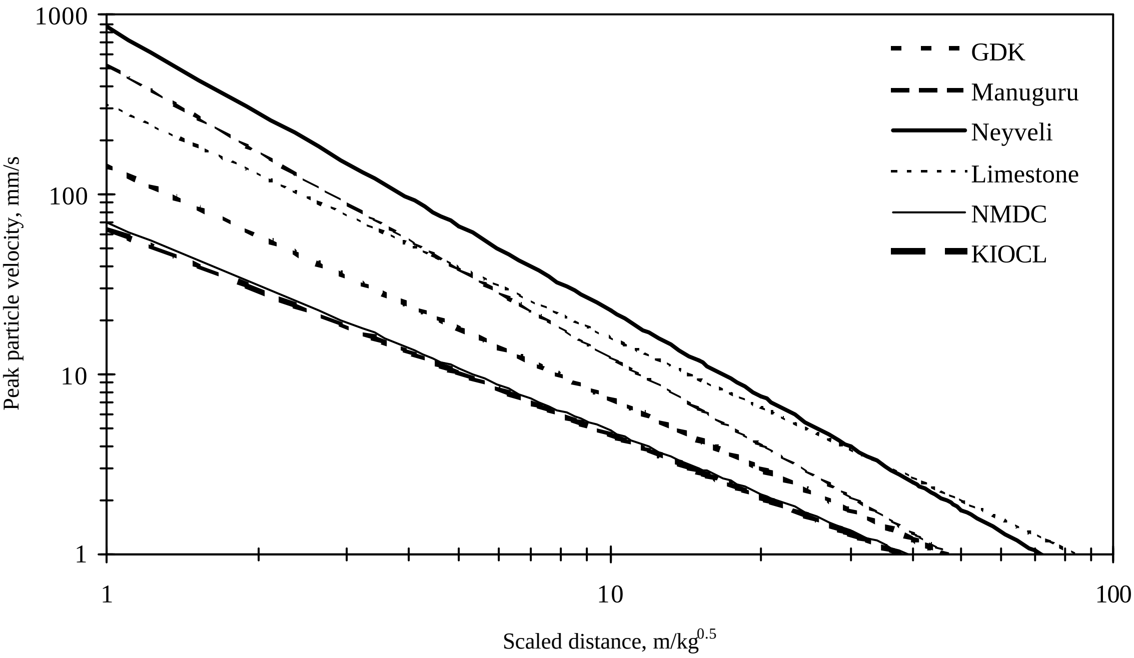


Fig. 2.8 Comparison of peak particle velocities for different surface mines

2.3 Frequency of Ground Vibration

The dominant frequency of ground vibration was determined through the software provided by manufacturers of the instruments. The range of observed frequencies for different mines and construction projects are also given in Table 2.1.

For Godavarikhani area, the dominant frequency of the ground vibration varies between 5 and 20 Hz at OC-1, 5 and 40 Hz at OC-2 and 5 and 20 Hz at OC-3 (Fig. 2.9). For Manuguru area, it varies between 5 and 30 Hz. In other coalfields also, presence of low frequency is reported (Bhushan and Sharma, 1992). At Neyveli lignite mines, frequency less than 10 Hz is usually present, though it varies from 5 to 27 Hz.

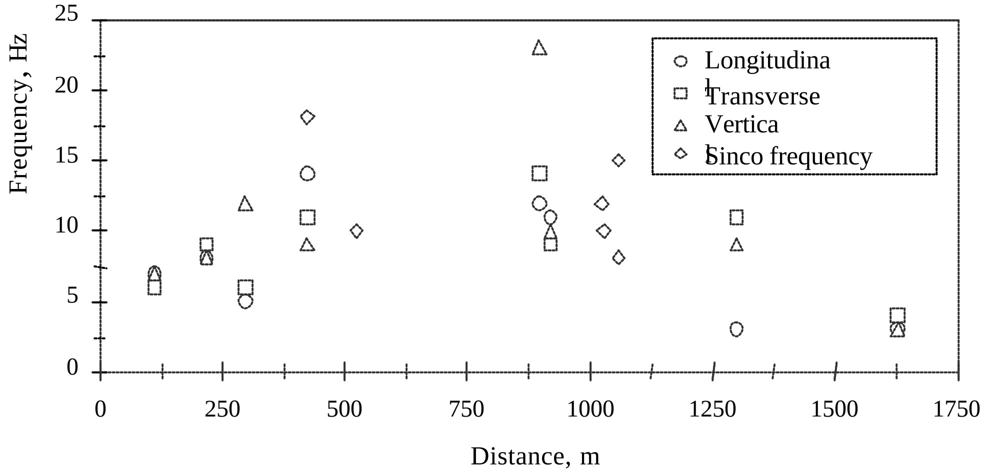
For iron ore mines too, the dominant frequencies for all the four iron ore mines of NMDC are below 20 Hz (Fig. 2.10). The frequency at KIOCL is also within 20 Hz in most of the cases (Fig. 2.11).

Frequency at Malanjkhand Copper mine ranged from 5 to 20 Hz while it was higher than 10 Hz at Rampura Agucha mine and Panna diamond mine.

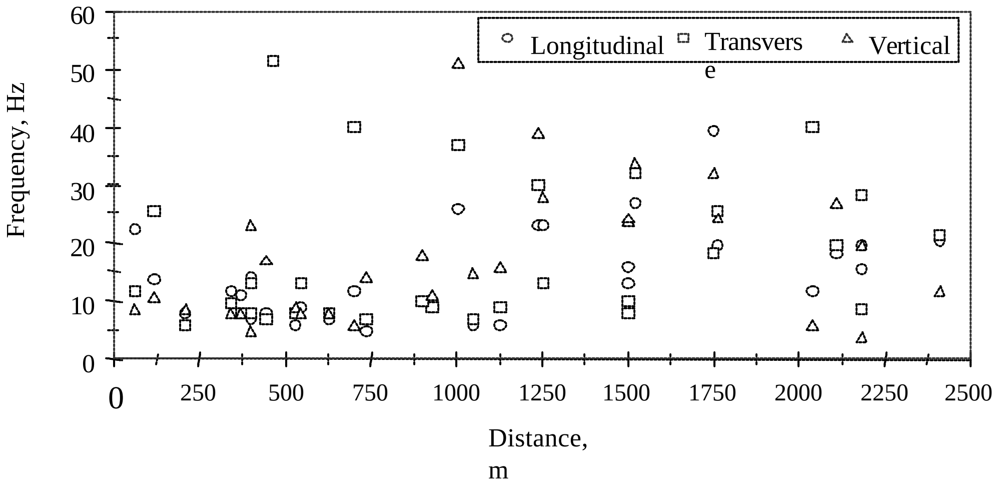
Assuming that frequency decreases with distance, the frequencies were plotted against distances for Godavarikhani area (Fig. 2.9), NMDC mines (Fig. 2.10) and KIOCL mine (Fig. 2.11). Unlike frequencies content in earthquake records, which gradually become lower with increasing distance due to preferential attenuation of high frequencies (Agrawal, 1991), frequencies of ground vibration remained more or less same with the distance. However, few data were available for distances less than 50 m from the blasts because the previous studies were concerned with far field monitoring of ground vibration.

A histogram of frequency range was plotted to know the most common frequency of ground vibration for limestone. Fig. 2.12 shows that the frequency is confined within 10-60 Hz in spite of differing blast geometries and the explosives used. Compared to mining blasts, higher frequencies were noted for construction blasts, which varied from 10 to 200 Hz typically greater than 20 Hz.

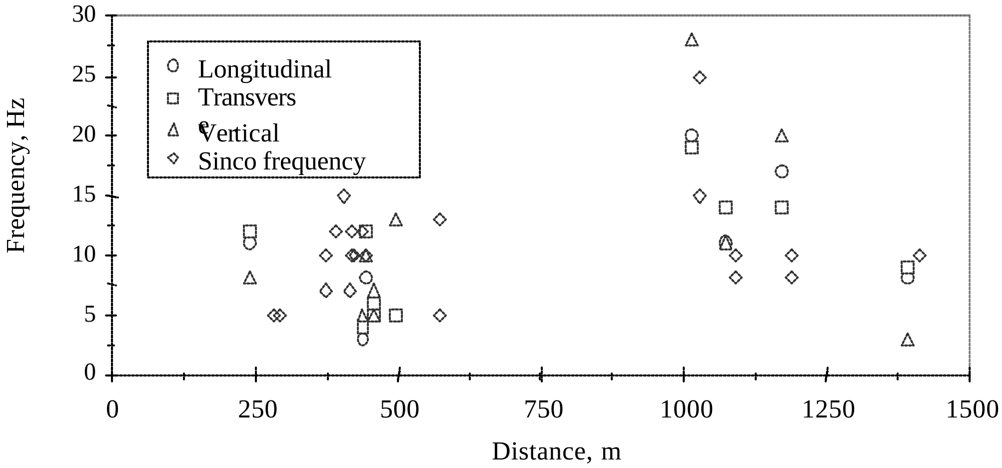
In spite of the differing blast geometries and the explosives used, blasts in coal, lignite and iron ore mines produced low frequency. Relatively higher frequencies were found in case of limestone quarries and construction projects. It is therefore inferred that large blasts using higher bench heights and larger diameter blastholes are more likely to produce lower frequency of ground vibration. These findings in general are similar to those of the U.S. Bureau of Mines (Siskind et al, 1980). They have found that frequency varied by industry. The lowest frequencies were associated with coal mine blasting, intermediate with quarry blasting and high frequencies with construction blasting. They inferred that relatively large blasts in coal and iron ore mines, monitored at far off distances were likely to produce low frequencies. On the other hand, construction blasts employing smaller quantity of explosives, monitored at short distances had a tendency to produce the highest frequencies.



a) OC-1



b) OC-2



c) OC-3

Fig. 2.9 Frequency vs distance for coal mines of Godavarikhani area (Theresraj et al, 2003)

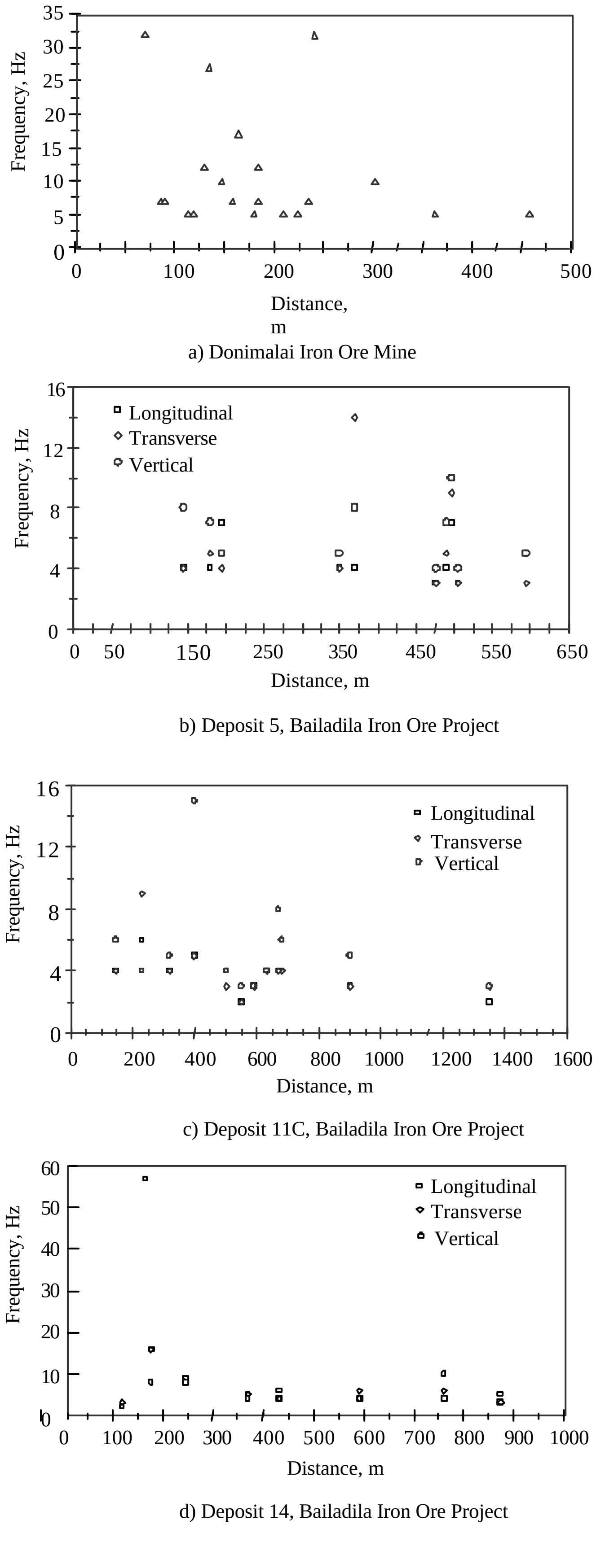


Fig. 2.10 Frequency vs distance for iron ore mines of NMDC (Theresraj et al, 2003)

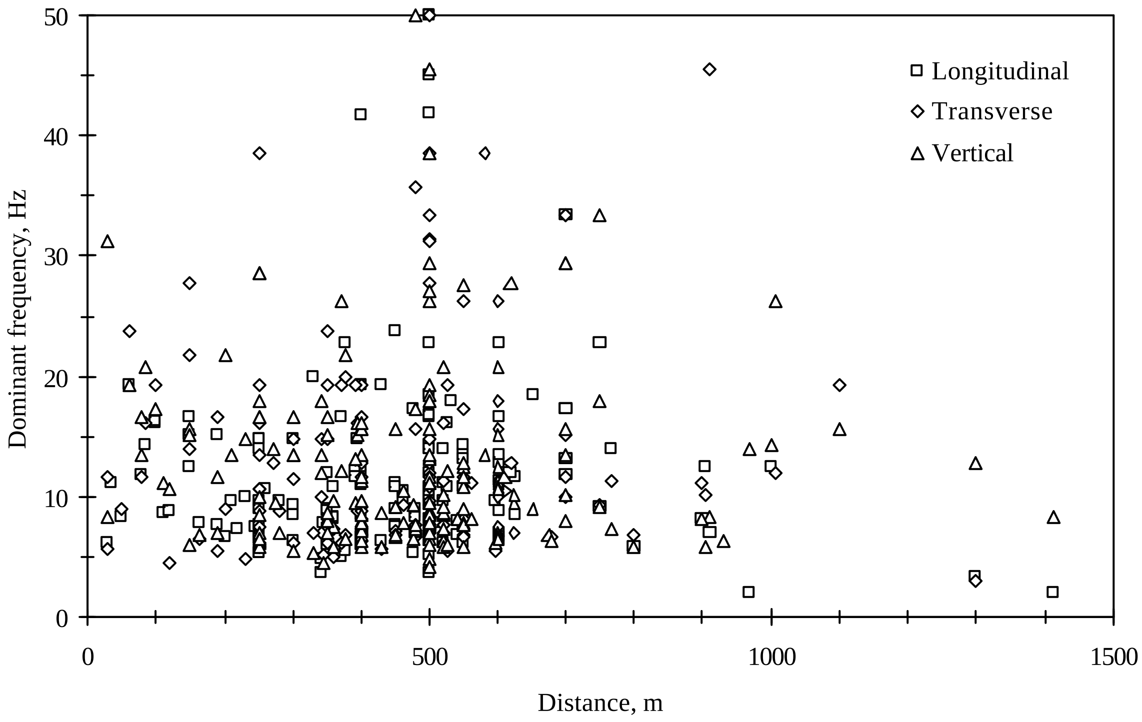


Fig. 2.11 Dominant frequency vs distance for Kudremukh iron ore mine

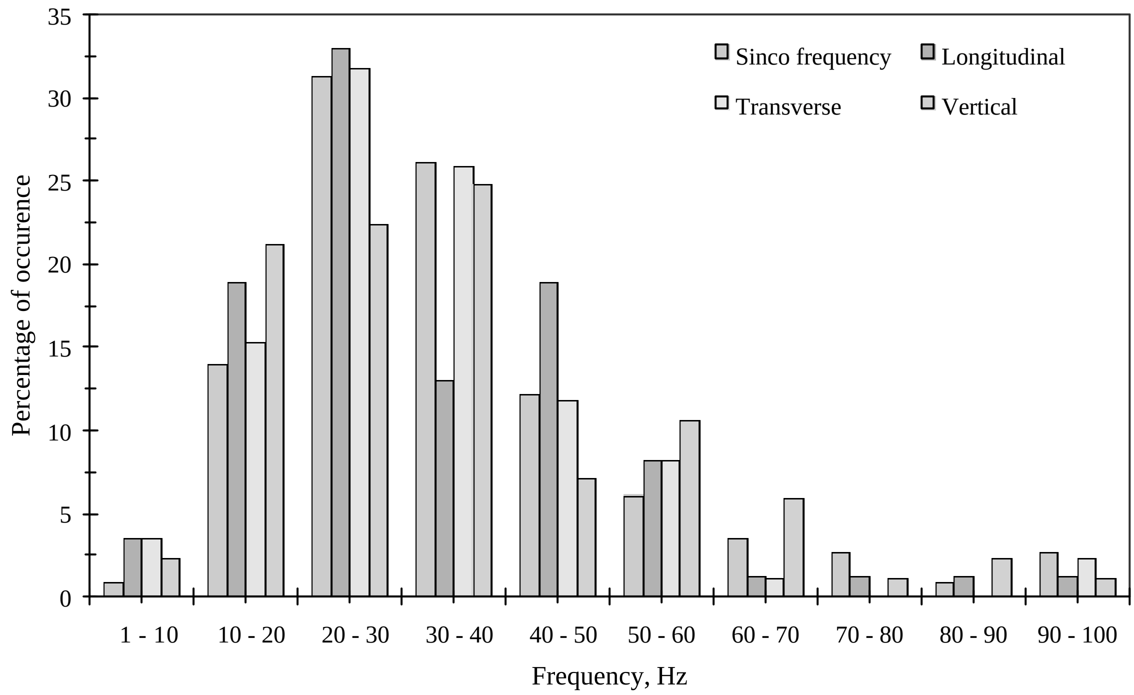


Fig. 2.12 Frequency content of ground vibration for limestone mines (Adhikari et al, 2004)

2.4 Observed Parameters Versus Statutory Compliance

As the frequencies of ground vibration for coal, lignite and iron ore mines are usually lower than 8 Hz, the permissible PPV as per the current regulation is 5 mm/s for residential structures that do not belong to the mines. Iron ore mines, which are usually located away from villages and townships, can comply with this limit without much problem. However, the imposed limit severely restricts the blasting operation in coal and lignite mines. The limestone quarries and construction projects are in a relatively better position, as frequencies are greater than 10 Hz for which permissible PPV is 10 mm/s.

If the frequency below 8 Hz can be shifted to higher ones, then higher PPV is permissible as per the existing vibration standard. Though the method of controlling frequency using an appropriate delay timing (Anderson et al, 1982) sounds reasonable, it is not effective, probably due to the use of pyrotechnic delays that are vulnerable to scattering in their delay timings. Moreover, higher frequencies can be generated only if they have a significant presence in the single hole waveform (Wheeler, 2005). As the frequency of ground vibration is difficult to alter, PPV has to be controlled within the specified level at the distance of concern. Due to the sprawl of dwellings around surface coal mines and the stringent statutory limits to be complied with, the mining industry has adopted various control measures. These measures has cut down the profit margins of most mines and put question marks for survival of some mines.

Chapter 3
SITE SELECTION AND TEST STRUCTURES

3.1 Site Selection

In consultation with the collaborating organisations the following mines were selected for conducting field investigations:

1) Kamptee OCP, Nagpur area of Western Coalfields Limited (WCL)
2) OC-2, Godavarikhani area of Singareni Collieries Company Limited (SCCL)

3.1.1 Brief description of Kamptee OCP

Kamptee OCP is located in Nagpur area of WCL. The leasehold area of the mine is 852 acres. It extract coal through ten workable seams namely Seam V, Seam IV (T), Seam IV (M&B), Seam III (A), Seam III (B), L1, II (T), I(T) & I (B). Out of all workable coal seams, only seams I(T), I(B) and L1 were virgin, rest of the seams were developed earlier by Bord & Pillar method of underground mining. Due to the steep gradient (1 in 3.5 to 1 in 4.5) and the stripping ratio of 2.62 m^3/t, horizontal slicing method with Shovel – dumper combination was adopted to produce the annual target of 0.65 Mt of coal. Some of the coal pillars were on fire due to spontaneous heating and hence, blasting in hot strata required adequate precautions.

The geology constitutes black cotton soil, sandstone and shale bands of thickness ranging from 28 to 32 m as overburden and from 24 to 34 m of coal. The coal bands are separated by shale beds intermittently the thickness of which ranges from 2 to 3 m.

3.1.2 Brief description of OC-2

OC-2 is located in the southern extremity of the Ramagundam coal belt on its outcrop side. The area covers 2.26 sq.km and lies around 0.6 km to south east GDK 10 Incline and about 1km south of GDK 10A Incline. There are seven seams occurring in this block which are numbered from top to bottom as 1A, 1, 2, 3B, 3A, 3 and 4 with the individual seam thickness ranging from 0.3 to 17.6 m, while the cumulative thickness of these seams vary from 9.76 to 30.79 m.

Fig. 3.1 A panoramic view of Kamptee OCP, WCL

In-pit crushing with conveying technology was the main feature of this project. Drilling, blasting and excavation with conventional shovel dumper combination was limited to feed the in-pit crusher. Dumpers of 85 T capacity transported the material to the crushers where the coal and overburden was crushed to 200 mm and 300 mm size making it amenable for belt conveyors. Overburden after crushing was carried out from crushers by steel cord belt conveyors and then dumped through the tipper car and spreader combination. Spreaders were crawler mounted and could dump to a width of 100 m, down dump of 30 m and high dump of 22.5 m from the crawler level.

3.2 Existing Structures for Damage Studies at Kamptee OCP

Surface structures such as evacuated/abandoned houses around the selected mines were surveyed so that they could be used for damage studies. There were several structures at Kamptee OCP, four of which were identified for damage studies.

The selected structures included two single storied houses of Central Recruitment Organisation (CRO) camp, Colliery Manager's old office building and Kali temple at Kamptee OCP. The blasting faces at this mine were progressing towards these structures.

The first structure consisted of one small room and a hall with a height of 3.1 m. The structure had an RCC roof with 260 mm thick brick wall. The second structure consisted of a small room (2.10 m by 2.0 m) and a hall (3.6 m by 3.75 m) with a height of 2.9 m. The roof of the hall was made of RCC whereas the room had a roof of asbestos sheet. The thickness of the brick wall was 260 mm including the thickness of plastering.

Fig. 3.2 View of the first structure selected for the damage study at Kamptee OCP, WCL

The third structure was the Colliery Manager's old office building. One of its rooms was selected for damage studies. The room had a dimension of 4.0 m × 4.0 m with a height of 3.1 m. The wall thickness was 300 mm, made of bricks and lime, plastered with cement and sand mixture. Roof was made of reinforced cement concrete and was supported by two I-section rails, placed 1.0 m apart. Kali temple, the fourth structure, had a circular wall of 3.0 m diameter and 3.0 m height, with a conical roof of 1.5 m height. It was an RCC structure with the wall thickness of about 250 mm.

Fig. 3.3 View of the second structure selected for damage study at Kamptee OCP, WCL

Fig. 3.4 View of the third structure selected for the damage study at Kamptee OCP, WCL

3.3 Design and Construction of Test Structures at OC-2

Since there were no suitable existing structures that could be used for damage studies around OC-2, three types of test structures representing typical residential structures in mining areas were constructed exclusively for this purpose (Fig. 3.5). The test structures consisted of:

a) Mud structure
b) Single storey two room brick structure with mud mortar
c) Double storey three room brick structure with cement mortar.

Figures 3.6 to 3.8 show the drawings of the structures. The structures were located on the top bench keeping the following points in mind:

- The test structures should be located in the direction towards which blasting faces are progressing.
- The test structures should be away from the blasting site to avoid cracking during construction period. At the same time it should be close to the blasting site so that the study could be completed within a period of five to six months.
- The site should be easily approachable for regular monitoring and damage studies.

Fig. 3.5 Test structures constructed exclusively for damage studies at OC-2, SCCL

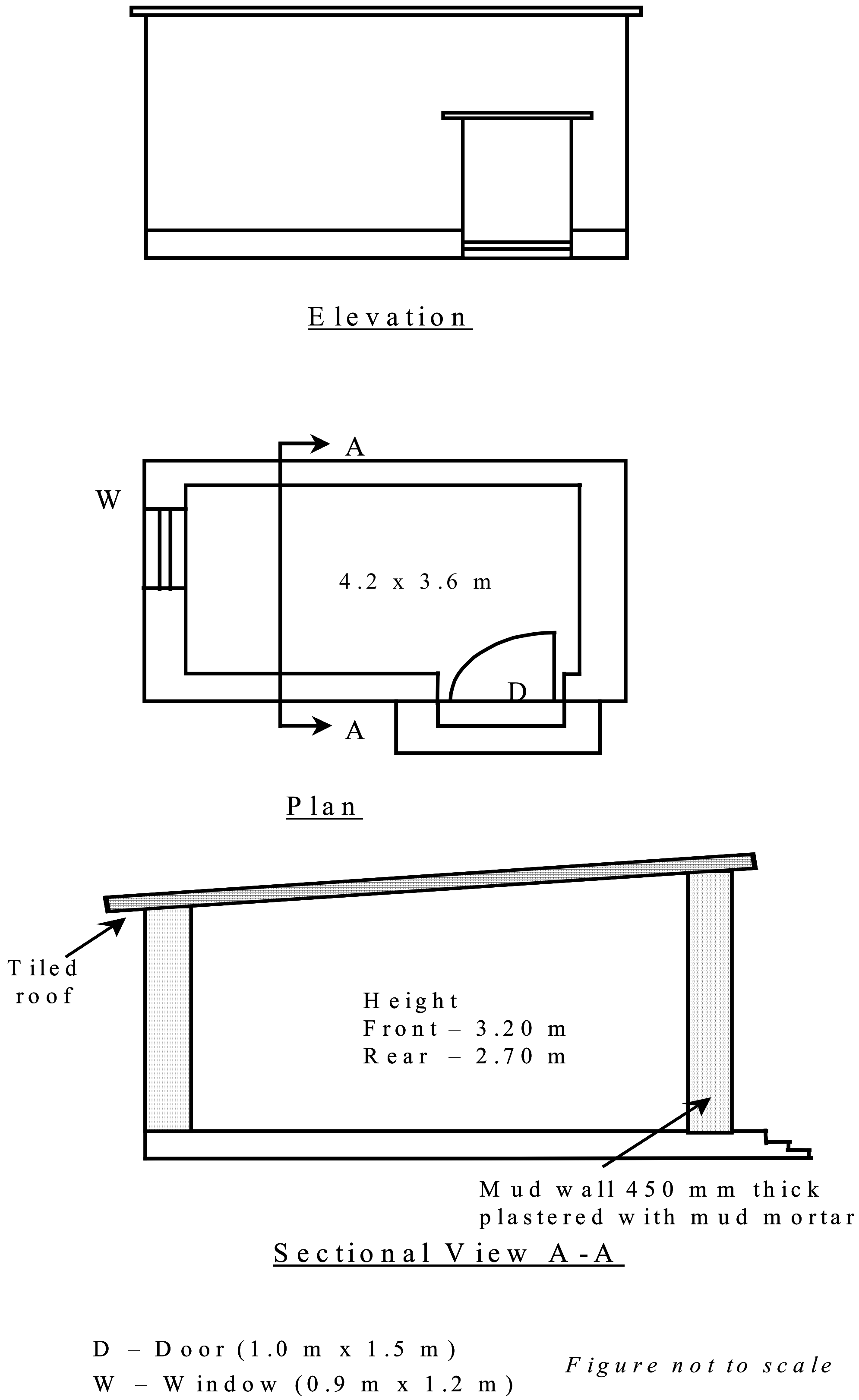


Fig. 3.6 Plan and section of the mud structure at OC-2, SCCL

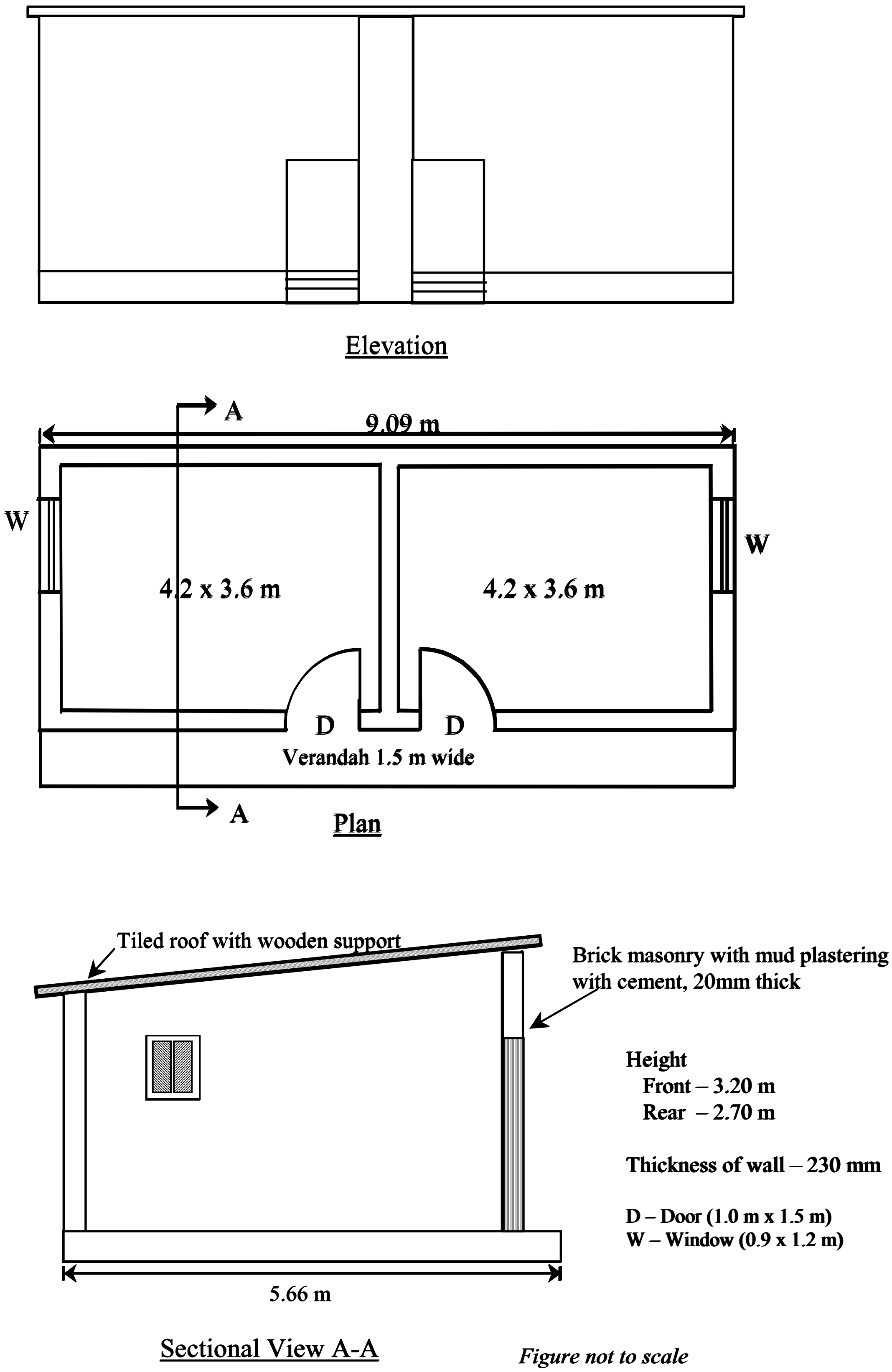


Fig. 3.7 Plan and section of the single storey structure at OC-2, SCCL

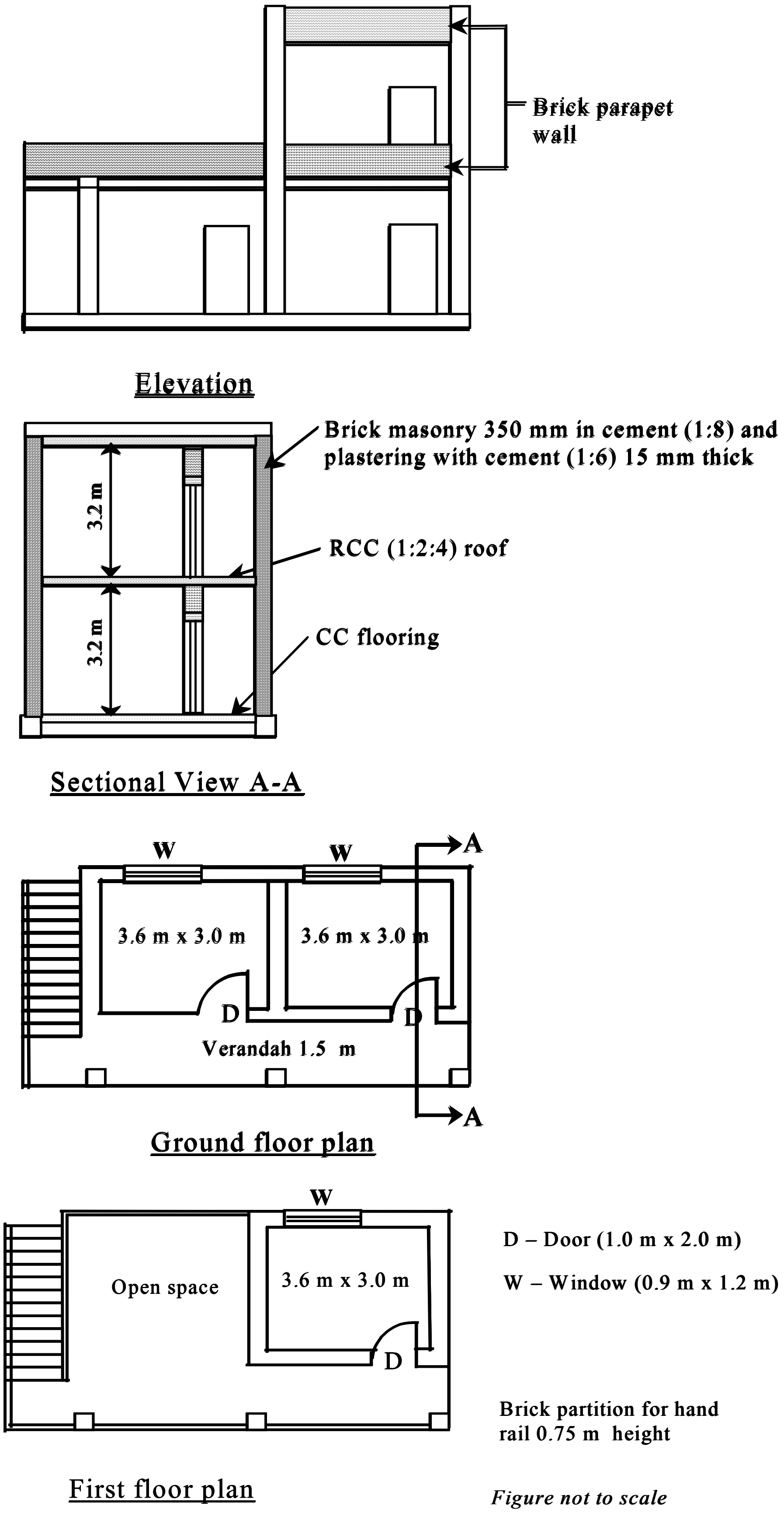


Fig. 3.8 Plan and section of the double storey structure at OC-2, SCCL

Chapter 4

INFLUENCE OF TRANSDUCER-GROUND COUPLING ON VIBRATION MEASUREMENTS

4.1 Introduction

One of the most critical aspects of ground vibration monitoring is the mounting (placement) of transducers in the field. Good coupling refers to the transducer that maintains proper contact with the ground. Poor coupling can cause slippage or toppling of the transducer resulting in distorted, often higher vibration levels.

Most recommendations agree that the best coupling can be achieved by burying the transducer when the measurement surface consists of soil and by bolting when the measurement surface consists of rock or concrete. Burial is desirable for particle acceleration exceeding 0.2 g but it is essential if it is greater than 1.0 g (Dowding, 1992; Stagg and Engler, 1980; ISEE, 2005).

If the friction between the geophone and the monitoring surface is sufficient to hold the transducer in place which is expected at acceleration less than 0.2 g, no burial or attachment is necessary (Dowding, 1992; Stagg and Engler, 1980; ISEE, 2005). This suggestion should be viewed with caution as the ground coupling problem is not only related to friction but also embedment, mount shape and the direction of motion. Moreover, Blair (1987) underlines that vibration transducers should never be placed free on the surface, whatever may be the anticipated vibration levels.

Many manufacturers of blasting seismographs supply their transducers with spikes and recommend using them. Spikes can be effective for ground accelerations less than 1.0 g (ISEE, 2005; Stagg and Engler, 1980) though Dowding (1992) specifically discourages the use of spikes since they may affect the response of the recorded motion. Blair (1995a) has reported that spiked mounting over-estimated the true ground vibration by 46.5% on the average and proposed a pre-cast mount embedded in soil (Blair, 1995b). It may be noted that spiked mount does not have to be decoupled to be poor.

With regard to the use of sandbags to improve coupling, Instantel (2003) recommends using them only when very small vibrations are expected. However, it does not specify the vibration level. Stagg and Engler (1980) mention that sandbags can be used when expected particle accelerations are below 1.0 g. On the other hand, others (Robertson, 1993; Blair, 1995a) suggest that sandbagging the transducers should be avoided.

The foregoing paragraphs reveal that the suggested transducer mounting methods differ, and are sometimes contradictory. Although burial is recognised as the best method, alternative methods are commonly used. Some of them may not provide sufficient coupling resulting in inaccurate measurements of ground vibration. The objective of this study was, therefore, to evaluate the most common transducer-ground coupling methods and their influence on vibration parameters, the findings of which could be used during the course of investigations.

4.2 Experimental Programme

Fourteen blasts in coal and overburden were monitored at Kamptee OCP. Blastholes of 150 mm diameter, loaded with slurry explosives were initiated with shock tube initiation system. The maximum charge per delay varied from 43 to 56 kg. The distance of the transducers from the blasts varied from 77 to 302 m. Four Instantel seismographs - MiniMate Plus with external tri-axial transducers - were mounted side by side using four different methods to simultaneously record ground vibrations. All the seismographs were new and factory calibrated. The frequency response of the transducers was 2 to 300 Hz. The measurement surfaces in all these experiments consisted of soil. The sod was removed prior to transducer mounting. The transducers were levelled and the arrow marked on the transducers was oriented towards the blast.

Four common methods of transducer mounting that were selected for this study are illustrated in Fig. 4.1. The first transducer was placed on a horizontal surface without any device to hold it (Fig. 4.1a). The second one was also mounted in the same way but a loosely packed sandbag was placed over it so that all sides of the bag were in contact with ground (Fig. 4.1b). The third transducer with attached spikes was firmly pressed into the ground such that the base of the transducer was in direct contact with the ground (Fig.4.1c). The fourth one was completely buried in soil, the soil being firmly compacted around and over the transducer (Fig. 4.1d). All the transducers were connected to the respective recording units.

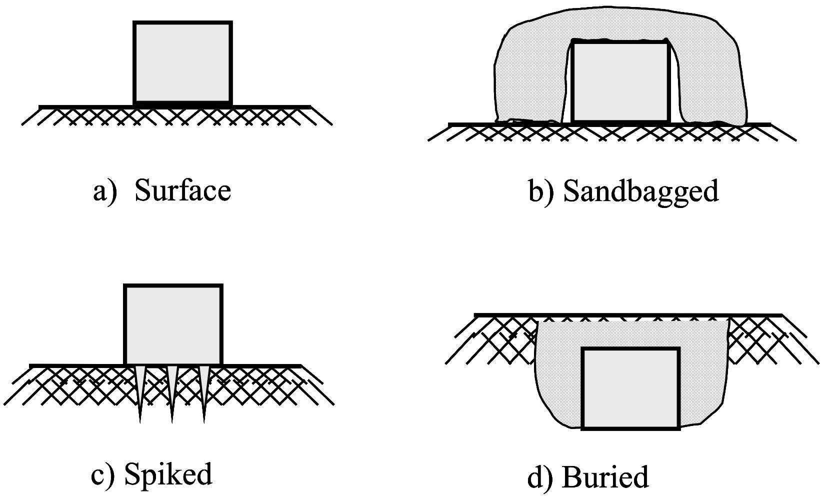


Fig. 4.1 Four common methods of transducer mounting

4.3 Results

A total of 56 events, four per blast, were recorded during the monitoring programme. These events were analysed for particle velocities and frequency contents and the results are given in Table 4.1. Peak particle velocity (PPV) is the maximum velocity in any of the components and peak vector sum (PVS) is the true vector sum of the three components. The Fast Fourier Transform (FFT) method was applied to compute dominant frequencies of the ground motion. The frequency at the greatest spectral amplitude was regarded as dominant frequency and corresponds to the same component of ground motion in which peak particle velocity was the maximum. The particle acceleration was computed by differentiation of the velocity time histories at the rate of 2048 samples per second per channel.

Table 4.1 Ground vibration results for different mounting methods

Blast No.	Strata	MCD (kg)	D (m)	Mounting Method	PPV (mm/s)	PVS (mm/s)	Frequency (Hz)	Peak accel. (g)
1	OB	56	88	Surface	28.80	31.60	4.7	0.48
				Sandbagged	29.80	31.70	4.7	0.42
				Spiked	30.60	31.40	4.7	0.40
				Buried	32.40	32.90	7.7	0.35
2	Coal	50	212	Surface	7.37	8.39	6.2	0.08
				Sandbagged	8.00	8.31	6.0	0.08
				Spiked	7.37	8.35	6.0	0.08
				Buried	7.49	8.54	6.0	0.08
3	OB	56	77	Surface	31.10	33.80	5.7	0.21
				Sandbagged	31.40	34.90	6.2	0.21
				Spiked	30.70	33.70	6.2	0.24
				Buried	31.40	33.80	6.2	0.21

Table 4.1 Ground vibration results for different mounting methods (Contd..)

Blast No.	Strata	MCD (kg)	D (m)	Mounting Method	PPV (mm/s)	PVS (mm/s)	Frequency (Hz)	Peak accel. (g)
4	Coal	50	192	Surface	13.30	14.60	5.0	0.11
				Sandbagged	12.70	15.00	5.2	0.11
				Spiked	12.40	14.70	5.0	0.11
				Buried	13.30	15.00	5.2	0.11
5	Coal	50	272	Surface	6.98	8.69	10.0	0.11
				Sandbagged	6.60	9.21	23.3	0.11
				Spiked	6.48	8.76	9.7	0.11
				Buried	6.48	8.96	9.7	0.11
6	Coal	50	287	Surface	3.05	3.11	5.2	0.05
				Sandbagged	2.67	3.59	7.0	0.05
				Spiked	2.79	3.77	7.0	0.05
				Buried	2.67	3.68	5.2	0.05
7	OB	50	158	Surface	17.80	19.20	6.2	0.21
				Sandbagged	11.40	13.90	6.2	0.19
				Spiked	11.40	13.70	6.2	0.19
				Buried	11.70	14.50	6.7	0.19
8	Coal	50	229	Surface	3.56	4.93	13.2	0.08
				Sandbagged	3.94	4.99	13.0	0.08
				Spiked	3.68	4.53	13.2	0.08
				Buried	3.68	4.74	13.0	0.05
9	OB	56	245	Surface	9.91	9.99	20.0	0.16
				Sandbagged	8.13	8.27	26.5	0.13
				Spiked	7.49	7.89	26.2	0.13
				Buried	7.49	8.94	26.2	0.13
10	Coal	43	276	Surface	6.30	6.67	13.0	0.07
				Sandbagged	6.37	6.81	13.2	0.07
				Spiked	6.37	6.47	20.3	0.08
				Buried	5.94	6.39	13.0	0.08
11	Coal	43	170	Surface	4.95	5.57	9.7	0.08
				Sandbagged	5.08	5.56	16.5	0.08
				Spiked	4.70	5.39	16.5	0.08
				Buried	5.08	5.38	16.5	0.05
12	OB	50	101	Surface	48.40	52.40	8.2	0.48
				Sandbagged	51.40	51.80	8.2	0.53
				Spiked	49.90	53.00	8.5	0.53
				Buried	49.50	52.20	8.2	0.48
13	Coal	50	167	Surface	9.65	11.40	7.0	0.11
				Sandbagged	9.78	11.40	7.0	0.11
				Spiked	10.00	11.20	7.0	0.13
				Buried	9.65	11.40	7.0	0.11
14	Coal	50	302	Surface	4.06	4.49	5.2	0.08
				Sandbagged	4.44	4.56	5.5	0.08
				Spiked	4.19	4.49	5.5	0.05
				Buried	4.57	4.59	5.5	0.08

Note: MCD = Maximum charge per delay; D = Distance, PPV = Peak particle velocity; PVS = Peak vector sum; OB = Overburden; Peak accel. = Peak acceleration.

4.4 Analysis and Discussion

The following assumptions were made for analysis of the monitored data:

1. The elastic properties of the transducer package and its impedance were close to those of the surrounding soil.
2. The influence of the soil condition (moisture, grain size) on the measured vibration parameters (Armstrong and Sen, 1999) was negligible as all the four transducers were mounted side by side.
3. The measured data of the transducer buried in soil were assumed to be accurate and the performance of others was evaluated with respect to it.

As the burial of the transducer was assumed to provide the best coupling, the readings of other three transducers were taken relative to those of the buried one. Relative PPV was defined as the ratio of PPV of a particular transducer to that of the buried transducer. Relative values of PVS and frequencies were also defined in the similar way.

Some of the vibration standards consider PPV while others consider PVS, which are slightly higher than PPV values, to specify the permissible vibration level of ground vibration. Therefore, all the three parameters (PPV, PVS and frequency) have been compared in the following sub-sections.

4.4.1 Comparisons of PPV and PVS

A plot of relative PPV for all the blasts with different mounting methods is shown in Fig. 4.2. Relative value equal to 1.0 represents the data of the buried transducer. Ignoring data scattering up to 10 per cent of the relative ratio (= 1.0) for instrumental and human errors, the data beyond this limit were considered anomalous. The PPV values of the surface transducer are anomalous for three blasts (# 6, 7 and 9).

A similar plot of relative PVS is shown in Fig. 4.3. The surface transducer shows anomalous PVS values also for the same three blasts (# 6, 7 and 9). The reading of the spiked transducer for blast # 9 has marginally crossed the limit but no anomalous values are noted with sandbagging. Anomalous values of PPV or PVS are obviously not representative of the ground vibrations being monitored.

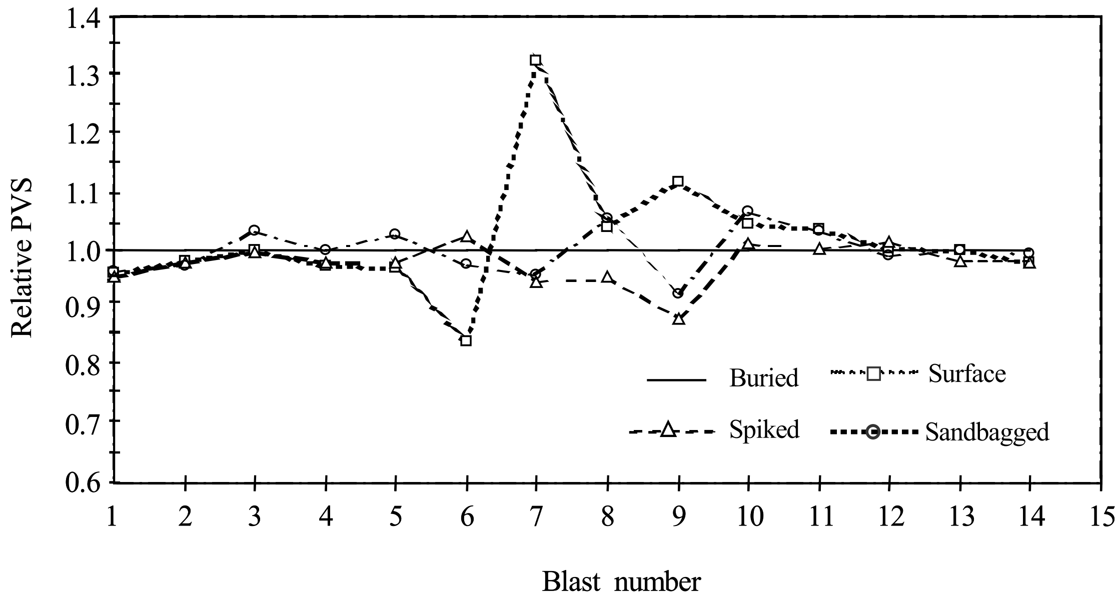


Fig. 4.2 Influence of the transducer mounting methods on peak particle velocity (PPV)

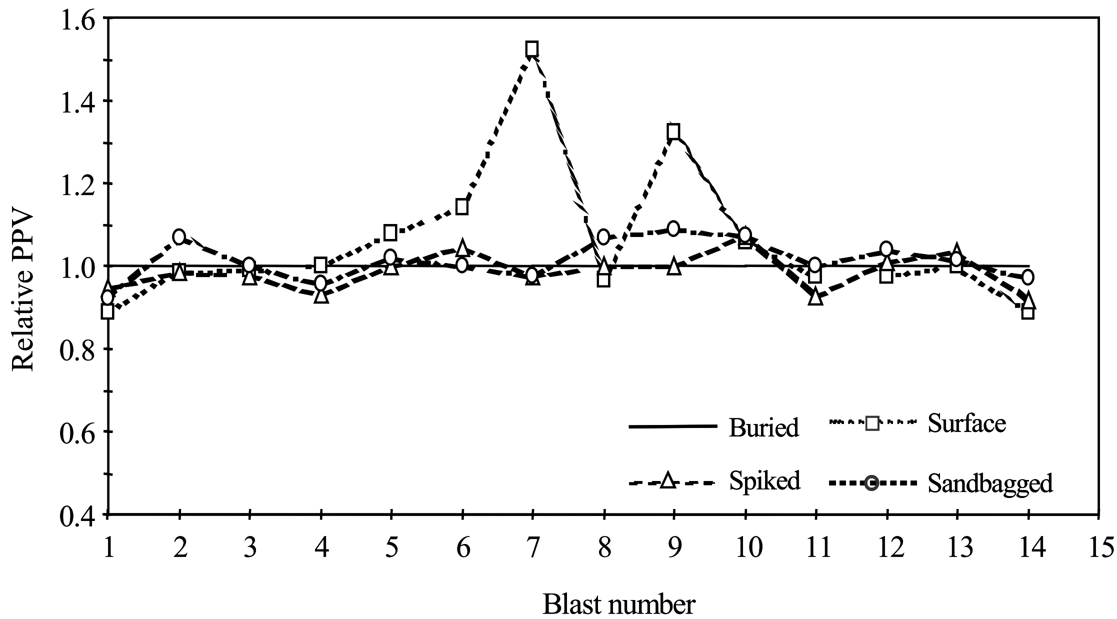


Fig. 4.3 Influence of the transducer mounting methods on peak vector sum (PVS)

From Fig. 4.2 and Fig. 4.3, the surface transducer in three blasts and the spiked one in one blast can be suspected poor coupling. As a decoupled transducer may record higher or lower ground vibration, any mounting method susceptible for decoupling is not reliable for vibration monitoring.

For a given soil condition, decoupling is most likely at higher magnitudes of ground vibration. The coupling problems in two blasts (# 6 and 9) were not expected, because the acceleration was less than 0.2 g. This shows that decoupling may occur even at a small vibration level. Blair (1987) clearly demonstrated exactly the same, i.e. decoupling can occur below 0.2 g. Of course, this contradicts with Dowding (1992) whose suggestions are not supported by any experimental evidence.

4.4.2 Comparison of frequencies

Relative frequencies for all the blasts monitored are shown in Fig. 4.4. The sandbagged transducer for blast #5 shows the largest deviation. The deviations in some other cases are comparatively small but not negligible. As these deviations do not necessarily correspond to those blasts where PPV or PVS were anomalous, the reasons for deviations may not be related to the mounting methods.

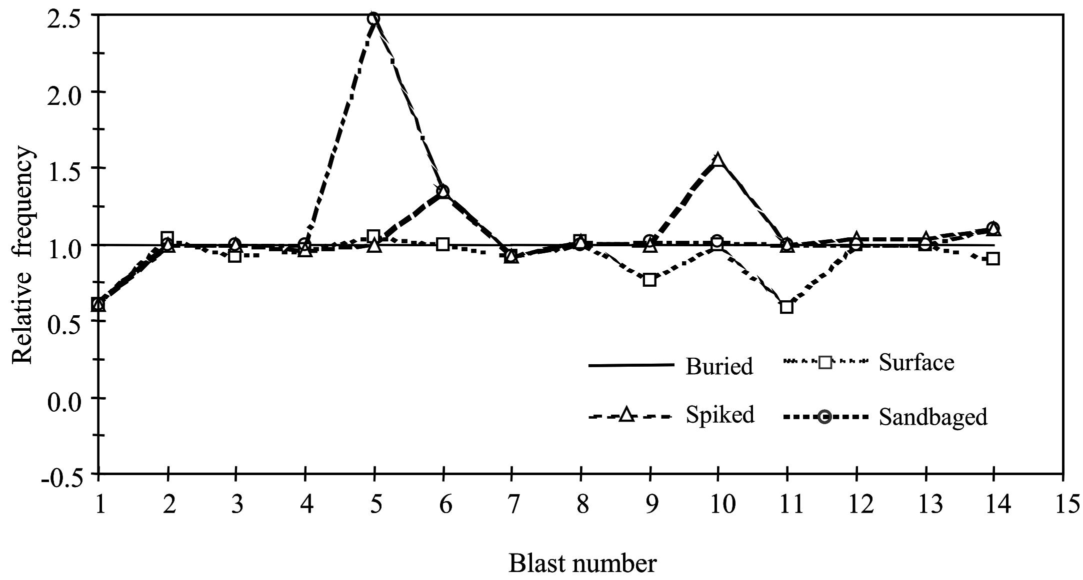


Fig. 4.4 Influence of transducer mounting methods on frequency of ground motions

The FFT analyses of the events recorded by the buried and sandbagged transducers for blast#5 are shown in Fig. 4.5. The dominant frequency for the buried transducer is 9.7 Hz while the secondary frequency is 23.3 Hz. For the sandbagged transducer, the secondary and dominant frequencies are interchanged. Such interchanges in frequencies were noticed when two or more frequencies, each having almost equal spectral amplitude, were present in an event.

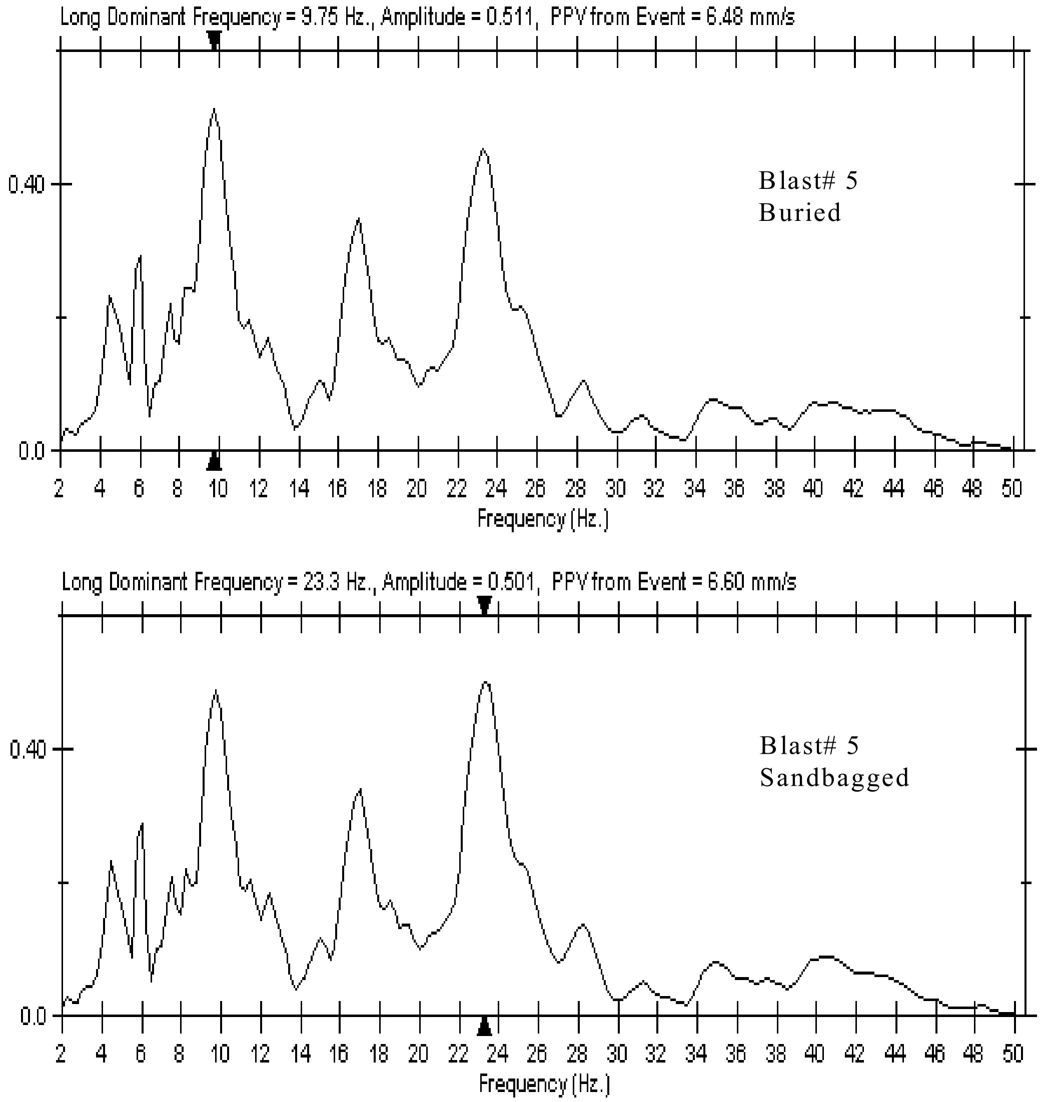


Fig. 4.5 Comparison of frequency spectra - buried and sandbagged transducers (Blast #5).

For blast # 11, the dominant frequency for the buried transducer is 16.5 Hz while the secondary frequency is 9.7 Hz (Fig. 4.6). The corresponding event of the surface transducer shows that these frequencies are again interchanged (Fig. 4.6).

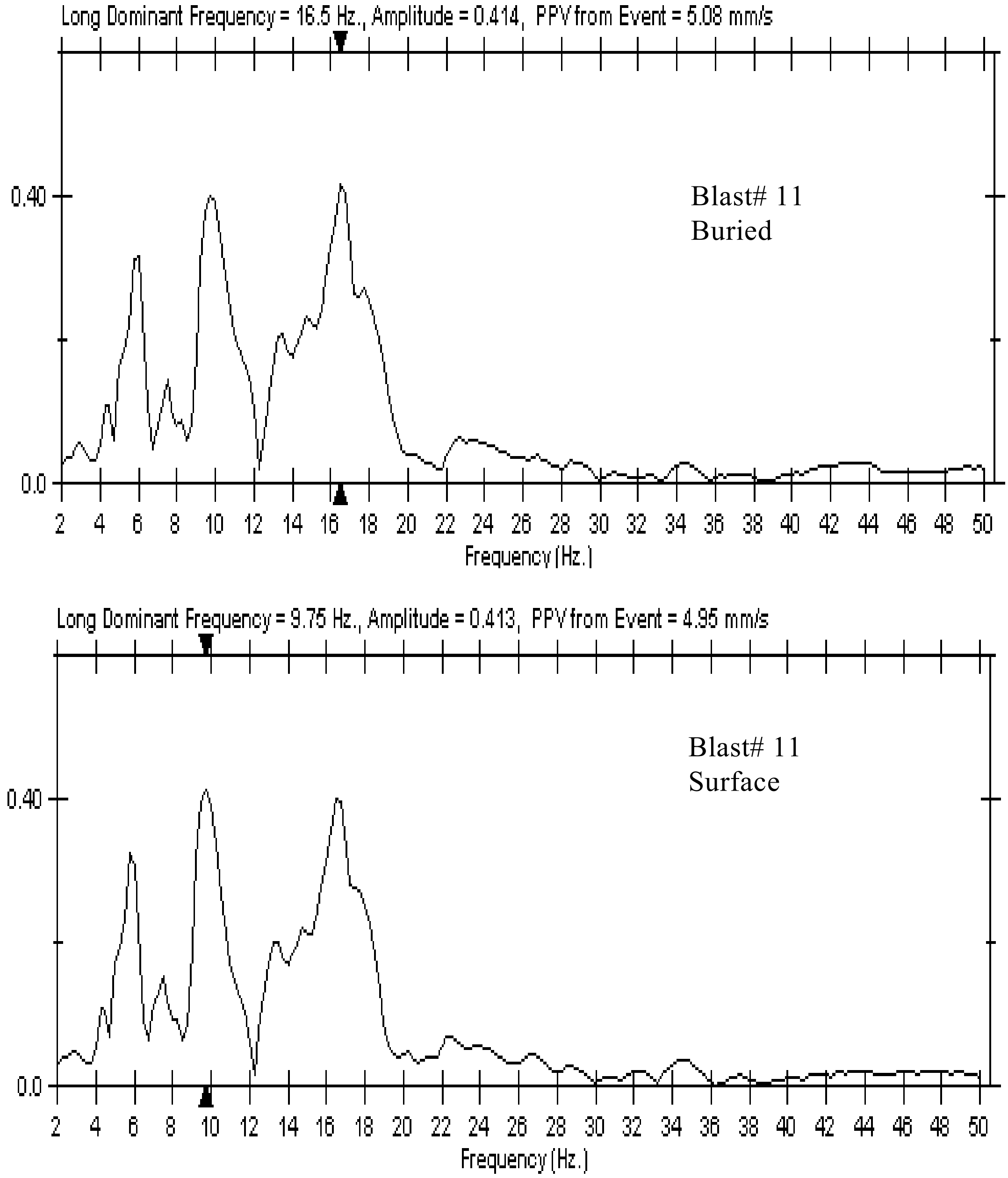


Fig. 4.6 Comparison of frequency spectra - buried and surface transducers (Blast #11).

A similar phenomenon is noticed once again for blast #10 (Fig. 4.7). The dominant frequency for the buried transducer is in fact the same as the secondary frequency for the spiked one. The secondary frequency of the buried transducer also matches with the dominant frequency for the spiked one.

The deviations were negligible when the waveforms were composed of a narrow range frequencies or the largest spectral amplitude was significantly greater than others.

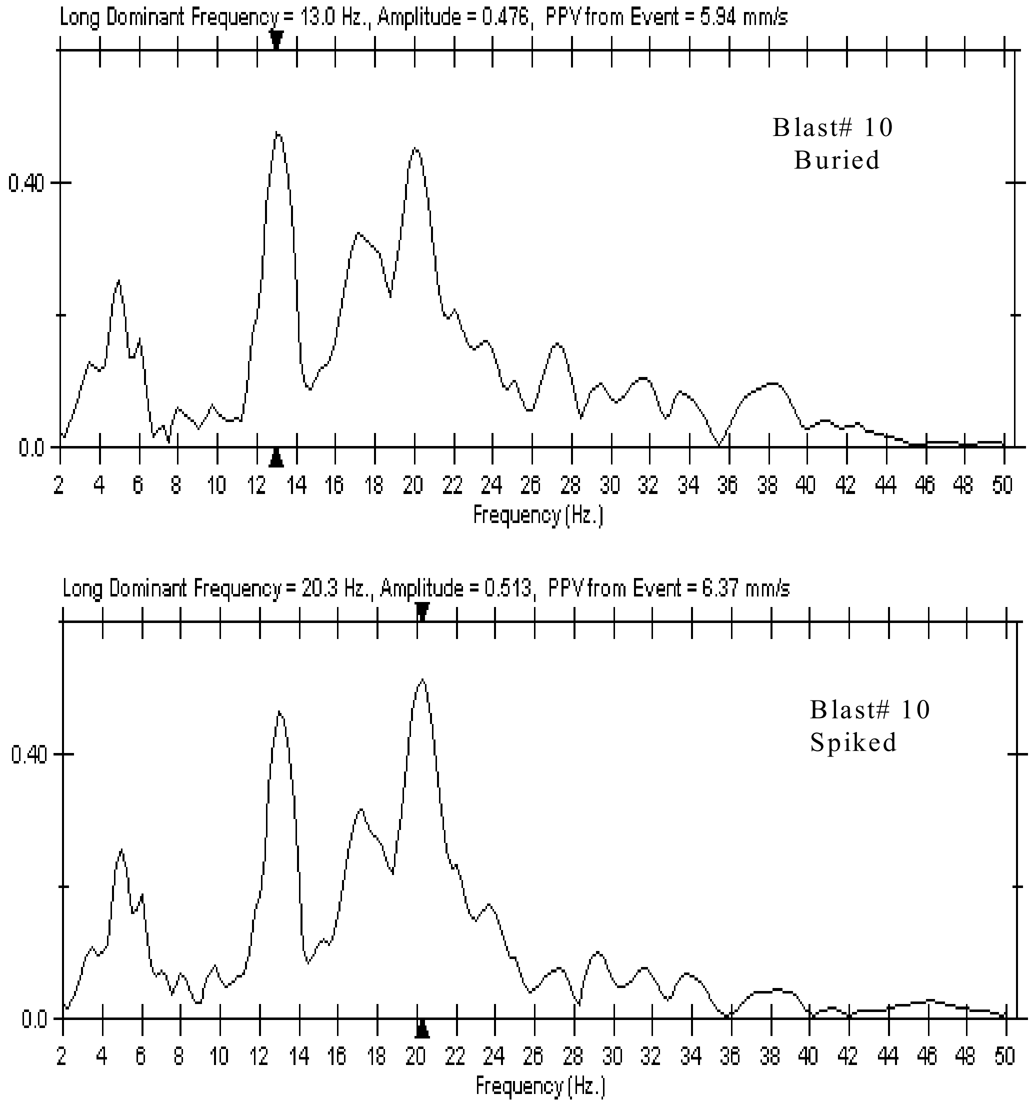


Fig. 4.7 Comparison of frequency spectra - buried and spiked transducers (Blast #10).

Considering that frequency spectra in one or other way should reveal decoupling problems, the presence of a very low frequency in all the events were examined, which according to Wheeler (2004) is a definite indication of decoupling. There was only one case (Blast #6) as shown in Fig. 4.8 where the dominant frequency is only 2 Hz. However, this method did not indicate the decoupling problem in other cases (Blast #7 and 9), which were suspected for decoupling.

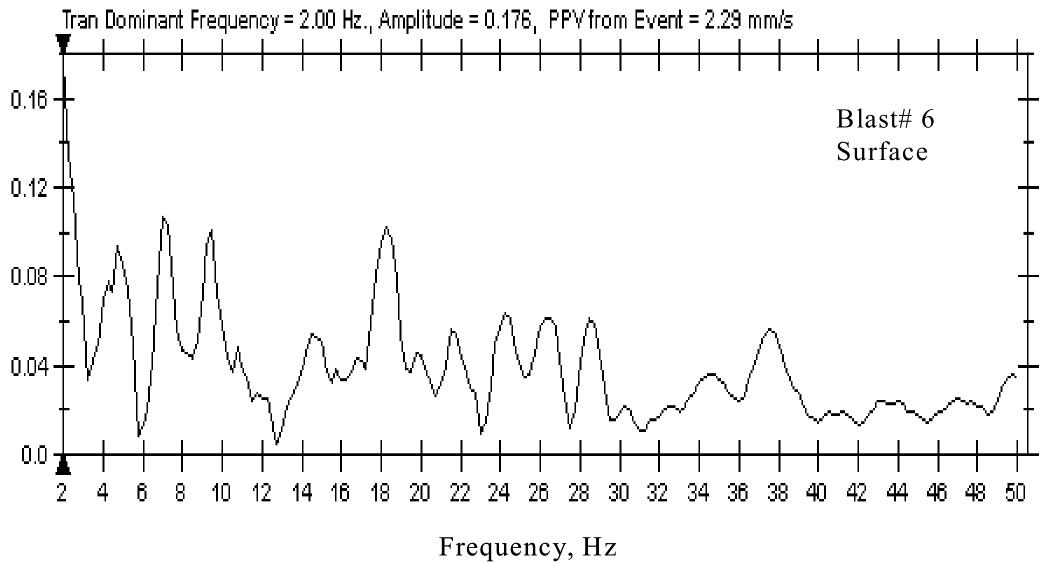
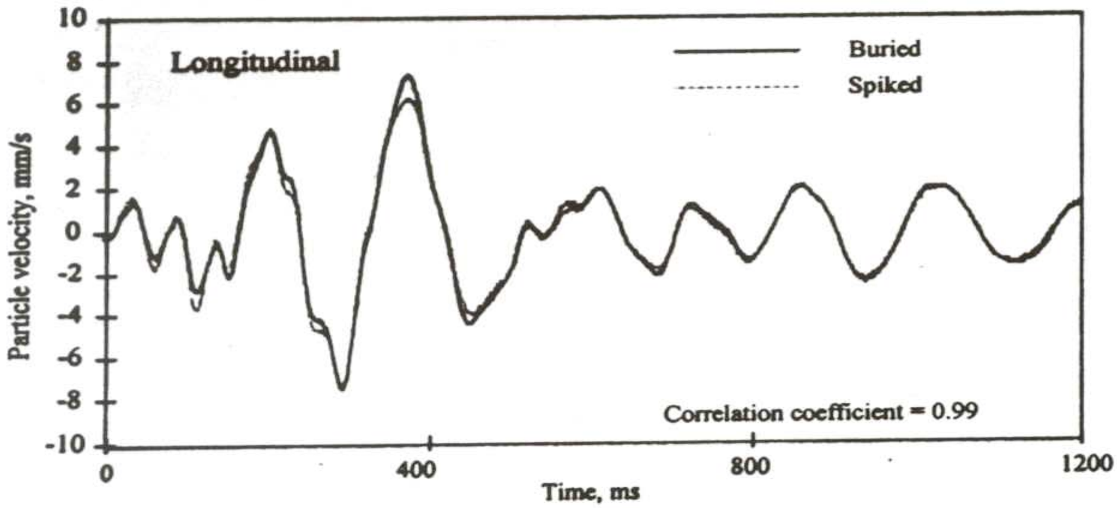


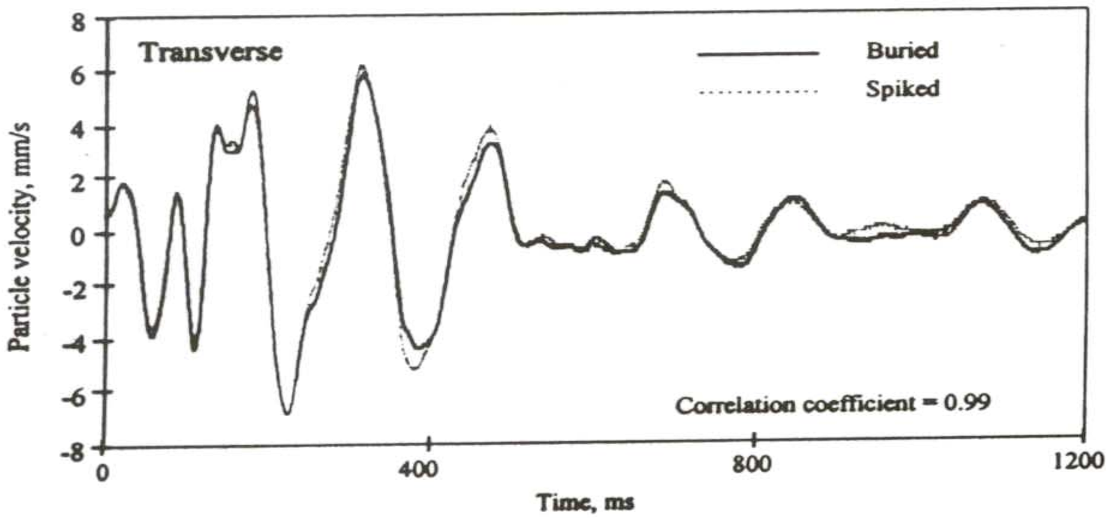
Fig. 4.8 Frequency spectra showing the presence of a very low frequency (Blast #6)

4.4.3 Comparison of waveforms (time histories)

PPV or PVS values pertain to a particular point in the time history of ground vibration. It is possible that PPV or PVS values with two different mounting methods are nearly equal, but their time histories are totally different. In such cases, comparison of PPV or PVS will fail to reveal the decoupling problem. Comparisons of waveforms or trace matching can reveal the differences, if any. Fig. 4.9 shows the time histories of the buried transducer superimposed on the time histories of the spiked transducer for blast #2. The correlation coefficient between two waveforms indicates the degree of their matching. The correlation coefficients between the waveforms of the buried transducer and those of others are given in Table 4.2. If two waveforms were identical, a perfect correlation would be obtained. If the amplitudes differed, the correlation coefficient would decrease. A very low correlation coefficient or a distorted waveform can indicate a possibility of decoupling. However, waveform distortion may also be due to mount resonance.

One of the worst cases of waveform matching was observed in the transverse component of the surface transducer for blast # 6 where clear distortion of the waveform can be noticed (Fig. 4.10). The correlation coefficient between these waveforms is only 0.29 (Table 4.2).





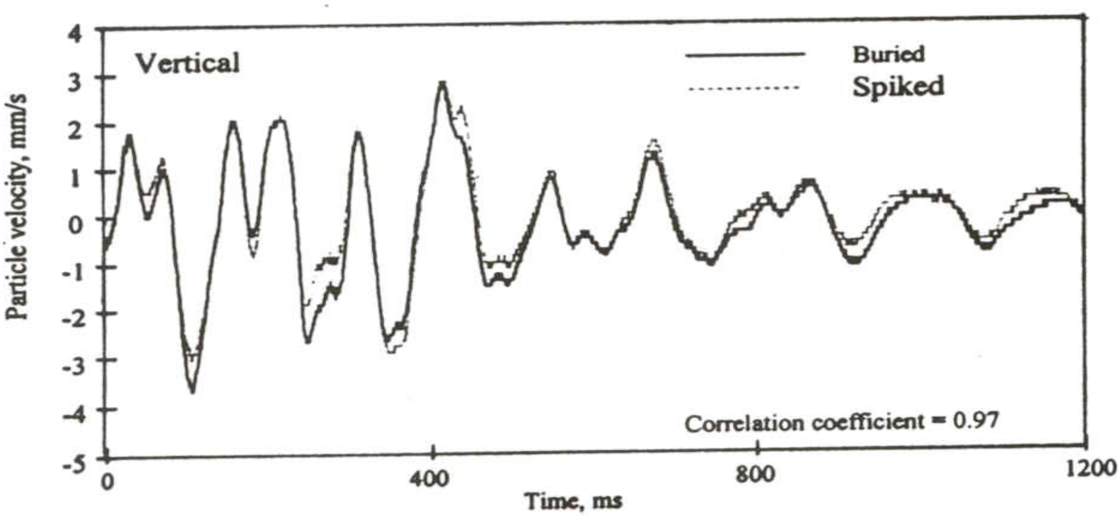


Fig. 4.9 Superimposed waveforms of the buried and spiked transducers (Blast # 2)

Table 4.2 Correlation coefficients between waveforms (buried with other transducers)

Blast Number	Between buried and other transducers	Correlation coefficient		
		Transverse	Vertical	Longitudinal
1	Surface	0.71	0.96	0.86
	Sandbagged	0.82	0.97	0.94
	Spiked	0.87	0.97	0.94
2	Surface	1.00	0.96	1.00
	Sandbagged	0.99	0.99	0.99
	Spiked	0.99	0.97	0.99
3	Surface	1.00	0.99	0.99
	Sandbagged	1.00	0.99	0.99
	Spiked	0.99	0.98	0.97
4	Surface	0.99	0.97	1.00
	Sandbagged	0.98	0.97	0.99
	Spiked	0.99	0.95	0.99
5	Surface	0.98	0.98	0.99
	Sandbagged	0.96	0.96	0.96
	Spiked	0.98	0.97	0.99
6	Surface	0.29	0.95	0.94
	Sandbagged	0.99	0.96	0.98
	Spiked	0.82	0.72	0.80
7	Surface	0.84	0.97	0.82
	Sandbagged	0.99	0.98	0.99
	Spiked	0.97	0.98	0.99
8	Surface	0.97	0.97	0.98
	Sandbagged	0.98	0.98	0.99
	Spiked	0.97	0.97	0.98
9	Surface	0.46	0.58	0.84
	Sandbagged	0.46	0.95	0.96
	Spiked	0.96	0.96	0.99
10	Surface	0.99	0.97	0.99
	Sandbagged	0.99	0.99	0.99
	Spiked	0.79	0.97	0.97
11	Surface	0.94	0.94	0.94
	Sandbagged	0.95	0.97	0.98
	Spiked	0.76	0.81	0.83
12	Surface	0.81	0.91	0.90
	Sandbagged	0.77	0.89	0.95
	Spiked	0.13	0.98	0.98
13	Surface	0.99	0.99	0.99
	Sandbagged	0.97	0.99	0.98
	Spiked	0.96	0.99	0.97
14	Surface	0.85	0.94	0.95
	Sandbagged	0.96	0.95	0.99
	Spiked	0.87	0.95	0.97

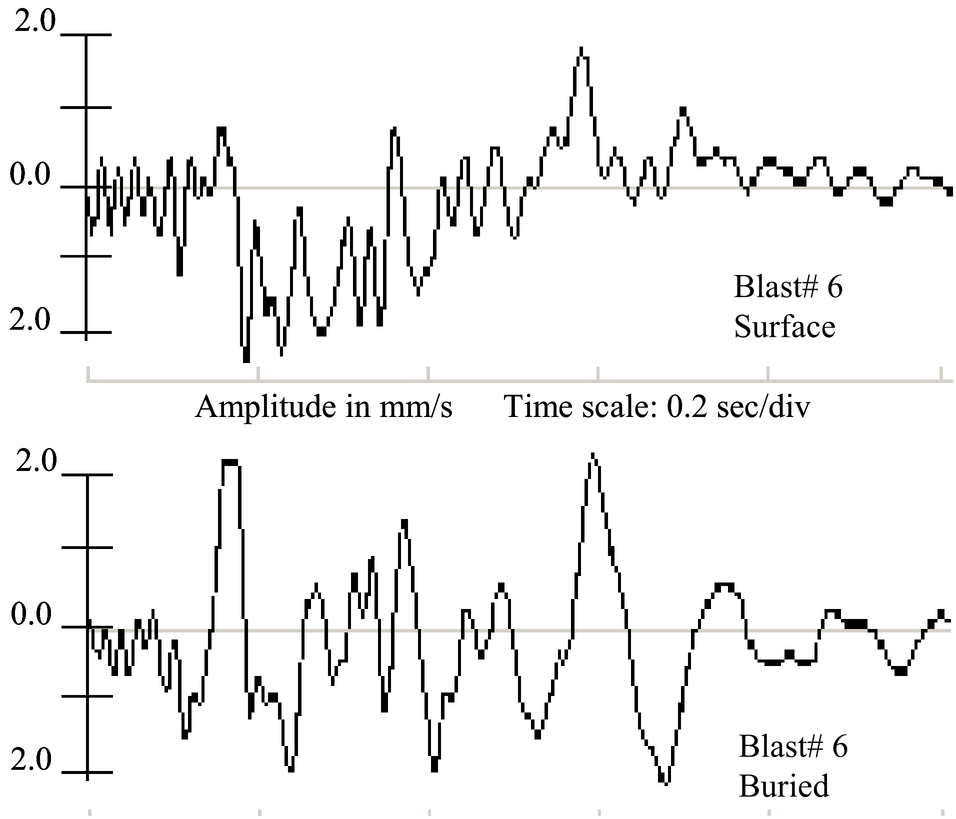


Fig. 4.10 Distorted waveform of the surface transducer compared to the undistorted one of the buried transducer (Blast # 6)

Trace matching, though confirmed poor coupling with the surface transducer for two blasts (Blast # 6 and 9), it did not find problems for blast #7. Instead, it showed problems with the spiked transducer for blast #12 and with the sandbagged transducer for blast #9.

A poorly coupled transducer may cause distortion of waveforms in only one or two components of ground vibration. If the distorted component is different from the one related to PPV, it might not affect PPV or even PVS. This was the reason why decoupling in the sandbagged and spiked transducers for blast # 9 and 12 was not suspected while comparing PPV or PVS values.

The anomalous PPV or PVS that are also corroborated by trace matching are regarded as confirmed decoupling. The deviations in relative PPV or PVS that are not supported by trace matching or vice versa, decoupling can only be suspected. A summary of all the mounting methods with confirmed decoupling or suspected decoupling is given in Table 4.3. Mounting transducers simply on the surface is most susceptible for decoupling, although the spiking and sandbagging are also not reliable methods for transducer mounting.

Table 4.3 Transducers suspected for poor coupling

Blast Number	Anomalous PPV	Anomalous PVS	Poor trace matching	Remarks
6	Surface	Surface	Surface (T)	Confirmed decoupling
7	Surface	Surface	-	Suspected for decoupling
9	Surface	Surface	Surface (T & V)	Confirmed decoupling
9	-	Spiked	-	Suspected for decoupling
9	-	-	Sandbagged (T)	Suspected for decoupling
12	-	-	Spiked (T)	Suspected for decoupling

Note: (T) and (V) in column 4 denote transverse and vertical components of the ground vibration

Chapter 5
STRUCTURE RESPONSE TO GROUND VIBRATION

5.1 Introduction

Seismic waves due to blasting propagate in all directions and may pass through adjacent structures. The structure may respond or shake due to incoming ground vibration. If vibration is strong enough, the structure may be damaged. Vibration standards are intended to preclude damage to structures. The DGMS Circular of 1997 on ground vibration prescribes permissible levels of peak particle velocity depending on excitation frequency which has been categorised into: < 8 Hz, 8-25 Hz and > 25 Hz. The German DIN standard (Appendix) categorises it into 1-10 Hz, 10-50 Hz, and 50-100 Hz whereas the USBM damage criterion (Appendix) identifies it in four different frequency bands (1-4 Hz, 4-15 Hz, 15-40 Hz and > 40 Hz). Thus, the frequency categorisation in these standards is inconsistent and is a subject of further research.

This chapter attempts to summarise the studies on structure response conducted in India and abroad. Further studies have been conducted at two coal mines. The purpose of this chapter is to suggest a method for frequency categorisation based on structure responses.

5.2 Previous Studies on Structure Response

5.2.1 Natural frequency of structures

Natural frequency is the frequency at which the structure freely vibrates after the cessation of the ground vibration. The free vibration resembles a sinusoidal motion with a single frequency. It can be measured directly from time histories of structure vibration (Siskind et al., 1980).

Medearis (1978) determined the natural frequencies of 63 residential structures subjected to micro-vibration testing. He found that the natural frequency of the structure was governed by the height of the structure. The natural frequencies of these structures were found to vary from 4 to 18 Hz.

Siskind et al (1980) also studied the natural frequencies of residential structures subjected to actual blasts rather than impulse loading as in Medearis (1978). It was found that the natural frequencies of these houses varied between 4 and 20 Hz.

For residential structures around Indian mines, Adhikari et al (1989) measured the natural frequencies of residential type of structure, which varied between 8 and 14 Hz. Subsequently, Pal Roy (1998) also measured the natural frequencies of different types of structures which varied from 12 to 14 Hz for single storey brick structures; 8 to 10 Hz for double storey brick structures, and 9 to 16 Hz for concrete structures. The natural frequencies measured in these studies were within 8-16 Hz.

5.2.2 Amplification factor

Superstructure or portion above the ground level of any residential or industrial structure tends to amplify the ground vibration. The amplification will be maximum when the predominant frequency of the ground motion matches the natural or resonant frequency of the structure. The structure at resonant frequency absorbs most of the energy of ground vibration and oscillates with larger amplitude for a longer duration. The proximity of the frequency of ground motion to the natural frequency also creates a favourable condition of resonance and amplification may increase several times. Because of the amplification, structural damage may occur even at a relatively low peak particle velocity.

Amplification (response) factor is the ratio of response of the upper story excitation to the excitation at the base, measured at the ground and time correlated. The US Bureau of Mines studied responses of 46 different structures as a function of frequency of ground vibration (Siskind, 2000). As shown in Fig. 5.1, the higher amplification factors correspond to excitation frequencies within 4 to 12 Hz range.

The US Bureau of Mines (Crum and Pierce, 1995) also carried out computer simulation of structure response using a single degree of freedom (SDOF) model. Fig. 5.2 shows that maximum absolute and differential responses of the system will occur from excitation at or near the natural frequency. Assuming that houses behave similar to simple SDOF systems, the largest potential for cracking in houses would come from excitation at the natural frequency, with less probability from excitations above or below the natural frequency.

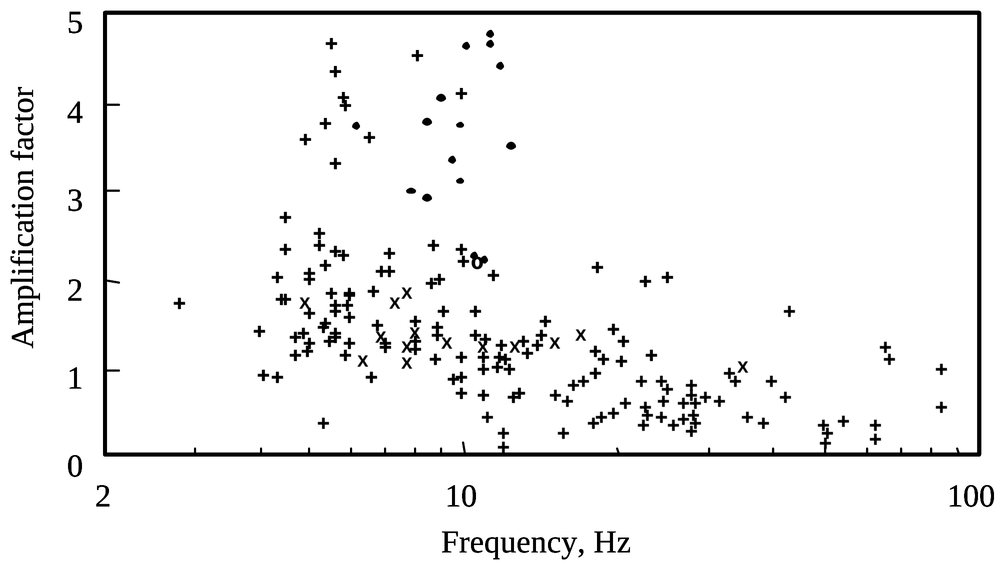


Fig. 5.1 Amplification factor versus frequency of ground vibration (Siskind, 2000)

Taking structure response data for Indian residential structures (Pal Roy, 1998; Singh, 1998; Adhikari et al, 1989), amplification factor was plotted against the frequency of ground vibration. Fig. 5.3 includes data for surface as well as underground blasts, monitored on the surface. It may be noted that the high amplification factors are confined to low frequency (20 Hz), which is the normal range of natural frequencies of structures. At frequencies greater than 50 Hz, structure vibration becomes less than the ground vibration. In subsequent sections, amplification is defined as the ratio of the peak structure vibration to the peak ground vibration. It is not time correlated. Except for a few unusually high amplification factor, all others are within the range reported by the USBM (Fig. 5.1).

5.3 Structure Response Studies at Kamptee OCP

In order to measure the response of the Colliery Manager's old office building (Fig. 3.4), fifty four blasts including those in coal, shale and sandstone were conducted. The bench height was about 7 m and the diameter of hole was 150 mm. Burden and spacing varied depending on the site conditions. The number of holes in a blast varied from 3 to 58. The holes were charged with slurry explosives of two manufacturers. The maximum charge per delay varied from 44 kg to 487 kg. Most of the blasts were initiated by shock tube system (EXEL) using 25 ms surface delays and 200 ms in-hole delays. A few blast were initiated using detonating cord downline system.

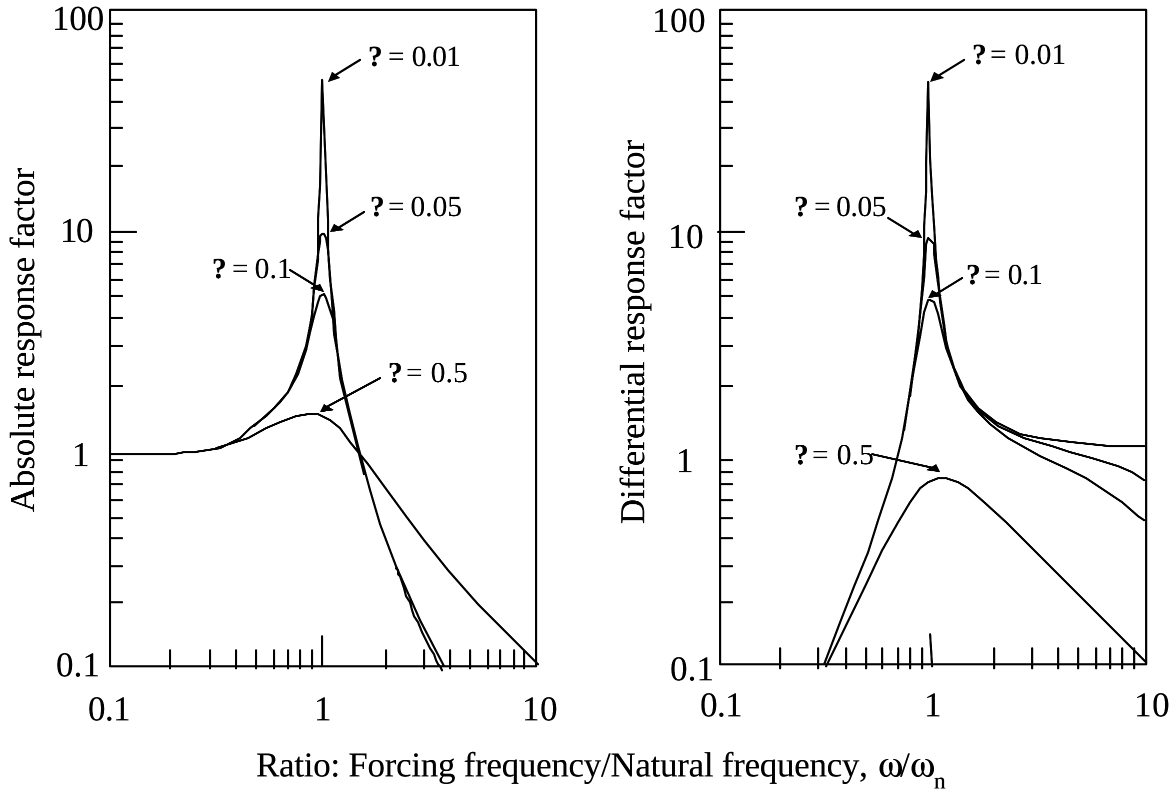


Fig. 5.2 Simulation of structure response using a single degree of freedom model (Crum and Pierce, 1995)

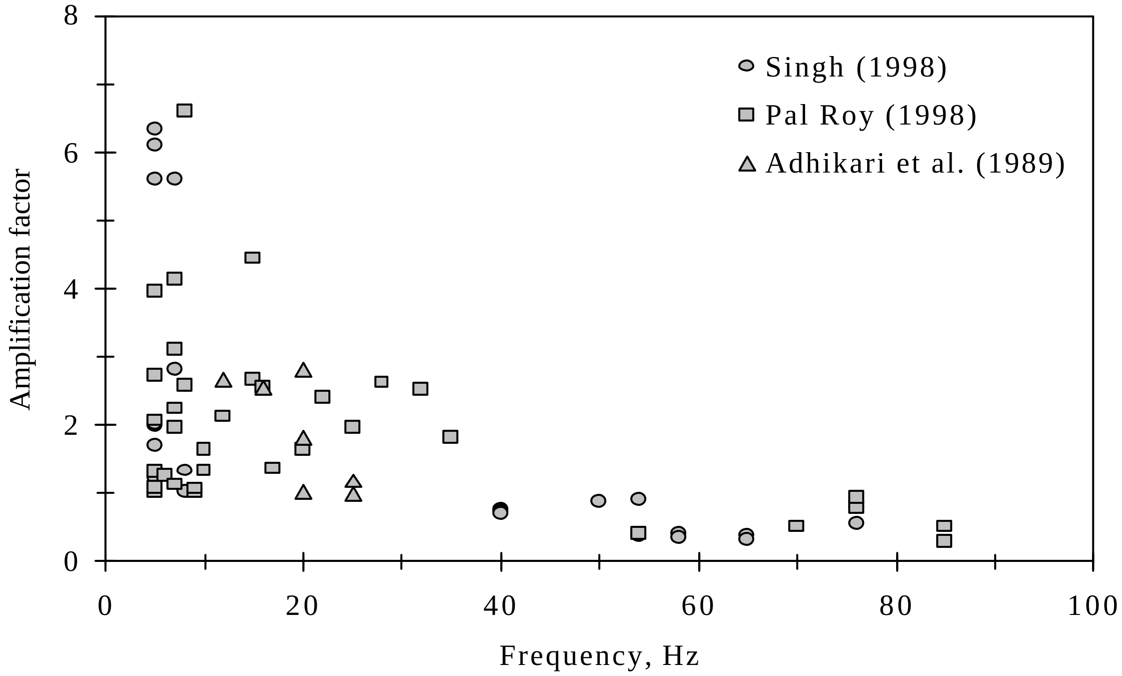


Fig. 5.3 Previous work on amplification factor in India

The measurement of structure response was carried out using two seismographs with external transducers. One of the transducers was mounted in the ground and another transducer was mounted at the corner on the roof of the building.

The recorded events were analysed for peak particle velocity and frequency of individual components, both for ground and structure vibrations. The frequency was determined by Fast Fourier Transform (FFT) method, which transforms the ground motion time histories (time domain) into frequency domain. Natural frequency and amplification factor were also computed and analysed.

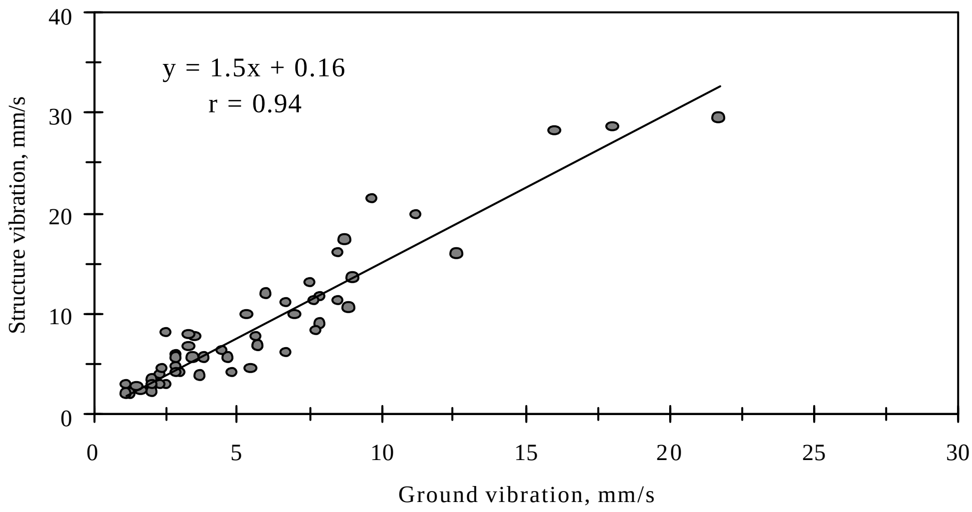
The sinusoidal portion of the time histories of structure response at its later stage was selected and FFT analysis was performed to compute the natural frequency of the structure, which varied between 6 and 12 Hz.

Amplification is an indicator of the observed structure response. Fig. 5.4 shows plots of structure vibration against the ground vibration for different components. It may be noted that structural vibration is directly and linearly proportional to ground vibration. Reducing ground vibration can therefore control the structure response also.

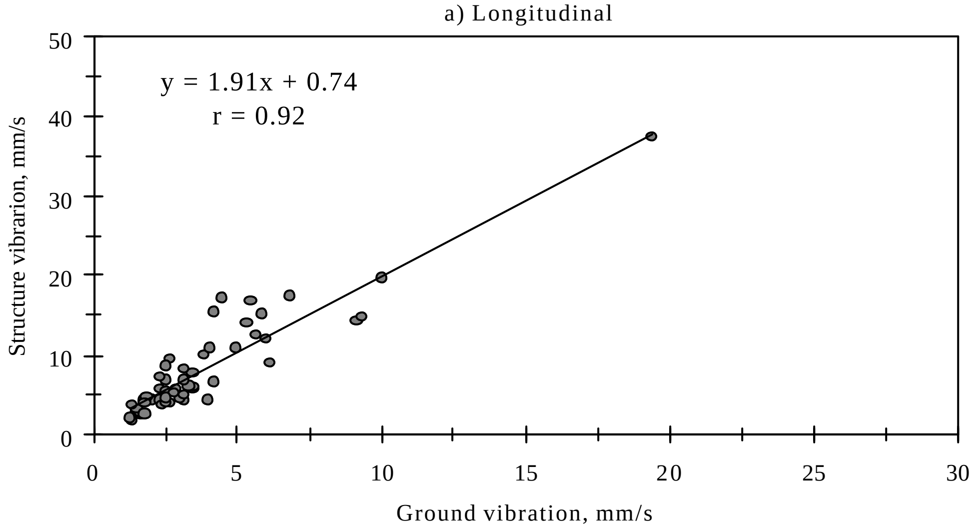
Fig. 5.5 shows that amplification factor varies with frequency of ground vibration at Kamptee OCP. The highest amplification (> 2.5 times) is noted at frequencies between 5 and 20 Hz, which is the range of natural frequencies of structures. The amplification factor at < 8 Hz is same as that at 8-20 Hz. The amplification factor continues to be greater than 1.0 up to 30 Hz, which is the highest frequency observed at this mine.

5.4 Structure Response Studies at OC-2

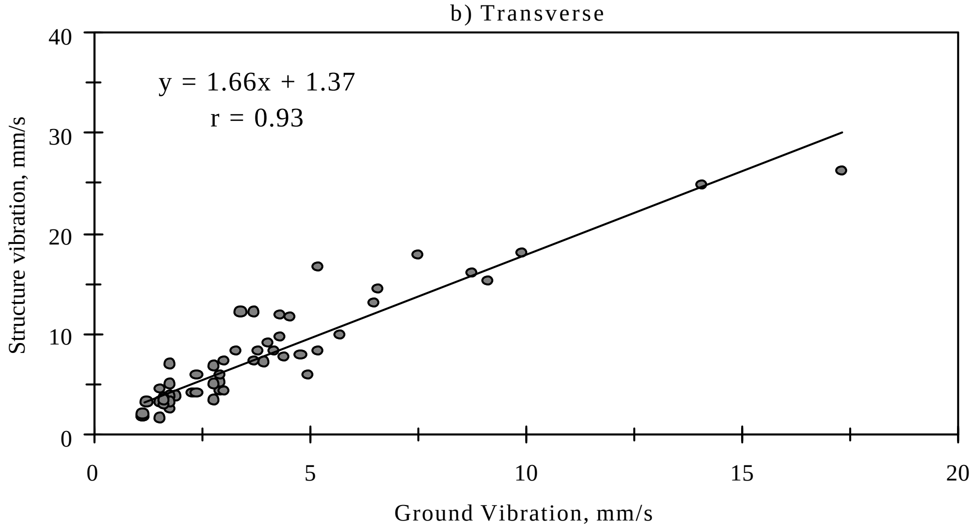
The response of the double storied house (Fig. 3.5), the details of which are given in Chapter 3 was measured for more than 100 blasts. Because of the two different blasthole diameters (150 mm and 250 mm) and varying bench height, blasting patterns varied widely.



a) Longitudinal



b) Transverse



c) Vertical

Fig. 5.4 Structure vibration versus ground vibration for different components at Kamptee OCP

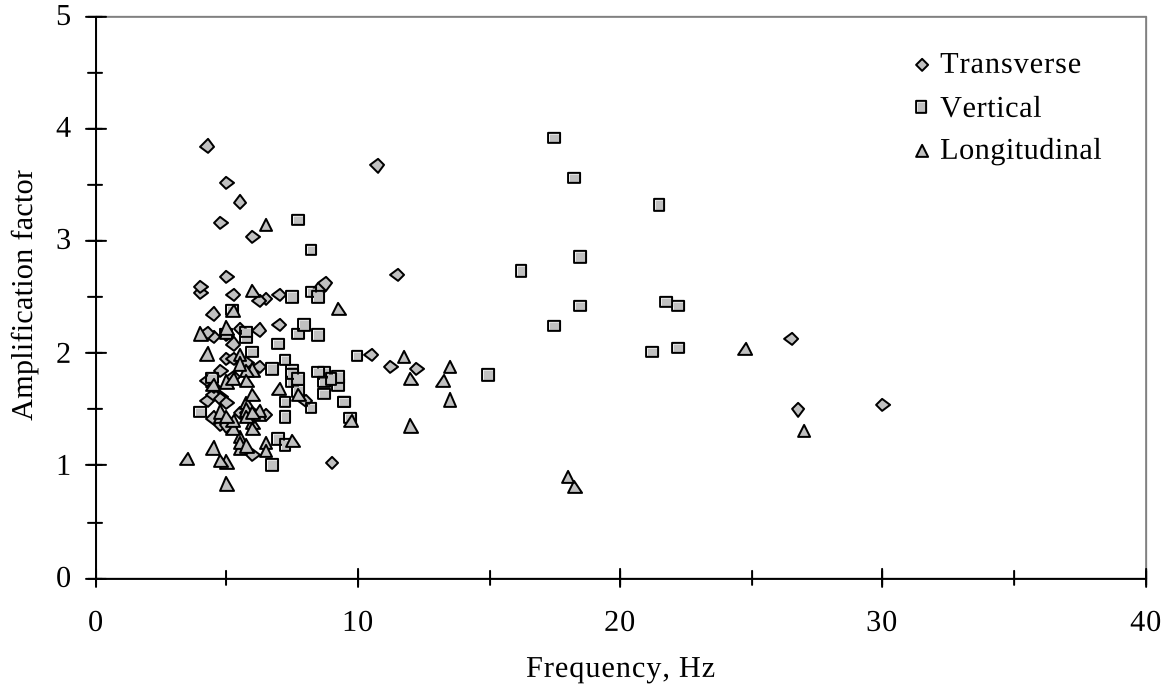


Fig. 5.5 Amplification factor versus frequency of ground vibration at Kamptee OCP, WCL

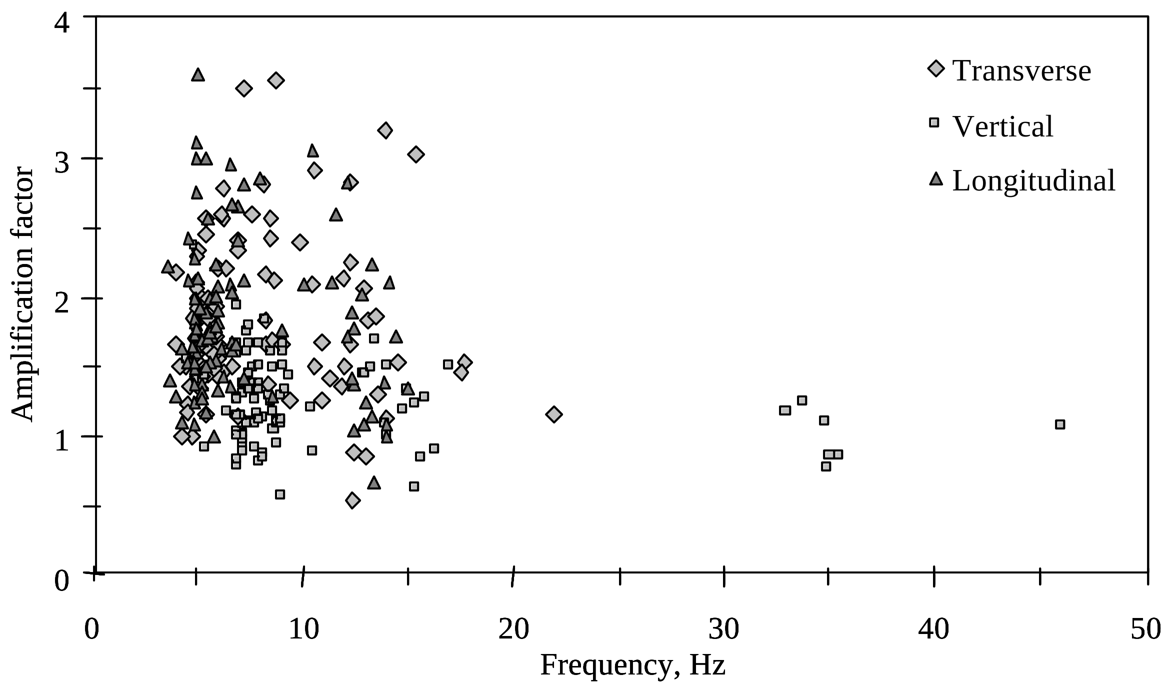


Fig. 5.6 Amplification factor versus frequency of ground vibration at OC-2, SCCL

The analysis of the data for OC-2 was also carried out in the same way as for Kamptee OCP. Fig. 5.6 shows the maximum amplification in the frequency band of 5-15 Hz at which ground vibration can produce increased displacement and strain in the structures and hence the highest risk of damage. The amplification factor at < 8 Hz is same as that at 8-15 Hz; that is, the risk of damage to surface structures at < 8 Hz and at < 20 Hz is almost the same.

5.5 Categorisation of Vibration Frequency

The frequency of ground vibration due to blasting varies from 4 to 100 Hz, rarely up to 250 Hz whereas the lowest frequency of ground vibration that a standard seismograph can record is 2 Hz. The frequency within the natural frequency of residential structures (< 20 Hz) is the most dangerous because it causes amplification of ground vibration up to 5 times. Frequencies below the natural frequencies of structures do not pose additional risk of damage (Siskind, 2002). At frequencies between 20 and 50 Hz, there is still some amplification (up to 2.5 times). At frequencies greater than 50 Hz or so, amplification is negligible. The DGMS standard has considered the influence of frequency of ground vibration while prescribing permissible peak particle velocity. However, amplification factor and the associated risk of damage at different frequency bands escaped their attention.

Based on structure responses, the frequency may be categorised into:

1) Low frequency (< 20 Hz): frequencies within or below the natural frequencies of residential structures, where the amplification factor is greater than 2.5.
2) Medium frequency (20-50 Hz): frequencies above the natural frequencies, where the amplification factor varies between 1.0 and 2.5.
3) High frequency (> 50 Hz): frequencies much higher than the natural frequencies, where amplification factor is less than 1.0.

Chapter 6
ASSESSMENT OF DAMAGE TO SURFACE STRUCTURES

6.1 Introduction

A house without cracks would be unusual. Cracks occur in the walls and ceilings of structures, and there are many causes ranging from poor construction to normal environmental stress. However, when a house is located nearby a mine, there would be frequent complaints about damage due to ground vibration. Because there is high probability of cracks being produced due to strains in structures induced by environmental or human activity, it is important to differentiate those damage, which are not due to blasting activity. The most common method is to monitor ground vibration along with damage assessment by visual inspection immediately before and after each blast. The type of damage may be classified as (Siskind et al, 1980):

- Threshold (cosmetic cracking) – Opening of old cracks, formation of new plaster cracks, and dislodging of loose objects.
- Minor (displaced cracks) – Superficial, not affecting the strength of the structures (e.g. broken windows, loosened or fallen plaster), hairline crack in masonry.
- Major (permanent distortion) – Resulting in serious weakening of the structure (e.g. large cracks or shifting of foundations or bearing walls), major settlement resulting in distortion or weakening of superstructures.

The type of damage depends on peak particle velocity and frequency of ground vibration. Of course, it also depends on the type of structure being excited.

This chapter deals with assessment of damage to various types of surface structure typically found in mining areas. The results are used to arrive at safe levels of ground vibration.

6.2 Damage Assessment Method

For assessment of damage, ground vibrations were monitored adjacent to four existing structures at Kamptee OCP and another three test structures at OC-2. A large number of blasts were monitored over a period of six months at each mine. In collaboration with mine personnel, blasts were planned such that the structures were subjected to ground vibration from a lower to higher level. All structures had cracks from natural causes, including settlement and atmospheric conditions. The length and width of these cracks were marked. Pre- and post blast observations were made for any noticeable change in the existing cracks or for the formation of new ones.

6.3 Damage Assessment at Kamptee OCP

Figures 6.1 to 6.4 show the plots of peak particle velocities against the corresponding frequencies for four existing structures at Kamptee OCP. The DGMS limits for the respective category are also drawn in these figures. No damage was observed in any of the structures at PPV exceeding 20 mm/s at frequencies varying by and large between 5 and 27 Hz. The studies could not be continued till the damage due to field constraints.

6.4 Damage Assessment at OC-2

For OC-2 also, similar plots of peak particle velocity against the frequency were made for single storey structure (Fig. 6.5) and double storey structure (Fig. 6.6). Peak particle velocity in excess of 20 mm/s was too low to cause any damage to these structures over a frequency range of 4 to 40 Hz. Unfortunately, the structures could not be subjected to higher vibrations due to field constraints. The mud structure was very much affected by the weather itself. Numerous cracks appeared on the walls after a good sunshine and disappeared after a rain. Progressive cracking was observed on the walls just below the beam due to static loading. Under these circumstances, observation of damage to the mud structure did not serve the purpose.

6.5 Possibility of Increasing Permissible Limits

The measured peak particle velocities at the test structures were lower than the threshold value of damage. Nevertheless, this study established that peak particle velocity up to 20 mm/s is absolutely safe over a frequency range of 5 to 30 Hz. Since human perception of ground vibration begins at a very low level (< 1.0 mm/s), structure damage rather than human perception should be the criteria for any ground vibration standard. On this ground, the permissible PPV of 5 mm/s at low frequencies may be increased by the DGMS at least to 10 mm/s. This is also substantiated by the following facts:

1) Prior to the DGMS Circular of 1997, peak particle velocity of 12.5 mm/s was widely used in India and there were no reported cases of actual damage to surface structures even at low frequency of ground vibration.

2) Permissible level of ground vibration at low frequency is 12.5 mm/s as per U. S. Bureau of Mines (Siskind et al, 1980) and 10 mm/s as per Australian standard (AS2187-1993).

3) Environmental changes and human activities produce strains equivalent of 12-15 mm/s and even higher in some cases (Dowding, 1992; Siskind, 2000). There is no logic to limit the ground vibration below the level caused by the environmental changes.

4) A research study in China (Yuan et al, 2002) recommended that the low-rise residential houses are safe for a vibration level of 20 mm/s at frequencies below 15 Hz.

5) Measurable and observable damage to internal plasterboard cladding occurred when peak particle velocity exceeded 70 mm/s at a frequency of 18 Hz (Moore et al, 2003).

The DGMS regulation was formulated at a time when the safety of surface structures due to blasting was increasingly important but limited technical information was available. Over the last eight years, the situation has changed. Two options are now available with the DGMS. They can either retain the present regulation, which is absolutely safe but severely restricts the blasting operation in mines, or revise the present regulation permitting higher levels that are still safe. Whichever option is followed, it will have enormous consequences on surface mining in future. It is the second option that is the need of the mining industry.

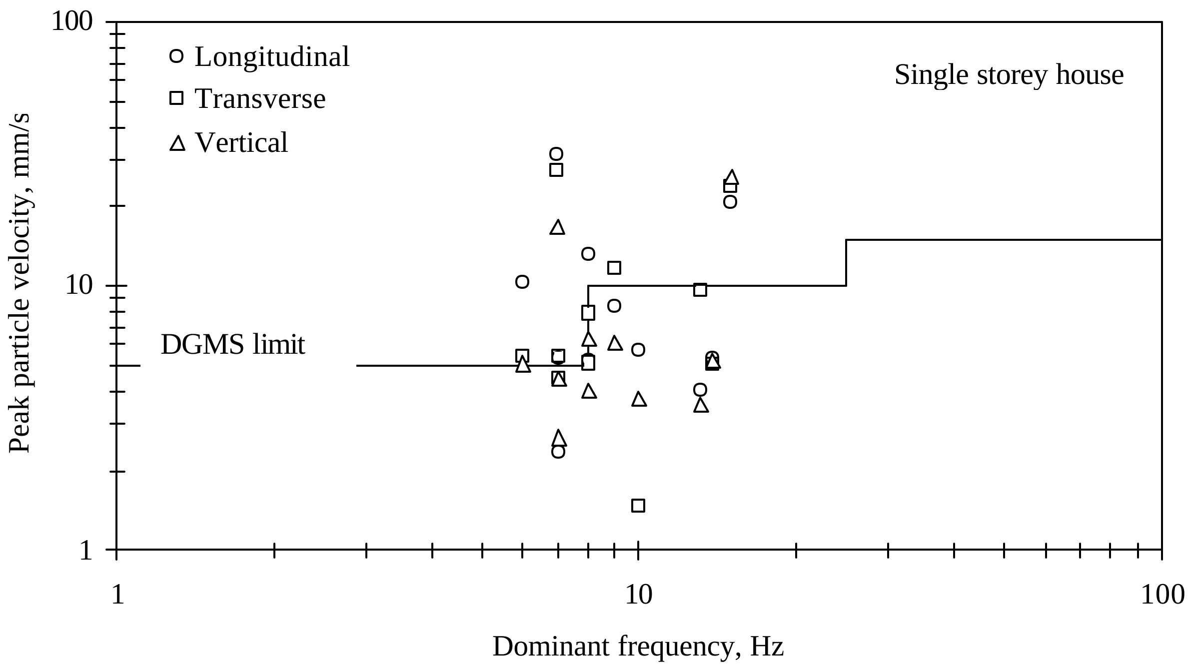


Fig. 6.1 Measured ground vibrations vis-a-vis DGMS limits for first structure at Kamptee OCP

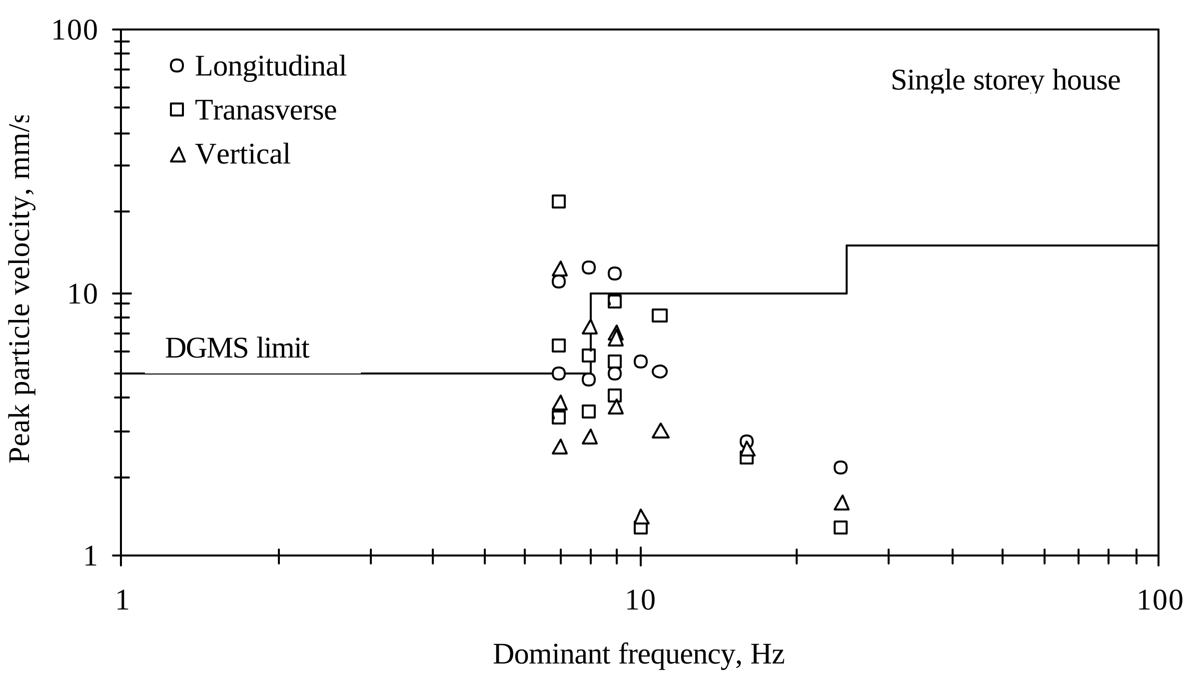


Fig. 6.2 Measured ground vibrations vis-a-vis DGMS limits for second structure at Kamptee OCP

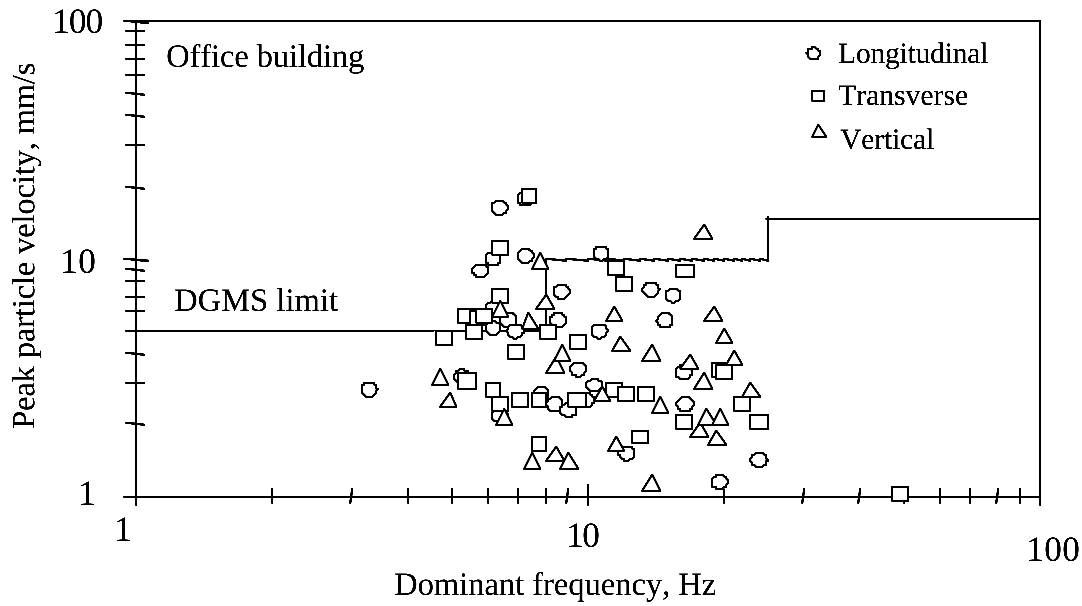


Fig. 6.3 Measured ground vibrations vis-a-vis DGMS limits for third structure at Kamptee OCP

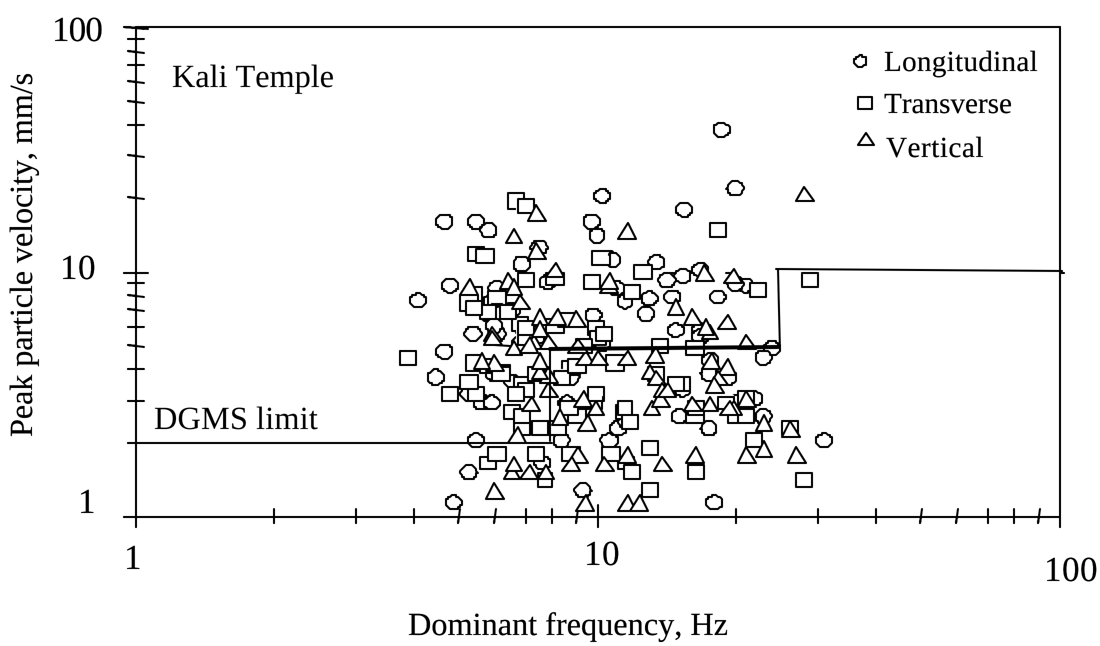


Fig. 6.4 Measured ground vibrations vis-a-vis DGMS limits for fourth structure at Kamptee OCP

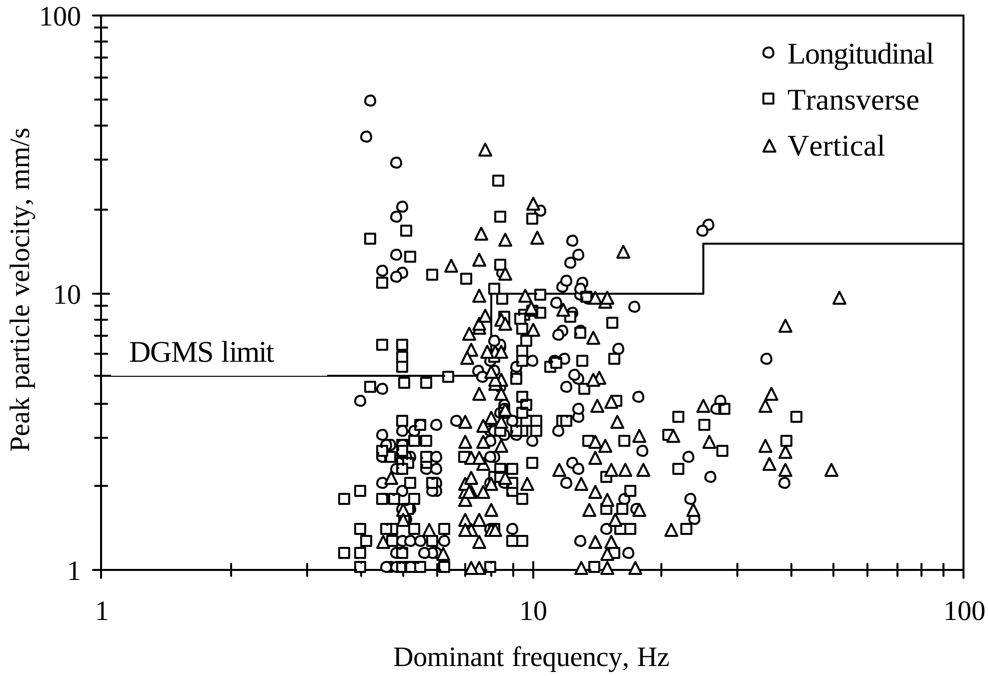


Fig. 6.5 Measured ground vibrations vis-a-vis the DGMS limits for single storey structure at OC-2

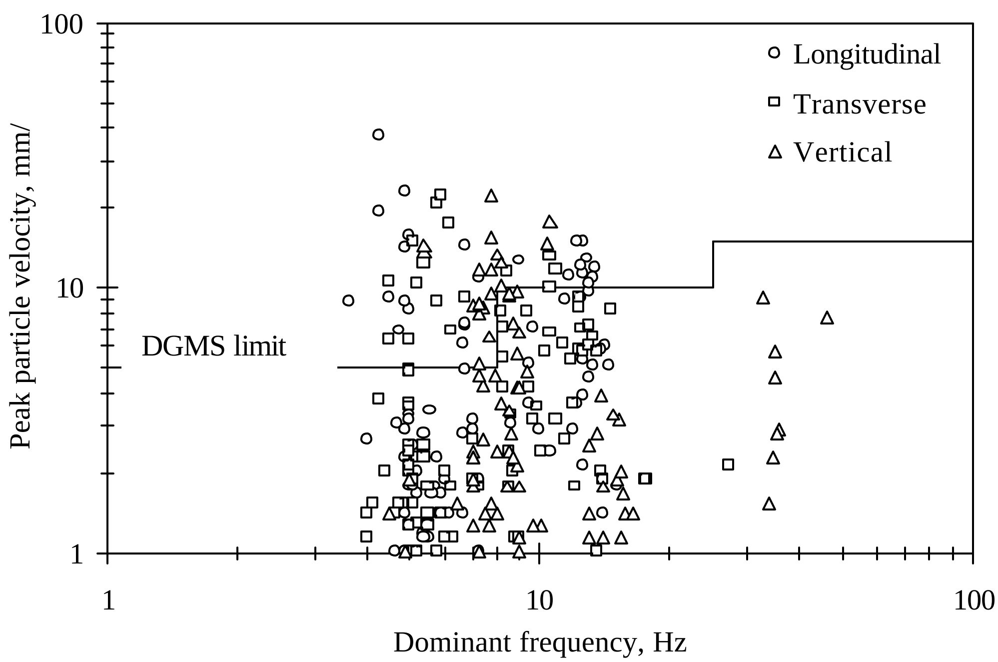


Fig. 6.6 Measured ground vibrations vis-a-vis the DGMS limits for double storey structure at OC-2

Chapter 7

INFLUENCE OF BLAST DESIGN PARAMETERS ON GROUND VIBRATION

7.1 Introduction

The ground vibration measured at a location is influenced by a number of parameters. Some of them like blast geometry, charging patterns, initiation sequence, explosive characteristics and delay timing are controllable while others like rock properties are uncontrollable. The degree to which each of these parameters has influence on ground vibration is to be established so that the most significant parameters can be suitably modified to control ground vibration.

A large number of blasts were monitored at two opencast mines and the data generated were analysed to estimate peak particle velocity (PPV) and frequency on ground vibration. An attempt is made to study the influence of blast design parameters or blasting conditions on ground vibration.

7.2 Studies at Kamptee OCP

7.2.1 Blast design parameters

Blastholes of 150 mm diameter were drilled both in coal and sandstone benches at Kamptee OCP up to a depth of 7.0 m. Burden and spacing were maintained at 4.0 m and 5.0 m respectively. Cartridged slurry explosives of different companies were used. The diameter and weight of the cartridge was 125 mm and 6.25 kg respectively. Normally each hole was charged with 50 kg of explosives having column to primer ratio of 3:1. Drill cuttings were used as stemming material and stemming length varied from 2.5 to 3.0 m. Most of the blasts were fired with shock tube initiation system using in-hole delays of 200 or 250 ms, surface delays of 25 ms within the row and 65 ms between the rows. A few blasts were fired with detonating cord downline with surface connectors of 65 or 67 ms between the rows.

As there was little scope to vary hole diameter, burden, spacing, etc, some other parameters like the explosives used, availability of free faces and total charge weight in a round were varied for this study.

7.2.2 Ground vibration from normal blasts

Using the vibration data for all blasts conducted at Kamptee OCP, peak particle velocity was plotted against the scaled distance and the derived empirical equation with the correlation coefficient is shown in Fig. 7.1. Similar plots were made and empirical equations were derived separately for both sandstone and coal. However, the correlation coefficients did not improve much. Probably, the transmitting media had an overriding influence on PPV, as this mine was developed earlier by the board and pillar method of mining.

The site-specific predictor equation can be used to estimate PPV at the mine. Alternatively, when the permissible PPV and the distance between the blast and the structure are known, maximum charge per delay can be calculated by substituting these values in the predictor equation.

Frequency for regular blasts, as shown in Fig. 7.2, is mostly confined to 5-20 Hz. It is very unfavourable because it would cause resonance in the structures.

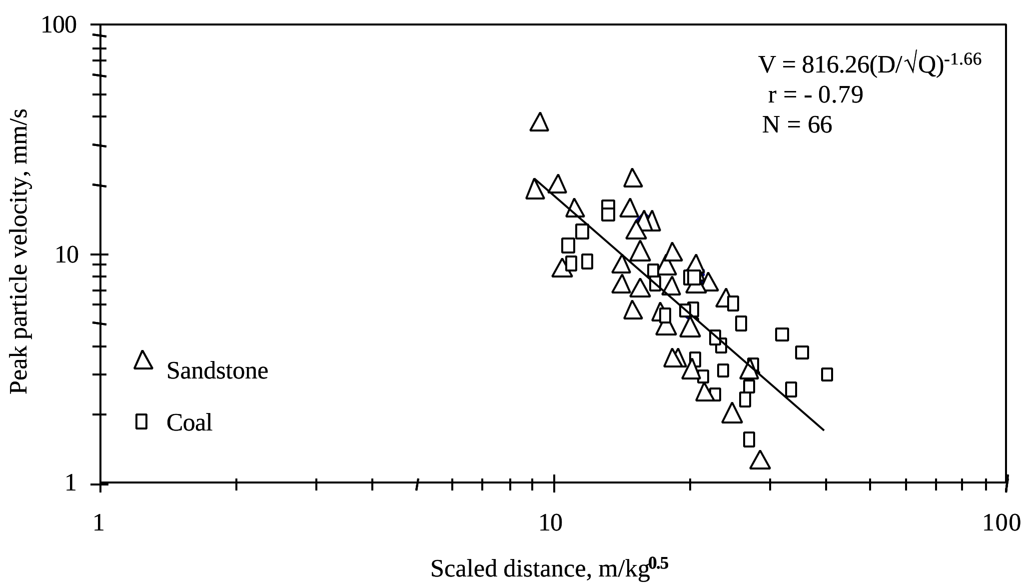


Fig. 7.1 Peak particle velocity versus scaled distance for Kamptee OCP

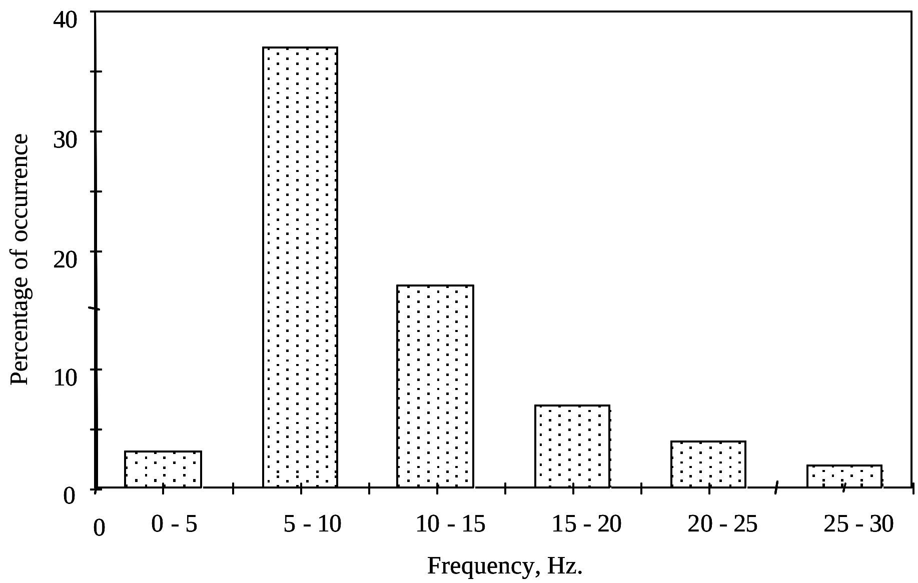


Fig. 7.2 Frequency of ground vibration at Kamptee OCP

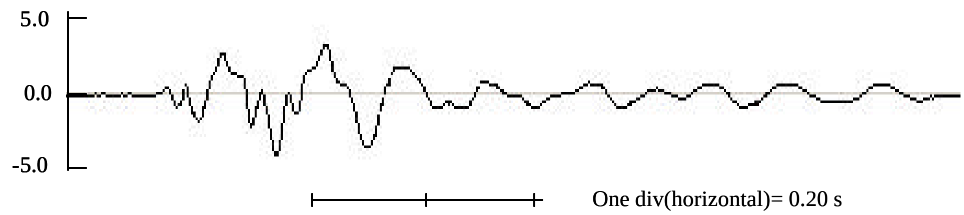
7.2.3 Ground vibration from single hole blast

A single hole blast with drilling and charging parameters identical to those of overburden blasts was conducted in the top sandstone bench. The ground vibrations were recorded at two locations. Fig. 7.3 shows the waveform of the peak component (longitudinal) recorded at a distance of 146 m and its frequency spectrum. Table 7.1 shows the peak values and the associated frequencies of ground vibration. The frequency of single hole blast varies from 5 to 18 Hz, and is similar to that of the normal blasts. It appears that frequency is mostly controlled by the local geology.

Table 7.1 Measured ground vibration from the single hole blast

No.	Distance (m)	Peak value (mm/s)			Frequency (Hz)		
		Transverse	Vertical	Longitudinal	Transverse	Vertical	Longitudinal
1	146	1.65	3.43	4.19	5-11	7-9	5-7
2	188	1.27	2.29	3.68	6-11	6-18	5-11

Hole diameter =150 mm, hole depth = 6 m, burden = 4 m, Charge per hole = 50 kg



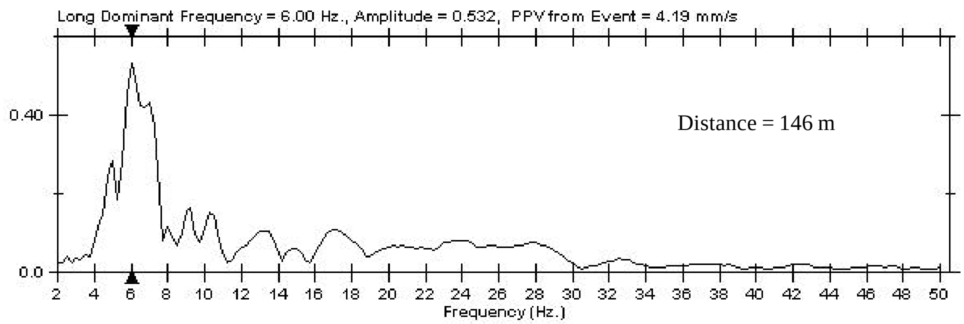


Fig. 7.3 Single hole waveform and its frequency analysis for Kamptee OCP, WCL

7.2.4 Influence of delay interval on PPV

The single hole blast waveform recorded at the mine was used to simulate the influence of delay interval between the two charges using the principle of linear superposition of waves (Hinzen, 1988; Anderson et al, 1985). It was assumed that the waveforms from the single hole blast were reproducible and were determined primarily by the geological characteristics of the path between the blast and the monitoring location.

As the amplitude of the seed waveform dies down after 60 ms, the simulation was done by varying the delay interval between 0 and 60 ms. Fig. 7.4 shows the effect of delay timing on PPV at Kamptee OCP due to constructive or destructive interference. PPV is the lowest when the delay interval is around 30 ms at a distance of 146 m and around 40 ms at a distance of 188 m. The simulated delay, which is longer than the mine's practice, might reduce ground vibration at the mine.

Since the delay timing that gives the lowest PPV is different for different distances, the waveforms should be recorded at a distance of concern for effective control of PPV.

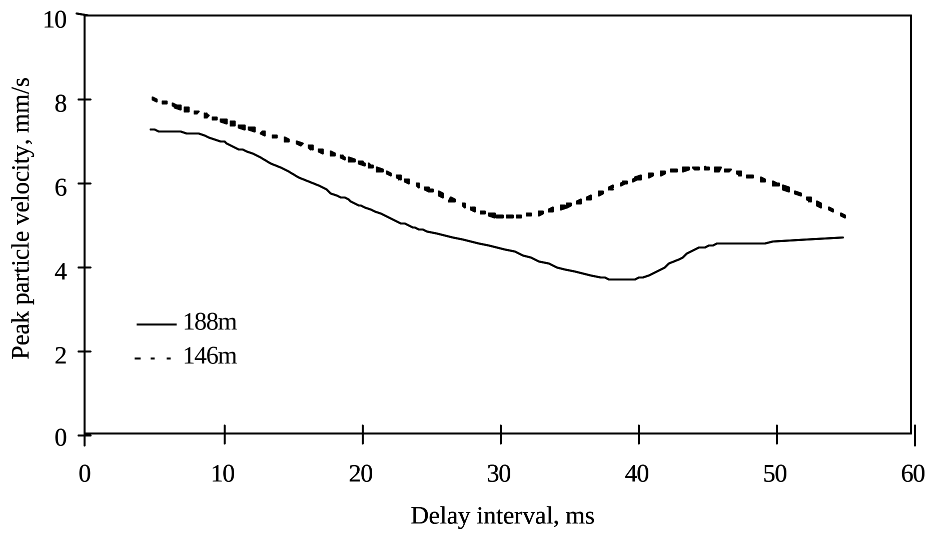


Fig. 7.4 Influence of delay interval on peak particle velocity at Kamptee OCP, WCL

7.2.5 Influence of explosives on PPV

It has been found that the type of explosives has significant influence on ground vibration. Hossaini and Sen (2004) have found that ANFO generates lesser vibration than slurry explosives. Among the different blends tested, Hunter et al (1993) found that the explosive with lower density and lower detonation velocity produced lower level of ground vibration. As the shock energy component of an explosive gives rise to unwanted vibrations (Harries and Gribble, 1993), explosives having larger portion of gaseous energy should be preferred.

At Kamptee OCP, only one type of explosive was used. That was cartridged slurry explosive but there were three suppliers. The explosives supplied by different suppliers were designated as Explosive-1, Explosive-2 and Explosive-3. The vibration data with different explosives were sorted out. Fig. 7.5 shows the peak particle velocity against scaled distance with different explosives at Kamptee OCP.

It is observed that there is a significant difference in ground vibration produced by these explosives. Among the three, Explosive-2 produced the least ground vibration. However, the comparisons are valid within the range of the data generated.

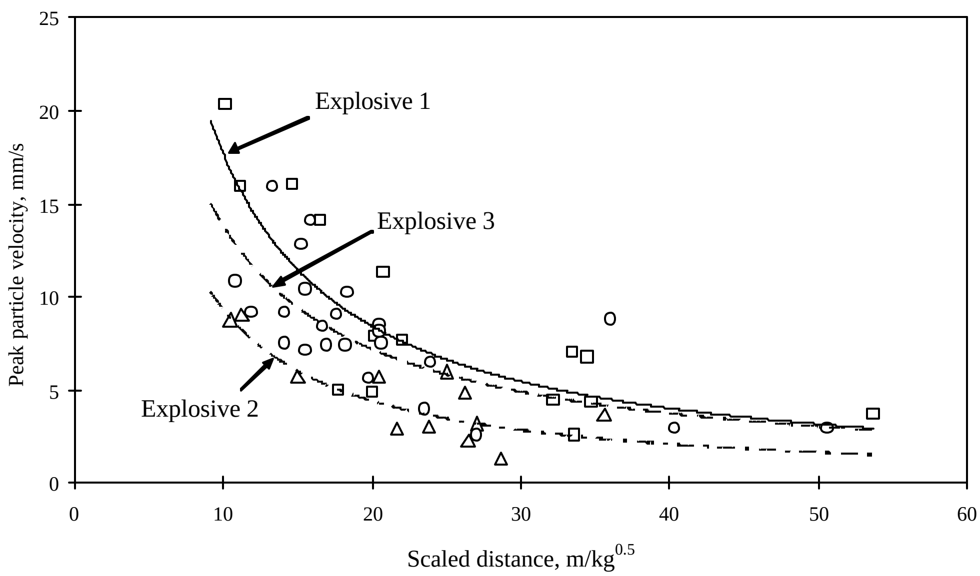


Fig. 7.5 Influence of explosives on ground vibration at Kamptee OCP, WCL

7.2.6 Influence of free faces on PPV

It is known from the crater theory that, if a charge is deeply buried with no free face nearby, the rock is not adequately broken and most of the energy goes into the generation of seismic waves. When it is buried at shallow depth, the same charge may break the rock properly while producing lower ground vibration. In case of bench blasting which normally has one or more free faces, vibration should decrease as the number of free face increases.

At Kamptee OCP, the number of free faces (excluding the top surface) normally varied from between one and two but a few blasts had no free faces. After grouping the data, regression analysis was carried out separately for each confinement condition. Fig. 7.6 shows that the PPV is indeed higher when there is no free face and it decreases as the number of free faces increases. Although there is no clear segregation of the data, the trend is apparent. Ground vibration can therefore be reduced by proper development of benches with free faces. In a multi-row blast, proper delay sequence and delay timing must be ensured to create successive (internal) free faces.

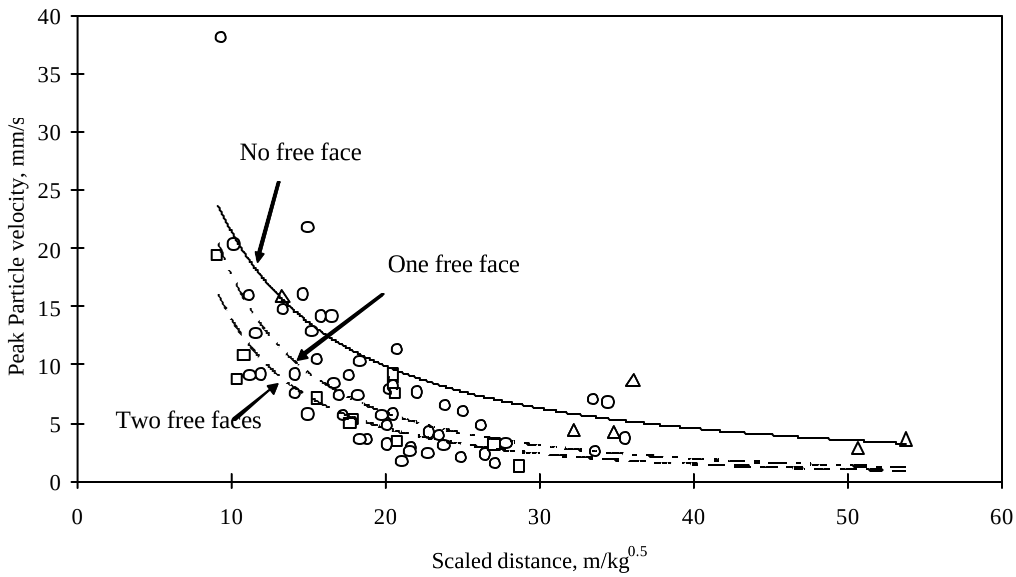
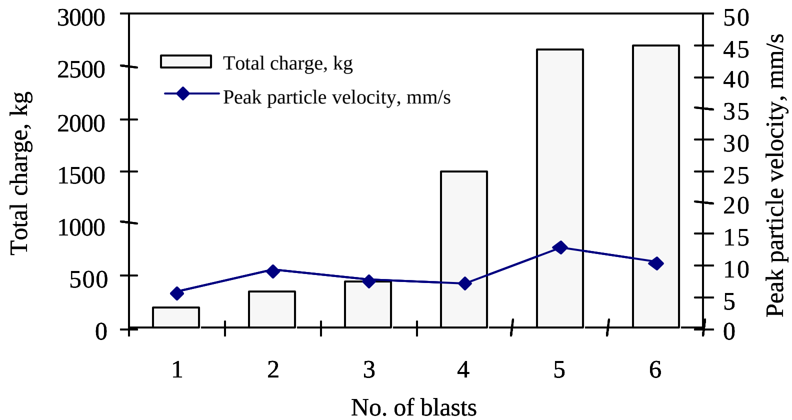


Fig. 7.6 Influence of free faces on peak particle velocity at Kamptee OCP, WCL

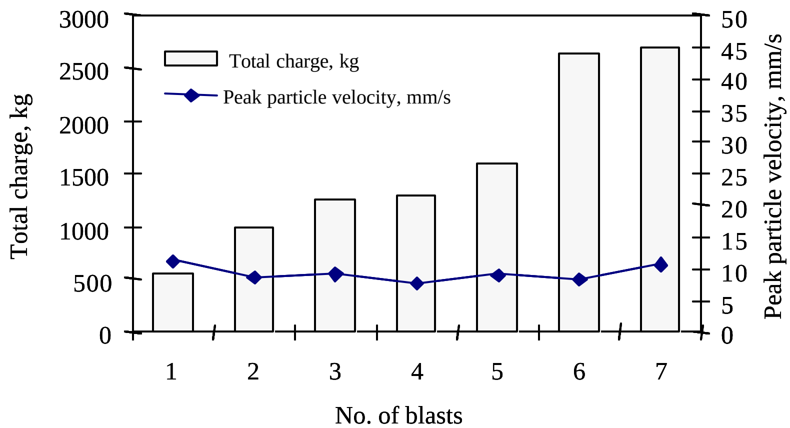
7.2.7 Influence of total charge on PPV

It is generally established that the total charge in a blast has insignificant influence on ground vibration if the delay interval is sufficient to avoid constructive interference between the waves generated by the different group of blast holes (Jimeno et al, 1995). However, Singh (1998) has reported that the total charge in a round affects the ground vibration at distances close to the blasts and its effect diminishes quickly with distance.

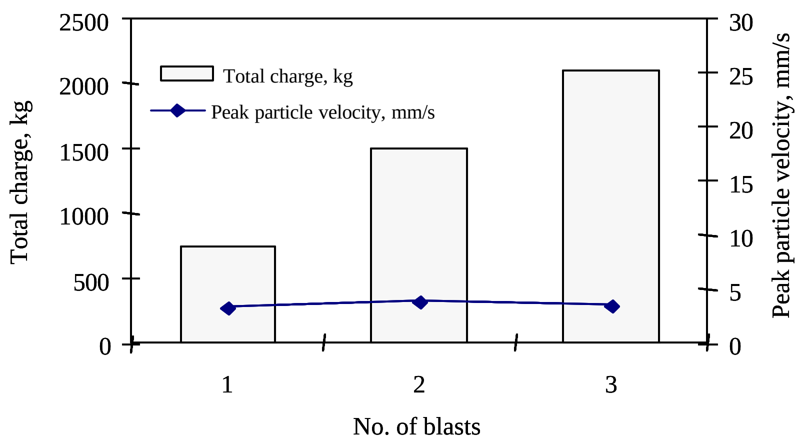
Fig. 7.7 shows PPV monitored at 100-110 m, 145-155 m and 250-260 m distances and the corresponding total charge for a number of blasts having the maximum charge per delay of 50 kg. Even though there is some variation in PPV at closer distances, it is negligible at far off distances. Since structures of concern are located beyond 250 m at Kamptee OCP, restricting total charge, which has been the normal practice in the mine, cannot reduce PPV.



(a) At a distance of 100 – 110 m



(b) At a distance of 145 – 155 m



(c) At a distance of 250 – 260 m

Fig. 7.7 Influence of total charge on ground vibration with the same maximum charge per delay of 50 kg at different distances

7.3 Studies at OC-2

7.3.1 Blast design parameters

Ground vibrations due to blasting at OC-2 were monitored at different distances for several blasts. Blast design and other relevant parameters such as hole diameter, hole depth, burden, spacing, number of holes, number of free faces, initiation systems, total charge, maximum charge per delay, and distance from the blast to the transducers were recorded. The mine used either 150 mm or 250 mm hole diameter, and blasts were initiated either with shock tube initiation system or with conventional system. The resulting vibration parameters such as peak particle velocity, peak vector sum and frequency were also recorded. The influence of explosives or rock masses cannot be established because blasts were conducted in sandstone benches of the same formation with site mixed emulsions. Burden, spacing etc varied so widely that it was not possible to study the influence of these parameters on the ground vibration by simple or multiple regression analysis.

7.3.2 Ground vibration from normal blasts

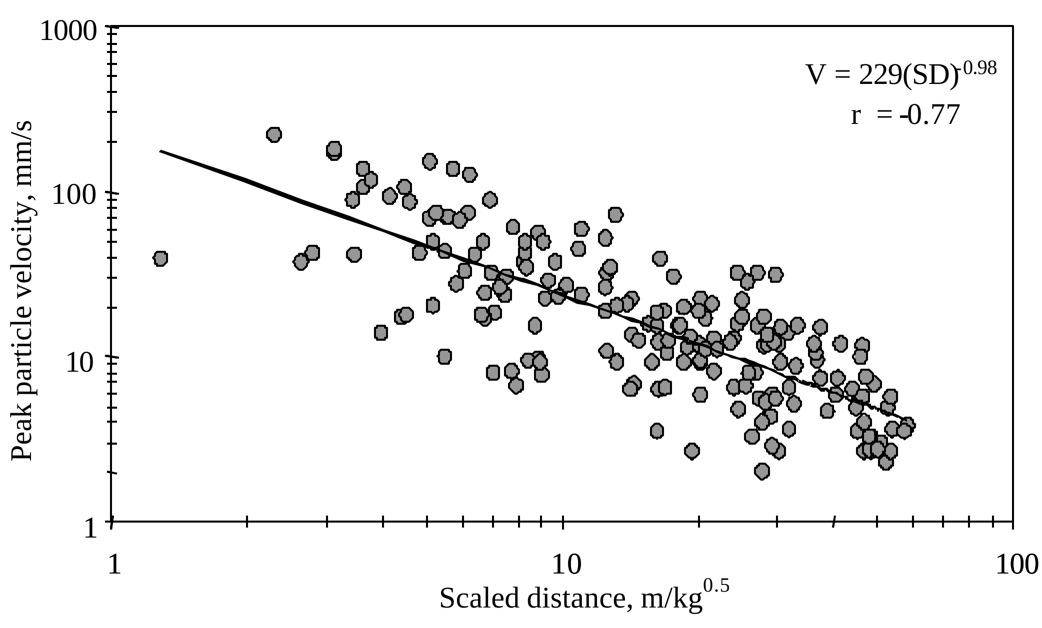


Fig. 7.8 Peak particle velocity versus scaled distance for OC-2, SCCL

Fig. 7.8 shows the peak particle velocity versus the scaled distance plot for OC-2. It also shows the empirical equation and the correlation coefficient between these parameters. Given the maximum charge per delay and the distance of concern, peak particle velocity can be predicted. Alternatively, when the permissible PPV and the distance between the blast and the structure concerned are known, maximum charge per delay can be calculated by substituting these values in this equation.

The frequency of ground vibration, as shown in Fig. 7.9, is mostly confined to the range of 5 – 25 Hz.

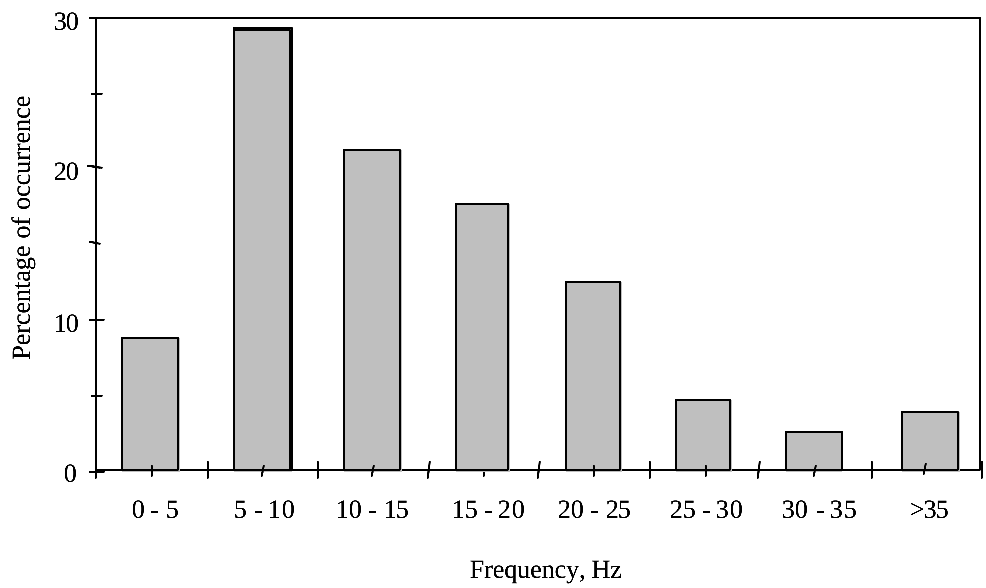


Fig. 7.9 Frequency of ground vibration at OC-2, SCCL

7.3.3 Ground vibration from single hole blast

A single hole blast was conducted in one of the lower benches in hard sandstone. Considering the normal practices in the mine, the blast was conducted with hole diameter of 250 mm, hole depth of 9 m, burden of 6 m and charge per hole of 120 kg. The measured vibration parameters are given in Table 7.2. The frequency, which varies from 10 to 26 Hz, is again similar to that of the normal blasts. The ground itself in all probability acted as a big filter that attenuated higher frequencies, allowing only lower ones. Otherwise, there would have been a significant presence of high frequencies in the single hole records at close distances. In the absence of high frequencies in single hole records, any attempt to control frequency by changing delay interval (Anderson et al, 1982) simply does not work.

Table 7.2 Measured ground vibration from the single hole blasts at OC-2, SCCL

No. of readings	Distance (m)	Peak value (mm/s)			Frequency (Hz)		
		Trans	Vertical	Long	Trans	Vertical	Long.
1	69	14.7	21.30	11.0	15-24	16-24	10-13
2	120	4.19	5.46	3.30	15-23	10-26	13-18

Note: Trans., & Long. = Transverse and longitudinal components of ground vibration

7.3.4 Influence of delay interval on PPV

Using a combination of single hole waveforms and computer simulation, the influence of delay interval was investigated for the conditions of OC-2. It is found that the delay of 25-35 ms produces the lowest vibration (Fig. 7.10). This agrees with the mine's practice.

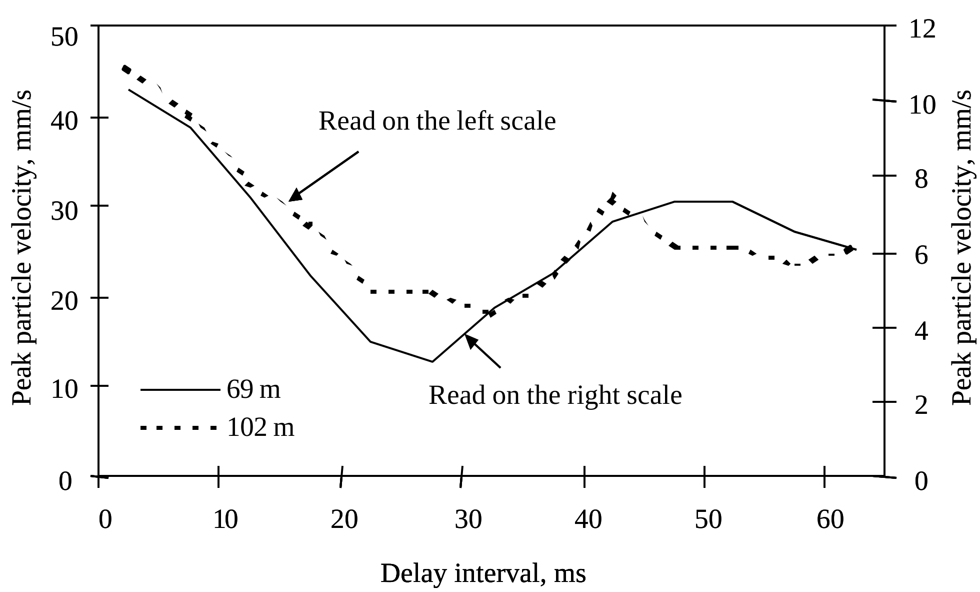


Fig. 7.10 Influence of delay interval on peak particle velocity at OC-2, SCCL

Chapter 8

NUMERICAL MODELING TO STUDY THE EFFICACY OF VIBRATION ISOLATORS

8.1 Introduction

There are many parameters that govern the generation and propagation of ground vibrations. Chapter 7 has dealt in detail the influence of various parameters on the intensity of ground vibrations. In some cases even by adjusting these parameters, vibration levels may not be within the acceptable levels. Under these constrained conditions some surface mines are resorting to vibration isolation by making trenches or pre-split planes. Though it is established that discontinuities like fault, pre-split, trench etc attenuate ground vibrations, many researchers and practicing engineers are of the opinion that field experiments to ascertain the extent of damping due to a trench or trenches becomes very expensive and cumbersome. Moreover, unless we have a prior knowledge of the extent of damping achieved by trenches, pre-split planes etc, it becomes a difficult proposition to practically execute these techniques. Venkatesh (2002) and Prakash et al (2004) have conducted some experiments with regard to use of trenches for reducing ground vibrations but it is felt that computer simulation may prove to be inexpensive, fast and realistic approach to arrive at the design parameters of a trench to meet the field requirements. Keeping this in view, 3DEC, a distinct element code is used to simulate opencast blasting to establish the extent of reduction in vibration intensity due to varying trench depth.

8.2 Details of the Software Used

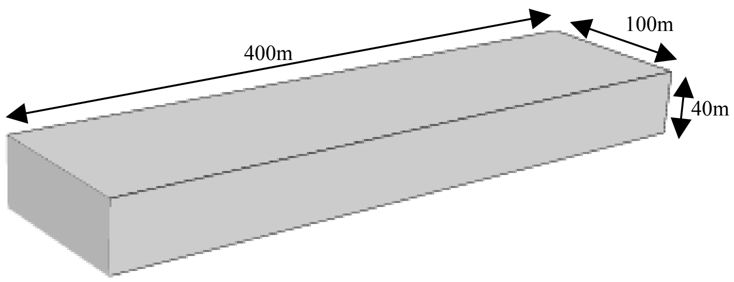
3DEC, a distinct element three-dimensional software was used to simulate the blasting. The distinct element method is a technique to simulate the mechanical response of systems composed of discrete blocks or particles (Anon, 1998). Basic assumption in this model is that particle shapes are arbitrary, any particle may interact with any other particle and there are no limits placed on particle displacements or rotations. Distinct element programs use an explicit time-marching scheme to solve the equations of motion directly. Bodies may be rigid or deformable (by subdivision into elements) and contacts are deformable. To customize the code to specific problem solving, 3DEC is embedded with FISH a programming language that enables the user to define new variables and functions. These functions are used to extend 3DEC's usefulness or to add user-defined features.

8.3 Computational Steps for Model Development

- Creation of geometry, placing the boundaries at sufficiently large distance from the area of interest to minimize the influence of the boundary conditions.
- Create discrete blocks cutting the geometry using the discontinuities like joints and excavations.
- Specify the rock mass properties like Young's Modulus, Poisson's ratio and joint properties like normal and shear stiffness and cohesion and friction angle.
- Apply boundary conditions including the energy absorbing boundaries by incorporating the standard viscous boundaries. These boundaries absorb energy of out ward moving waves.
- Apply in- situ stresses.
- Apply loading along the walls of the blastholes. The unbalanced forces in each block will give acceleration to the blocks, causing the blocks to move, thereby transmitting forces to adjacent blocks.

8.4 Creation of the Model

A computational model of dimensions 400 m x 40 m x 100 m size was used (Fig.8.1). A free face was created with a bench height of 7 m (Fig. 8.2). A blast hole of 150 mm diameter was created at a distance of 4 m (burden) from the free face. These parameters are of those full-scale single hole blast conducted at Kamptee Opencast mine (Chapter 7). A set of vertical joints with a spacing of 20 m and horizontal joints with a spacing of 10 m was generated (Fig. 8.3). Energy absorbing viscous boundaries was applied on all sides except on the top, which is a free boundary.



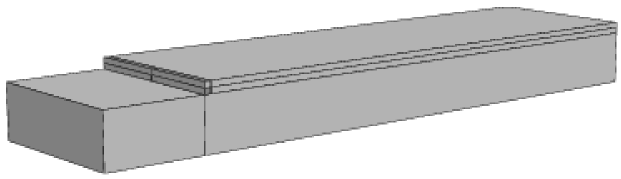


Fig. 8.2 Creation of a 7 m bench and a hole of 150 mm diameter at 4 m burden

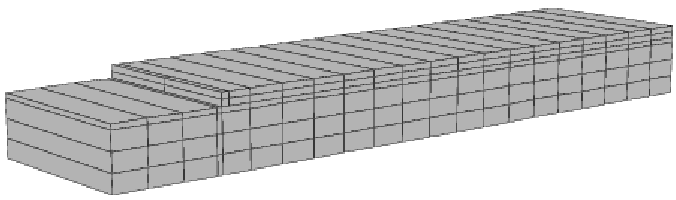


Fig. 8.3 Joints incorporated in the model

Once the basic geometry was created, the next step was to apply the blasting load on the blasthole walls. As such 3DEC cannot simulate the explosion process and the explosion load should be provided to the 3DEC model. The explosion load is very difficult to determine for actual explosion events (Chen et al., 2000). In many other studies, the representation of explosion load is often empirically assumed as a triangular pulse (Francois et al., 1993). In the present case also, velocity time history was applied as triangular pulse to the blasthole walls by developing a FISH function for this purpose. The boundary logic in the 3DEC code had to be modified to provide a more general command structure. Here, the memory location of block grid points, where loading is to be applied was stored in an array. The blast loading was applied as functions of time at these block vertices. The loading history is shown in Fig. 8.4. The velocity monitoring was carried out at two locations i.e., at 146 m (Location A) and at 188 m (Location B) behind the hole parallel to the X-axis. The developed model was calibrated by repeated runs by altering the input velocity and adjusting the material and joint properties (Fig. 8.5). The model output is close to the actual field test at both the monitoring locations (Table 8.1). This shows that the attenuation characteristic of the rock mass has been replicated in the model. This calibrated model was used to study the extent of reduction in ground vibrations due to a trench.

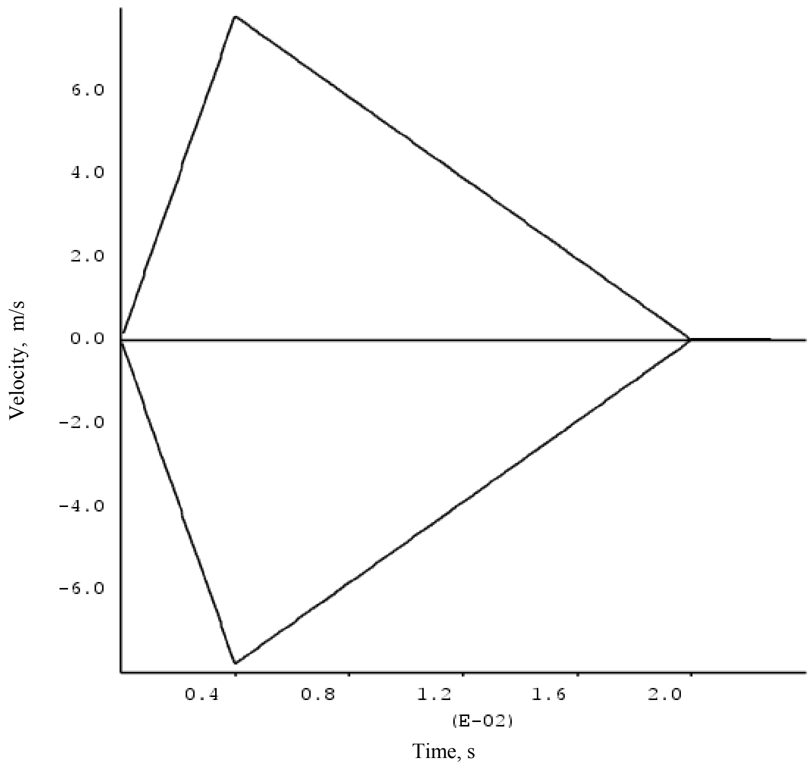


Fig. 8.4 Loading of blasthole with a triangular pulse – 7.8 m/s velocity

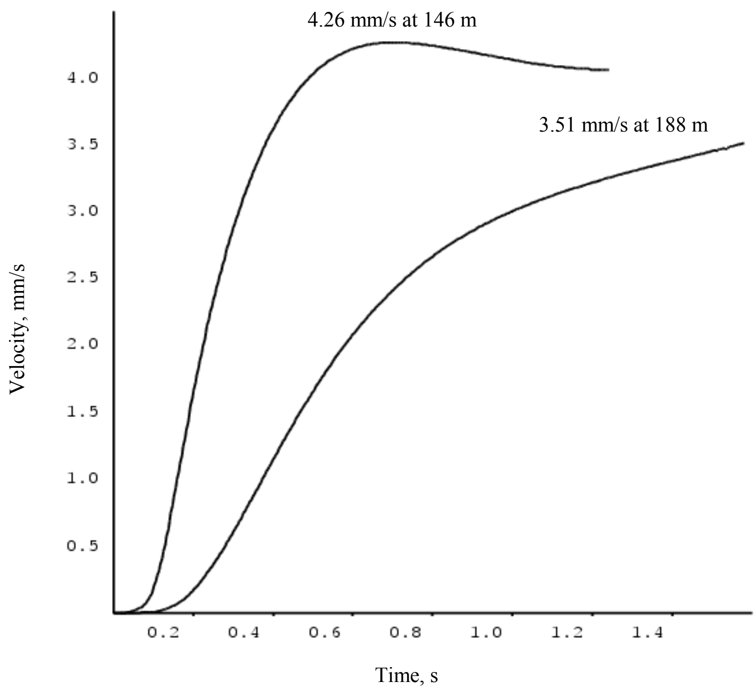


Fig. 8.5 Generated vibration histories - calibrated model

Table 8.1 Comparison of 3DEC modeling results with field test

Distance (m)	Velocity (mm/s)	
	Field test	3DEC modeling
146	4.19	4.26
188	3.68	3.51

8.5 Computation of Vibration for Different Trench Conditions

In order to establish the extent of reduction in ground vibration due to trench, model studies were carried out for trench depths of 3.5 m, 7 m, 10.5 m and 14 m. They represented the trench depths (T) equal to, 1.5 times and twice the blasthole depth (H). To start with, a trench of 1 m wide was created in the calibrated model at a random distance of 66 m behind the blasthole and this was 80 m and 122 m before the monitoring stations A and B respectively (Fig. 8.6).

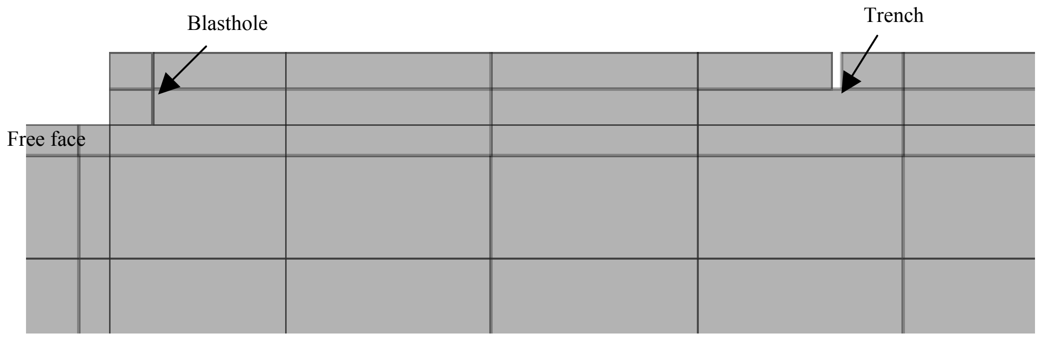


Fig. 8.6 Creation of a trench with its depth equal to half the hole depth

Fig. 8.7 shows the generated vibration histories at two monitoring locations A and B for a trench depth to hole depth ratio of 0.5.

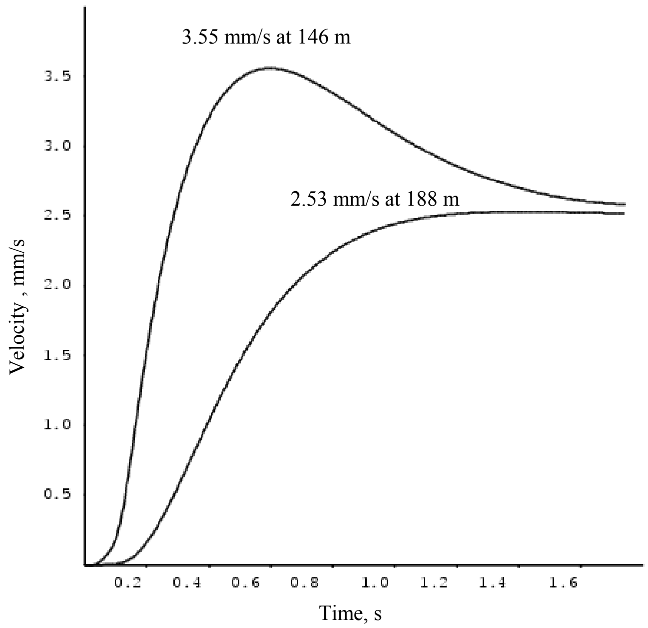


Fig. 8.7 Generated vibration histories at two locations with a trench equal to half the hole depth

Fig. 8.8 shows the generated vibration histories at two monitoring locations A and B for a trench depth to hole depth ratio of 1.0.

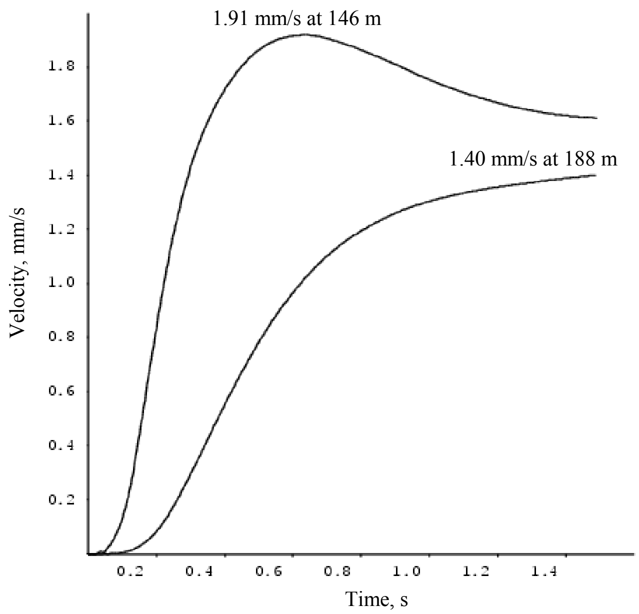


Fig. 8.8 Generated vibration histories at two locations with a trench equal to hole depth

Fig. 8.9 shows the generated vibration histories at two monitoring locations A and B for a trench depth to hole depth ratio of 1.5.

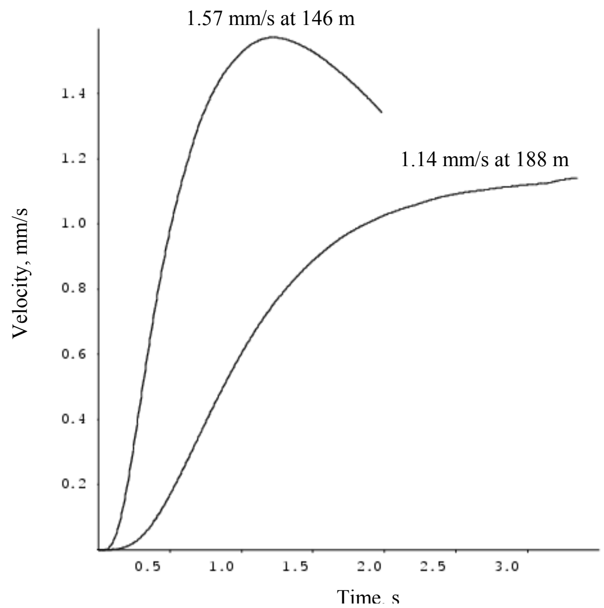


Fig. 8.9 Generated vibration histories at two locations with a trench equal to one and half times the hole depth

Fig. 8.10 shows the generated vibration histories at two monitoring locations A and B for a trench depth to hole depth ratio of 2.0.

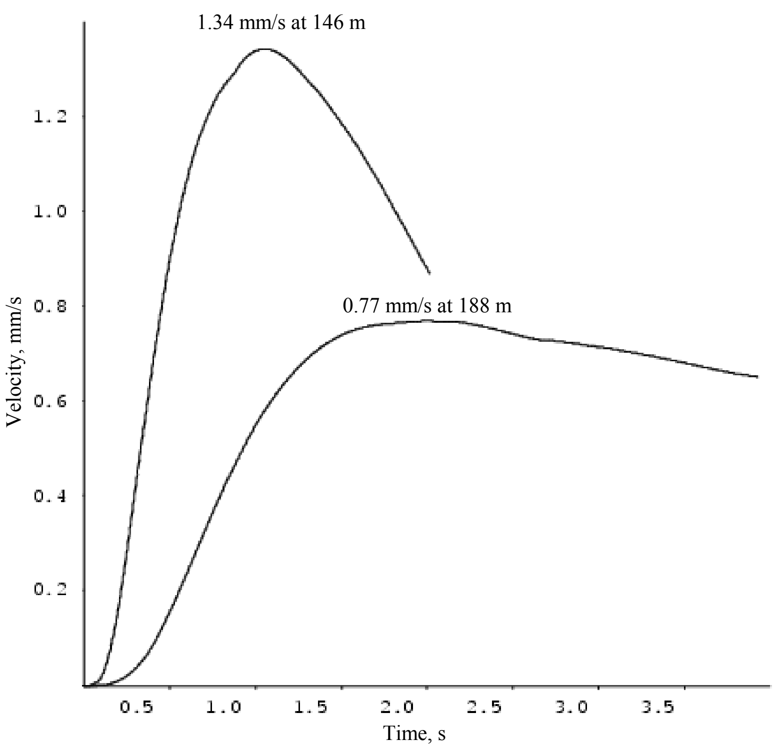


Fig. 8.10 Generated vibration histories at two locations with a trench equal to twice the hole depth

8.6 Results and Discussions

The model studies reinforce that trenches do reduce the vibration levels. Table 8.2 summarises the results from the model studies. It can be concluded that the reduction in vibration level is related to the depth of a trench and that the maximum efficiency of the trench is for the T/H ratio between 1 and 1.5. Cutting beyond 2 T/H ratio seems to be redundant as the reduction in vibration from 1 to 2 T/H ratio is only 13% as compared to that at T/H ratio of 1 which is about 55%. It is better to dig a parallel trench than deepening it to twice the blasthole depth.

Table 8.2 Summary of the results from the model study

Depth of trench (m) T	Hole depth (m) H	Ratio T/H	Velocity at 146 m from blast (mm/s) A	Velocity at 188 m from blast (mm/s) B	Percentage reduction at A	Percentage reduction at B
Nil	7	Nil	4.26	3.51	-	-
3.5	7	0.5	3.55	2.53	16	27
7	7	1.0	1.91	1.40	55	60
10.5	7	1.5	1.57	1.14	63	67
14	7	2	1.34	0.76	68	78

Prakash et al (2004) measured vibrations on two sides of a trench and varied the trench depth for each experiment. The ratios of trench depth to blastholes were 0.3, 1.0 and 1.125 and the damping varied from 16.6 to 55 per cent. In these cases, the blast locations and trench location were the top overburden bench. For deep-seated blasts with a trench at the top bench, Venkatesh (2002) measured vibrations on two sides of a trench deeper than hole depth and concluded that the reduction in vibration intensity is between 11 and 18.5 per cent. The results from the model studies are in accordance with the field experiments and hence prove to be a reliable and cost effective tool to decide the vibration isolation parameters.

Chapter 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

An overview of ground vibration in terms of peak particle velocity (PPV) and frequency for different mines in India revealed the dominance of low frequencies in coal mines at which the permissible PPV as per the current DGMS regulation is 5 mm/s. With this restriction, a number of coal mines, located close to habitation, are striving for their survival. This study was aimed at providing technical justifications for revision of the DGMS vibration limits and at suggesting a proper strategy to contain ground vibration.

Relative performance of transducers mounted in four different ways was evaluated in terms of peak particle velocity (PPV), peak vector sum (PVS) and frequency. For the given tolerance, the transducer freely placed on the surface recorded a few anomalous values of PPV and PVS at corresponding acceleration levels lower than 0.20 g, which might be suspected for poor coupling. Trace matching by superimposing one waveform on the other did highlight the difference between the two waveforms. The greater the difference, the lower was the correlation coefficient. This study indicates that transducers should never be placed freely on the surface irrespective of anticipated vibration levels. As a few cases were also suspected for decoupling with sandbagging or spiking, it would always be safer not to use these methods.

Based on the measured structure responses, frequencies of ground vibration were categorised into: (1) Low frequency (<20 Hz) – those within or below the natural frequencies of residential type of structure, where amplification factor is greater than 2.5; (2) Medium frequency (20-50 Hz): those above the natural frequencies, where amplification factor varies between 1.0 and 2.5; and 3) High frequency (>50 Hz): those much higher than the natural frequencies, where amplification factor is less than 1.0.

The damage studies at two coal mines revealed that the DGMS levels of ground vibration are very conservative. In other words, the factor of safety is very high. There is therefore ample scope for revising the current limits without defeating its basic purpose - adequate safety of surface structures.

Of the two parameters of ground vibration, peak particle velocity (PPV) can be controlled but the control measures may severely restrict the blasting operation. Besides maximum charge per delay, other variables such as delay interval, the explosives used and the numbers of free faces were found to have significant influence whereas total charge had an insignificant influence on PPV. Frequency, on the other hand, could not be increased beyond its normal range as it was primarily controlled by the ground conditions.

Numerical analysis using 3 DEC software indicated that a trench between the blast and the monitoring location could substantially reduce ground vibration. It was the trench depth (T) to blasthole depth (H) ratio that was crucial for the percentage of vibration reduction. At T/H ratio equal to 1.0, vibration could be reduced by 55-60 per cent. The results of the model studies were comparable to those of field measurements.

9.2 Recommendations

1) While no compromise can be made with regard to protection of surface structures from ground vibration, permissible vibration levels should not be unduly restrictive, posing constraints to mining operations. On the basis of this study, the DGMS vibration levels may be modified as proposed in Table 9.1. The permissible levels are given in terms of peak particle velocity and dominant frequencies are to be determined by FFT method.

Table 9.1 Proposed modification of the DGMS vibration limits (Permissible PPV in mm/s)

Type of structure	Dominant frequency, Hz		
	< 20 Hz	20 – 50 Hz	> 50 Hz
A) Buildings/ structures not belonging to the owner			
Domestic houses/ structures (Kuchha brick and cement)	10	15	25
Industrial Buildings (RCC and framed structures)	20	25	35
Objects of historical importance and sensitive structures	5	7	10
B. Buildings belonging to owner with limited span of life			
Domestic houses/ structures (Kuchha brick and cement)	15	25	35
Industrial buildings (RCC & framed structures)	25	35	50

2) Any legislation without serious enforcement and compliance serves little purpose. The mine management should therefore monitor ground vibrations for all blasts that are conducted close to surface structures to ensure that vibrations are within the permissible levels. Apart from other guidelines, due care should be given to transducer mounting for accurate monitoring of ground vibration.

3) The specification of seismographs, in the mentioned DGMS Circular, also needs to be changed. According to the DGMS, triaxial transducers for recording blast vibration shall have a linear frequency up to 500 Hz, capable of recording particle velocity up to 100 mm/s. Since the observed frequency is less than 100 Hz for mining blasts and not over 250 Hz even for construction blasts, triaxial transducers with a linear frequency response of 2 - 250 Hz capable of monitoring particle velocity up to 100 mm/s are sufficient for compliance monitoring. By incorporating these changes, the cost of seismographs would be reduced.

4) The practical measures that can be adopted to control ground vibration are:

a) Reduce the maximum charge per delay by:
   - Utilising the maximum number of delays
   - Using in-hole decking with two or more delays
   - Reducing the blasthole diameter
   - Reducing the bench height

b) Optimise the delay interval using a combination of field measurement and computer simulation using the linear superposition of waves.

c) Create free faces and maximum relief for subsequent rows to be blasted.

d) Try different types of explosives or for the same type from different manufacturers.

e) Use optimum specific charge, as both inadequate and excessive specific charge will increase ground vibration.

f) Wherever possible, ensure that the initiation sequence of blastholes progresses away from the structure.

g) Optimise blast design parameters for a given site condition.

h) Use special techniques like presplitting/trenching, only as a last resort.

5) In extreme cases, where the vibration limits cannot be adhered to, non-explosive method of excavation may be considered.

REFERENCES

Adhikari, G. R., Balachander, R., Theresraj, A.I., Venkatesh, H.S. and Gupta, R. N. (2005) Prediction of ground vibration from construction blasts, *Journal of Rock Mechanics and Tunnelling Technology*, Vol. 11, No. 1, pp. 3-12.

Adhikari, G. R, Singh, R. B. and Gupta, R. N. (1989) Structural response to ground vibration due to blasting in opencast coal mines, *J. Mines, Metals & Fuels*, April, pp. 135-138.

Adhikari, G.R., Theresraj, A.I., Venkatesh, H.S., Balachander, R. and Gupta, R.N. (2004) Ground vibration due to blasting in limestone quarries, *International Journal for Blasting and Fragmentation*, Vol. 8, No. 2, pp. 85-94.

Agarwal, P.N. (1991) *Engineering Seismology*, Oxford & IBH Publishing Co. Pvt. Ltd., New Delhi.

Anderson, D.A., Ritter, A.P., Winzer, S.R. and Reil, J.W. (1985) A method for site-specific prediction and control of ground vibration from blasting, *Proc. 1st Mini-Symp. Explosives and Blasting Research*, San Diego, Ca, pp. 28-43.

Anderson, D.A., Winzer, S.R. and Ritter, A.P. (1982) Blast design for optimising fragmentation while controlling frequency of ground vibration. *Proc. 8th Conf. Explosives and Blasting Technique,* New Orleans, pp. 69-89.

Anon (1998) *Itasca user manual – 3DEC*.

Armstrong, LW and Sen, G.C. (1999) The measurement of blast induced vibrations in soil. *Proc. EXPLO'99*, Kalgoorlie, 7-11 November, pp. 99-104.

Beattie, M, (1992) The effect of water table on peak particle velocities from blasting operations, *Proc 18th Conf. Explosives and Blasting Technique*, Orlando, Florida, Jan. 19 - 23, pp. 181-189.

Bhushan, V. and Sharma, R.C. (1992) Vibration studies at Bina, *Indian Mining & Engineering Journal*, February, pp. 23-28.

Blair D.P. (1987) The measurement, modelling and control of ground vibrations due to blasting. *Proc. 2nd International Symp. Rock Fragmentation by Blasting*, Keystone, Colorado, pp. 88-101.

Blair, D.P. (1995a) Blast vibrations in soil and on large resonant structures. *Proc. Explo'95*, 4-7 September, Brisbane, pp. 317-322.

Blair, D.P. (1995b) Soil-Embedded Detector Mounts for Seismic Monitoring. *Geophysics*, 6(1), pp. 120-133.

Chen, S. G., Cai, J. G., Zhao, J., and Zhou, Y. X. (2000) *3DEC modeling of a small scale-field explosion test, Pacific Rocks-2000*, Balkema, Rotterdam: 571-576.

Crum, S.V. and Pierce, W.E. (1995) House response to low frequency ground vibrations from coal mine overburden blasting: A technical updates," *Proc. 11th Symp. Explosives & Blasting Technique*, Feb 5-9. Tennessee, pp. 76-88.

Dowding, C.H. (1992) Suggested method for blast vibration monitoring, *International Journal Rock Mech. Min. Sci. & Geomechanics Abstr.* Vol. 29, No. 2, pp. 143-156.

Francois, E.H., Walton, O. R., Maddix, D. M., Shaffer, R. J., and Butkovich, T. R. (1993) Analysis of explosions in hard rocks: The power of discrete element modeling, *Comprehensive Rock Engg.* Vol. 2: 387-413.

Harries, C. and Gribble, D.P. (1993) The development of low shock energy explosive ANRUB, *Proc. 4th Int. Symp. Rock Fragmentation By Blasting*, Vienna. pp. 379-386.

Hinzen, K.G. (1988) Modelling of blast vibrations, *Int. Journal Rock Mech. Min. Sci. & Geomech. Abstr.*, Vol. 25, No. 6, pp. 439-445.

Hossaini, S.M.F. and Sen, G.C. (2004) Effect of explosive type on particle velocity criteria in ground vibration, *The Journal of Explosive Engineering*, July/August, pp. 34-39.

Hunter, C., Fedak, K. and Todoeschuck, J.P. (1993) Development of low density explosives with wall control applications, *Proc.19th Annual Conf. Explosives and Blasting Techniques*, Jan, 31-Feb. 4, San Diego, California, USA, ISEE, pp. 549-555.

Instantel (2003), Canada. *MiniMate Plus Operator Manual.* Software version 7.1.

ISEE (1998) *Blasters' Handbook*, International Society of Explosive Engineers, Ohio, USA.

ISEE (2005) Field Practice Guidelines for Blasting Seismograph. www. isee.org.

Jimeno, C.L., Jimeno, E.L. and Carcedo, F.J.A. (1995) *Drilling and Blasting of Rocks*, A. A. Balkema, Rotterdam, Netherlands.

Medearis, K. (1978) Blasting damage criteria for low rise surface structures. *4th Annual Conf. on Explosives and Blasting Technique*, Louisiana, Feb. 1-3, pp. 280-290.

Moore, A.J., Richards, A.B., Gad, E and Wilson, J. (2003) Structural response of brick veneer houses to blast vibration, *Proc. 29th Annual Conf. Explosives and Blasting Technique.*

Pal Roy, P. (1998) Characteristics of ground vibrations and structure response to surface and underground blasting, *Geotechnical and Geological Engineering*, Vol. 16, pp. 151-166.

Prakash, A.J., Palroy, P., and Misra, D.D., 2004. Analysis of blast vibration characteristics across a trench and a pre-split plane, *Fragblast*. Vol, 8, No.1: 51-60.

Robertson, D.A. (1993) The Neglected Step to Accurate Blast Monitoring: Proper Coupling. *Proc. 9th Annual Symp. on Explosives and Blasting Research*, ISEE, January 31-Feb.4, San Diego, pp. 83-98.

Singh, B., Pal Roy, P., Singh, R.B., Bagchi, A., Singh, M.M. and Nabiullah, Md. (1993) *Blasting in Ground Excavations and Mines*. Oxford & IBH Publishing Co. Pvt. Ltd. New Delhi.

Singh, P.K. (1998) *A Study on Ground Vibrations due to Rock Blasting*, Ph.D. Thesis, Technical University of Clausthal, Germany, Papierflieger, Clausthal-Zellerfeld.

Siskind, D.E. (2000) *Ground Vibrations From Blasting*, International Society of Explosives Engineers, Ohio, USA.

Siskind, D.E., Stagg, M.S., Kopp, J.W. and Dowding, C.H. (1980). *Structure Response Produced by Ground Vibration from Surface Mine Blasting*, U.S. Bureau of Mines, RI 8507.

Stagg, M.S. and Engler, A.J. (1980) *Measurement of Blast-induced Ground Vibrations and Seismograph Calibration*, US Bureau of Mines RI 8506.

Theresraj, A.I., Adhikari, G. R. and Gupta, R.N. (2004) Possible reasons for higher blast vibrations at Neyveli Lignite mines, *National Seminar on Rock Fragmentation*, BHU, Varanasi, January 23-24, pp. 151-159.

Theresraj, A.I., Balachander, R., Venkatesh, H.S., Adhikari, G. R. and Gupta, R. N. (2003) Blast vibration studies in some coal and iron ore mines, *Mining Engineers' Journal*, April, pp. 17-22.

Venkatesh, H.S. (2002) Influence of explosive charge on blast vibrations in surface mines, *Ph.D Thesis*, NITK, Mangalore University.

Vidyarthi, D. (2004) Personal communication.

Wheeler, R. (2005) The importance of saving the full wave from and frequency analysis, *Proc. 31st Annual Conf. Explosives and Blasting Technique*, Orlando, FL. 6-9 February.

Wheeler, R. (2004) The importance of proper seismometer coupling. *Proc. 30th Annual Conf. on Explosives and Blasting Technique*, 1-4 February, New Orleans, Vol. 2, pp. 147-161.

Yuan, L.,Jianwu, L., Quanjun, X., Xizhi and Xiang, Z. (2002) Study on low-rise residential house's damage caused by blasting, *Proc. 7th Symp. Rock Fragmentation by Blasting*, China, pp. 605-609.

ACKNOWLEDGEMENTS

This research work was carried out by National Institute of Rock Mechanics in collaboration with Western Coalfields Limited (WCL) and Singareni Collieries Company Limited (SCCL). The collaborators provided financial support for conducting experiments. Out of the total cost of the project of Rs. 37.6 lakhs, WCL's contribution was Rs. 5.0 lakhs and SCCL's Rs. 3.0 lakhs. In addition, SCCL constructed three test structures exclusively for damage studies. We are highly thankful for their financial supports and encouragements.

Our thanks are due to General Manager (NT) of WCL (HQ), General Manager (R&D) of SCCL and S&T Department of Central Mine Planning & Design Institute, Ranchi for coordinating and monitoring the project work.

We are thankful to Mr. K. Ravi Shanker, Additional General Manager, OC-2, SCCL and to Mr. P. N. Thakur, Sub Area Manager, Kamptee, WCL for extending their supports, cooperation and active involvement while conducting field investigations. Last but not least, we thank officials of SCCL and WCL who have directly or indirectly contributed to the successful completion of the field experiments.

APPENDIX

SURVEY OF GROUND VIBRATION STANDARDS

Introduction

Some of the standards used abroad are presented in Appendix. These include the following:

1) Recommendations of US Bureau of Mines (Siskind et al, 1980)
2) British Standard BS 7385 Part-2 of 1993
3) Australian Standard AS2187-1993
4) German standard DIN 4150 of May 1986

1) Recommendations of US Bureau of Mines

The US mining industry basically follows the recommended limits of the US Bureau of Mines, which is presented in Figure A (Siskind et al, 1980). These limits are for residential houses.

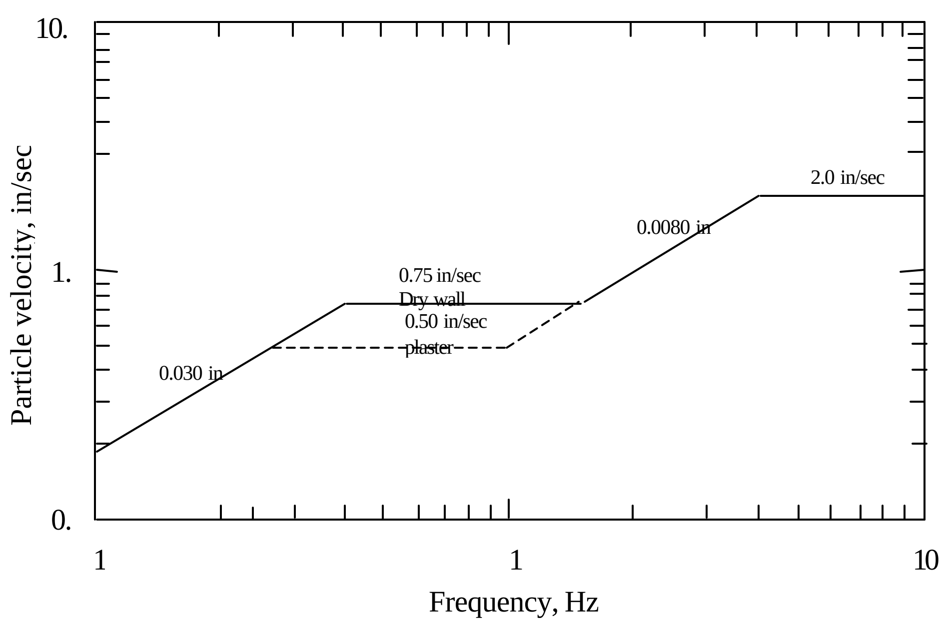


Fig. A Safe limits of blast vibration for houses (Siskind et al, 1980)

2) British Standard BS 7385, Part-2 of 1993

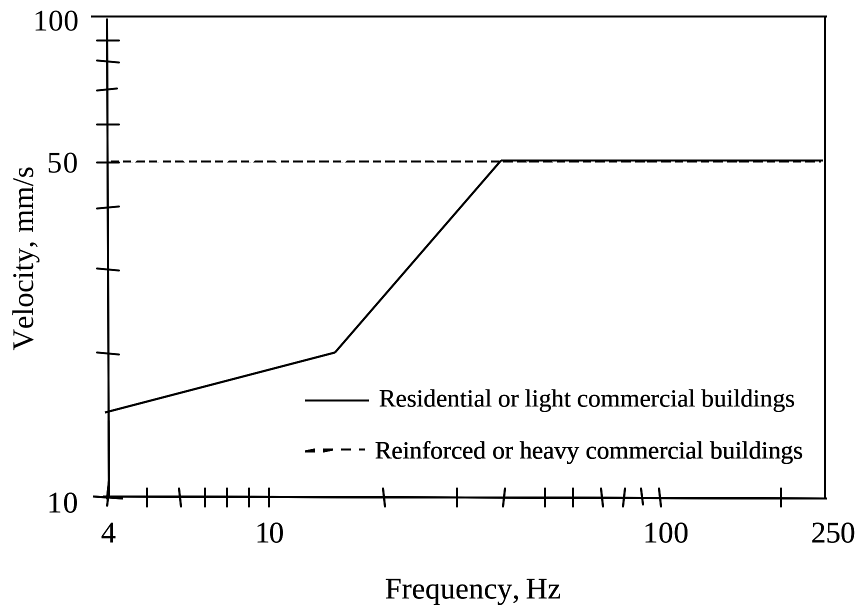


Fig. B British Standard BS 7385 of 1993

3) GERMAN STANDARD - GERMAN DIN 4150 (May 1986)

Guide values in terms of peak particle velocity (mm/s)

	Base	Base	Base	Upper floors
Type of structure	1-10 Hz	10-50 Hz	50-100 Hz	Any frequency
Offices and industrial buildings	20	20-40	40-50	40
Residential buildings and similar constructions	5	5-15	15-20	15
Buildings that do not come under the above because of their sensitivity to vibration	3	3-8	8-10	8

Measurements are made on the base of building. Peak particle velocity is defined as the maximum value of any direction. For high rise structures, the values are to be measured in the horizontal direction on the top floor of the building. They are applied independent of frequency.

4) AUSTALIAN BLAST VIBRATION LIMITS

Recommended Maximum Peak Particle Velocity

(Refer the Standards Association of Australia (SAA) Explosives Code AS2187-1993)

Type of building or structure	Peak Particle Velocity (mm/s)
Houses and low-rise residential buildings; commercial buildings not included below	10
Commercial and industrial buildings or structures of reinforced concrete or steel construction	25

Notes:

1. This recommendation does not cover high-rise buildings, buildings with long-span floors, specialist structures such as reservoirs, dams and hospitals, or buildings housing scientific equipment sensitive to vibration. These require special considerations which may necessitate taking additional measurements on the structure itself, to detect any magnification of ground vibrations which might occur within the structure. Particle attention should be given to the response of suspended floors.
2. In a specific instance, where substantiated by careful investigation, a value of peak particle velocity other than that recommended may be used.
3. The peak particle velocities consider both human discomfort and structural integrity together with the effect on sensitive equipment located within buildings.
4. Higher levels may be permitted for ground vibration with high frequencies

The Australian Standard is under revision.

Most Significant Findings of this Study

1. Compared with other surface mines, there is a significant presence of low frequencies (< 8 Hz) of ground vibration due to blasting at coal mines. For low frequencies, the permissible peak particle velocity as per the DGMS is 5 mm/s for residential structures and 10 mm/s for industrial structures. These limits are low by international standards.

2. This study provides a strong technical justification for revision of the DGMS limits. Based on field investigations at two large opencast coal mines involving pre- and post blast survey of structures, response structures to ground vibration and the analysis of the data, a framework is evolved for revision of the current DGMS standard.

3. The influence of four common methods of transducer mounting on vibration measurements was studied in the field. The results indicate that decoupling is most likely with the transducer freely placed on a horizontal surface. However, the sandbagged and spiked transducers are also prone to decoupling, resulting in higher or lower ground vibration. Therefore, burial should be the preferred method for mounting of transducers in soil.

4. This study identifies the blast design parameters that can be suitably modified to control peak particle velocity (PPV). The efficacy of a trench in further controlling PPV was analysed using numerical modelling. The results show that with a trench depth to blasthole depth ratio of 1.0, PPV can be reduced by 55 per cent.

National Institute of Rock Mechanics

(An Autonomous Research Institute under Ministry of Mines, Govt. of India)
P.O. Champion Reefs, Kolar Gold Fields – 563 117, Karnataka, India
Phone: +91-8153-275000, +91-8153-275008, Fax No. +91-8153-275002
Email: nirm@vsnl.com, nirm@sancharnet.in, Website: htpp:/nirm.kar.nic.in