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Advanced vibration management in quarries using a predictive blast vibration model

D. Grothe
Electronic Blasting System Specialist, Orica Mining Services (Germany)

P. Reinders
Blast Based Services, Orica Mining Services (Germany)

ABSTRACT: Most quarries are located in close proximity to cities and are tightly regulated because of the undesirable vibration and airblast effects of their blasting practices. Orica has developed an advanced proprietary model for the management and control of vibration and airblast at sensitive locations in the vicinity of mining activities. This model, a predictive Monte Carlo vibration model, is encapsulated in a suite of blast design software called SHOTPlus®-i Pro. The combination of innovative blast design, simulation capabilities and of accurate electronic delay timing opens a whole new range of possibilities to achieve environmental compliance and at the same time to improve the operation efficiency in the quarry industry. This paper gives an example for advanced vibration control in a German quarry using the predictive vibration model in combination with electronic initiation.

1. INTRODUCTION

In many quarry operations blast vibrations and airblasts dictate the mining sequence and technology and therefore have a negative impact on the blasting efficiency and as a consequence also on the productivity of the quarry operation. The approach of production areas to sensitive structures or residences typically leads to an increase of vibration and airblast levels. Often the charge weight per delay time must then be reduced to keep the blast vibrations within the compliance limits, which inevitably increases production costs.

Orica has a unique capacity through its Advanced Vibration Management (AVM™) service to measure, analyse and predict the blast induced vibrations with a proprietary model for the management and control of vibration at sensitive locations. An AVM™ service can be used for optimising blasting design and technology under consideration of the maximum allowable vibration levels in specific points of interest and also for predicting the expected vibration levels in these points (Grothe et al. 2006). Typically electronic initiation is used as part of the Advanced Vibration Management service, because it gives the most effective vibration control mechanisms due to its precision, free programmability and flexibility (Blair et al. 1999). The need for this type of service is increasing as urbanisation encroaches on quarries, restricting their ability to operate. Vibration restriction can also limit the mineable

quarry assets and consequently an improvement in blast vibrations would thus effectively increase those mineable assets for the quarry operator, a very valuable outcome in an environment of increasing urban encroachment and sensitivity to blasting operations.

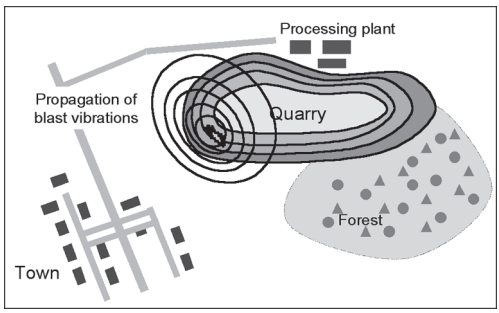

Figure 1. Environmental constraints in quarrying.

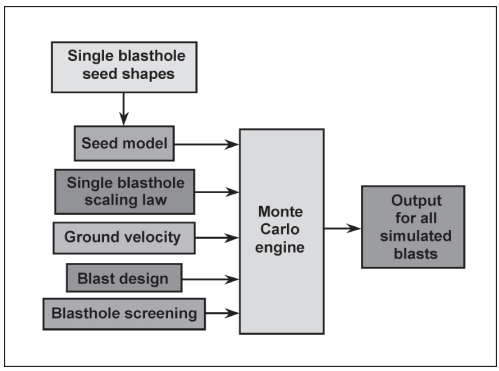

Figure 2. Block diagram of the vibration model.

2. STATISTICAL VIBRATION MODEL

The statistical vibration prediction model uses a Monte Carlo technique and has been used successfully for more than 10 years in the Australian mining industry to control vibration and airblast problems (Blair 1999). The capability of the prediction model has been, and still is being, continuously improved on the basis of comprehensive field studies and research work (Blair 2007). The application of the model - the Monte Carlo engine - requires the definition of a variety of input parameters as shown in the block diagram in Figure 2. The input parameters include the vibration seed shape waveforms from signature blastholes, the single blasthole scaling law, the ground velocity, the delay scatter of the initiation system, the charge weights, the delay time sequence and the geometric layout of the blast pattern with respect to the monitoring site. The model also considers the blasthole screening effect, i.e. the vibration reduction through fractured rock caused by previously fired blastholes in the initiation sequence.

In simple terms, the model simulates the vibration and /or air blast at a point of interest for a given blast design by linear superposition of a sequence of phase delayed single blasthole signatures. The prediction output is - amongst others - given in the form of a probability for the expected peak vibration level for a specific compliance location.

3. CASE STUDY

The quarry is located near Frankfurt in the country of Rheinland-Pfalz, Germany. The mineral is a highly compact lime stone which developed in the geological era of the Devonian some 350 million years ago. In close proximity to the quarry there are two villages, one in a distance of 800 m to the west and the other about 400 m to the east. Furthermore, there are two single residences located at distances of 300 m and 250 m respectively from the quarry boundaries.

Together with the quarry management four houses were identified as the most critical compliance locations, all east of the quarry. These are two single residences (house 1 and 4) and two houses in the centre of the village (house 2 and 3). Figure 3 shows these compliance locations on the basis of a three dimensional CAD file of the quarry that was imported into the blast design program.

3.1 Site exploration study

The initial step in an advanced vibration management program must always be a comprehensive site exploration study, as the vibration modelling capability is entirely dependent on the quality of the input parameters for the ground model. The seismic exploration has three main objectives:

– to record the single blasthole seed waveforms, which represent the unique geological site signature between the quarry blasting area and the monitoring locations, the points of interest (POIs)
– to establish the site specific scaling law, which

describes the expected vibration level as a function of charge weight and distance
– to determine the wave propagation velocity through the ground

All three objectives can be achieved through the firing of single blastholes. In this case eleven single blastholes were fired for the seismic survey of the site. All signature blastholes were charged with explosives, which are also used in production blasting.

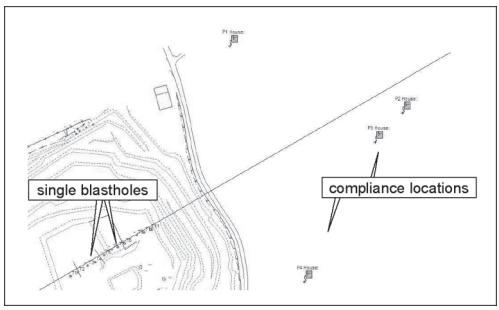

Figure 3. Seismic site exploration study.

The accelerometers of the free field vibration monitors were coupled to the ground in front of the houses to record blast induced vibration waveform over the time as well as the wave propagation velocity. Two techniques were used to achieve a strong ground coupling, one where the accelerometer was bolted to a circular disk that was in turn bonded to an outcrop of rock, and the second where the accelerometer was bolted to a cylinder carefully inserted and compacted into the ground. For the determination of the ground velocity a wire break trigger mechanism was used with the monitors. Therefore some signature blastholes had trigger cables connected between primers and monitors. With the known distance between the signature blastholes and the monitors, the p-wave ground velocity to each point of interest can be calculated.

3.2 Vibration control and blast design optimisation

After realisation of the seismic investigation and subsequent processing of the recorded data the site specific model could be established for each point of interest. With this a vibration control and blast optimisation program could start.

In a first step each blast was designed to minimise the vibration levels based on the existing blasthole loading technology, but using electronic initiation. Blast design and vibration modelling was carried out using SHOTPlus®-i Pro. It was found that the predicted peak vibration levels were in good agreement with the measured peak vibration levels at the POIs.

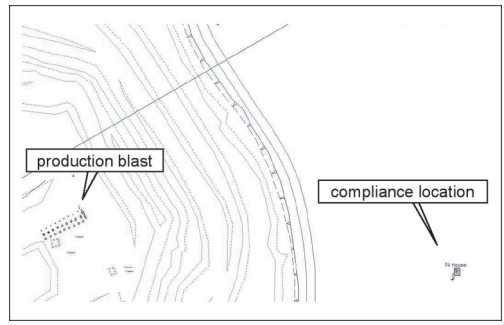

Figure 4. Vibration control for compliance location.

The German standard for vibration monitoring is the DIN 4150 ‘Vibration in construction work’. According to the DIN the vibrations have to be measured inside the building, whereas the vibration model is developed for a free field prediction. Therefore additional measurements had to be made inside the residential buildings at the start of the program. From the comparison of the predicted free field peak vibration with the measured peak vibration at the ground floor inside the residential building a transfer factor was determined for each POI. These factors were 50% for house 1, 100% for house 2, 70% for house 3, and 70% for house 4.

An example for the successful vibration control and reduction of the production blast shown in Figure 4 through blast design modelling is given in the following. Figure 5a shows the predicted blast waveform for an optimised blast design using electronic initiation, based on 1000 simulations for house 4. The upper blast waveform represents the envelope of the vector peaks for all 1000 simulations, the lower line the vector sum for the first predicted waveform. Figure 5b shows a histogram of the predicted peak vibration levels of the simulations. The predicted average peak vibration level is 3.0 mm/s with a standard deviation of 0.4 mm/s for the monitoring location in front of house 4. With the transfer rate (70%) for house 4 this gives an expected PPV of 2.3 mm/s for the ground floor measurement. The real measurement in house 4

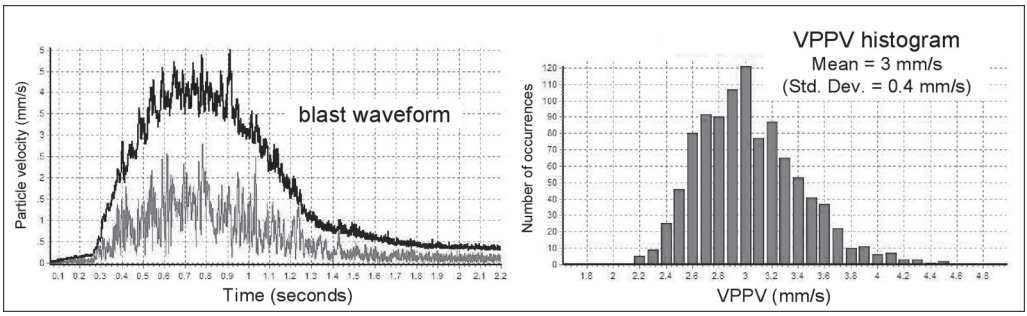

Figure 5. a) Predicted blast waveform b) Predicted vector PPV at house 4 using electronic initiation.

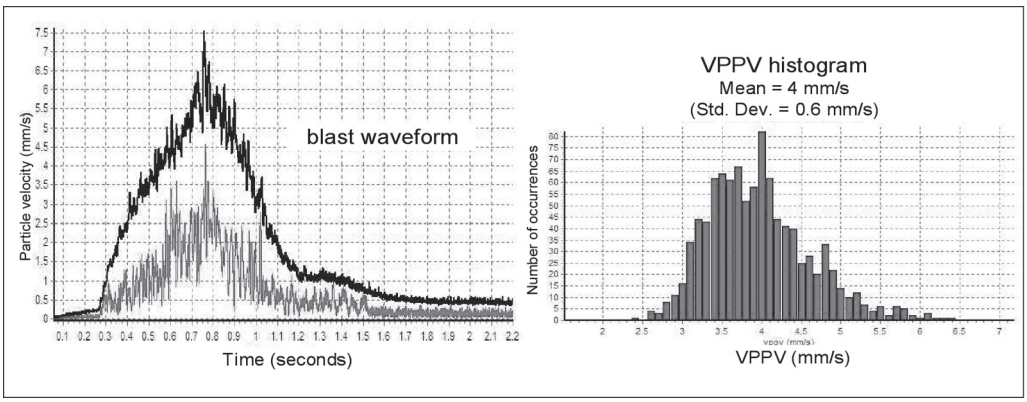

Figure 6. a) Predicted blast waveform b) Predicted vector PPV at house 4 using electric initiation.

gave a PPV reading of 2.37 mm/s.

In contrast to this Figure 6 shows the prediction results of the same blast using electric initiation with pyrotechnical delay timing based on traditional delay sequencing, also for 1000 simulations. A look at the blast waveform already reveals that higher vector peaks are predicted when compared with figure 5a. The predicted average peak vibration level as shown in the histogram in figure 6b is 4.0 mm/s with a standard deviation of 0.6 mm/s for the monitoring location in front of house 4. Compared to the optimised blast design, which additionally uses electronic initiation, the resulting vibrations in the compliance location would - most likely - have been significantly higher.

So far the vibration model has solely been used to minimise and control the ground vibration in the most critical compliance locations near the quarry, in order to avoid complaints from the neighbourhood. The control and management of the blast induced vibrations in daily production blasting environment has proofed to be successful, which gives room for the next level of optimisation for the quarry. The aim is to reduce the overall mining costs in limestone production under retention of the currently achieved successes in vibration control. In the near future a new bench at the bottom of the pit will be mined with an opening point in the most vibration sensitive area of the quarry. At this stage productivity improvements through changes in the blasting technology like blast pattern optimisation and increased size of blasts will be assessed under control of the blast induced vibrations.

4. CONCLUSIONS

The application of the statistical vibration model in an environmentally constrained quarry operation has helped to minimise and control the blast induced vibrations in daily production blasting using electronic initiation. The predictive

capabilities of the site model have shown to be in good agreement with the measured vibration levels at the compliance locations in close proximity to the quarry boundaries. Based on the current achievements in vibration control at the site further changes to the blast technology for productivity improvements will be evaluated. The advanced vibration modelling capabilities will ensure that new blasting technologies can be implemented without negative impact on the vibration levels at the critical compliance locations.

5. ACKNOWLEDGEMENTS

The support by the quarry management and personnel is gratefully acknowledged, in particular the effort of the blast crew during the seismic investigation study. Many thanks also to our Orica colleagues Mike Noy and Nathan Cotter who helped during the site investigation and gave invaluable assistance throughout the program.

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An example of preliminary seismic survey to evaluate the feasibility of blasting works in proximity of a sensitive monument

M. Cardu
CNR-IGAG, Turin, Italy / Politecnico di Torino, Land, Environment and Geo-Engineering Department, Turin, Italy

A. Giraudi, E. Lovera, E. Michelotti
Politecnico di Torino, Land, Environment and Geo-Engineering Department, Turin, Italy

ABSTRACT: The case study refers to the proposed excavation, by drilling and blasting, of the planned access facilities to an ancient abbey. Campaigns of test blasts approaching the features of the actual blasts needed to excavate a shaft and a tunnel, as required by the programme, have been carried out and the seismic effects have been recorded and analysed. A detailed account of the research is given, the results are discussed and practical advice to plan and actually perform that kind of surveys are underlined.

1. GENERAL

Ancient hermitages owe their fascination to the intrinsic beauty of the structures and of the art pieces hosted, to the intact surrounding landscape, and to the feeling of wonder they excite for having survived the centuries. To enable many people to enjoy these marvels, and to ensure the monument's survival in the coming centuries, it is often necessary to create inconspicuous and safe access facilities, obviously excluding any static or aesthetic damage to the buildings and to the surrounding environment.

Usually, in these cases, excavation work is carried out by mechanical means. This is not possible in the case of the 'Eremo di S. Caterina del Sasso' shown in the Figure 1, sited on the coast of Lago Maggiore, because heavy machinery cannot be carried to the working places, nor serviced, due to the lack of suitable roads and space.

The planned access facilities are shown in Figure 2, and consist of a shaft equipped with a lifting system, connecting the tophill level (254 m a.s.l.), where a parking area is available, to the level of the monks gardens (203 m a.s.l.), and an horizontal tunnel connecting the gardens to the bottom of the shaft. The rock to be crossed (the same on which the church is founded, and of which are composed the steep rock walls bordering the site) is a massive, but fractured, limestone (see Table 1).

It is therefore necessary to excavate both the shaft and the tunnel by very controlled drilling and blasting. The critical (most sensitive) objects are, obviously, the architectural complex of Medieval age, containing delicate frescoes, and the rock walls, reinforced by bolting and wire mesh protections, having already shown rock fall problems; also the old houses close to the shaft collar are at risk of damage from blasting work. Politecnico di Torino has been charged by the Province of Varese (responsible of the Conservation of the monument) to carry out a vibrometric study, aimed to set the charge limit values to be respected, in the shaft and

Figure 1. General picture of the 'Eremo di S. Caterina del Sasso' area. The two areas chosen for the test blasts are indicated.

Table 1. Technical characteristics of the rock mass interested by the excavation.

d	Mass density	2543 kg/m^3
c	Seismic velocity within the rock mass	1500 m/s
σ_{cm}	Uniaxial compressive strength of the rock mass	6,25 MPa

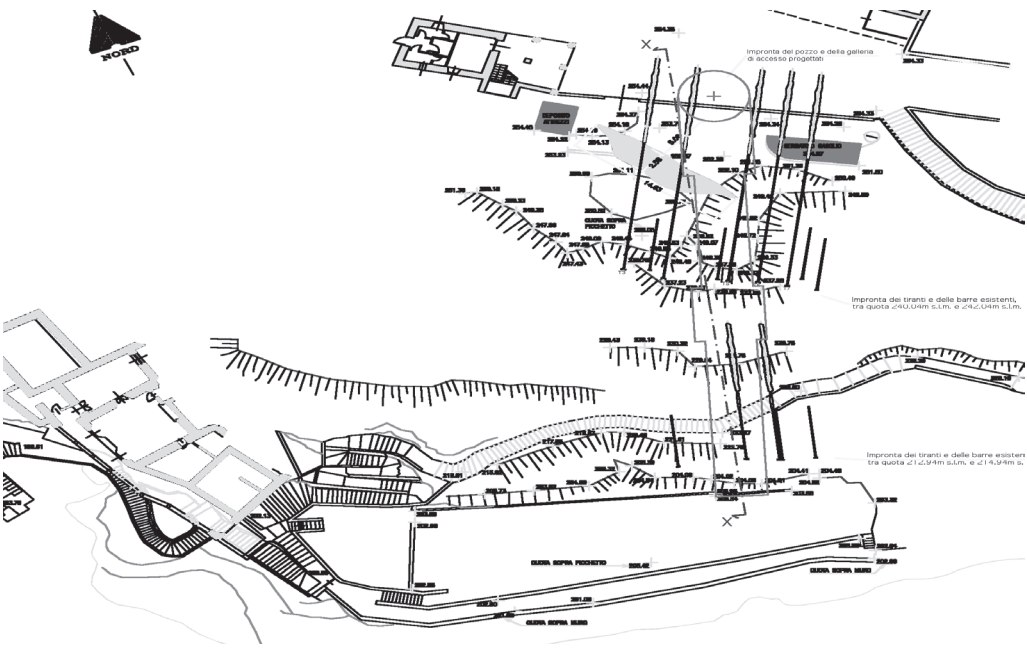

Figure 2. Plan view of the area and location of the objects to be protected with respect to the layout of the shaft-tunnel system. The sensitive elements are indicated as Obx; the most critical blasting positions during effective excavation are indicated as Sx

tunnel excavation, and to suggest any caution to be taken in the blasting work, in order to minimize the risk of damaging the monument.

An account of the experimental studies and results is summarized in the paper.

2 SAFE P.P.V. LIMITS

For the historical building the p.p.v./frequency limit values set by norm DIN 4150 were assumed; the same was made for the houses close to the shaft collar: the Medieval monument was assigned to class 3, and the houses to class 2.

For the rock walls, a kind of object not listed in the norms and recommendations, conservative limits have been set through analogies with criteria accepted for comparable objects:

- The Swiss norm SN 640312a sets a p.p.v. limit of 30 mm/s for stone walls built without mortar, a kind of structure quite similar to a fractured rock wall; therefore, we retained this limit as acceptable also in our case
- For mining works, Ju and Vongpaisal (1996) developed and tested a criterion (Blast Damage Indicator, BDI criterion) to set safe p.p.v. limits for stone rock walls close to blasts, represented by a safety factor against rock mass tensile failure under the action of the reflected stress wave produced by the blast, which can be calculated from the geomechanical characteristics of the rock mass

The formula runs as:

$$p.p.v. = \frac{BDI \cdot \frac{\sigma_{cm}}{4}}{d \cdot c} \qquad (1)$$

Where: p.p.v. is the peak particle velocity, BDI the blast damage indicator (safety factor), σ_{cm} the compressive strength of the rock mass, d the mass density of the rock, c the propagation speed of the seismic disturbance. Being $\sigma_{cm} = 6.25 \cdot 10^6$ Pa, d = 2543 kg/m^3, c = 1500 m/s, and assuming for BDI the very conservative value 0.1 (excluding damage risk according to the quoted authors), we find a limit p.p.v. value of 0.04096 m/s = 40.9 mm/s.

The limit value so found is somewhat higher than the limit set (by analogy) by the Swiss norm (30 mm/s), which proves the norm is still more conservative. We decided to apply the more conservative value (30 mm/s). We remind that, though being a rock wall by far less delicate than a medieval relic, the rock wall deserves close attention because of the proximity to the foreseen blasting works, and because even a small rock fall, or a slight weakening of the rock, represents a serious danger (both for the monument and for the people).

Summing up, the p.p.v. limit values set, taking into consideration also the fact that fractured rock preferentially transmits low frequency disturbances, are:

- 3 mm/s for the historical monument
- 5 mm/s for the inhabited old houses close to the shaft collar
- 15 mm/s for other buildings close to the shaft collar, of more robust structure, belonging in part to the commercial buildings category
- 30 mm/s for the rock wall

The location of the objects to be protected with respect to the layout of the shaft-tunnel system is reported in Figure 2.

The 'critical object' (the most endangered by blasts) obviously changes during the excavation, and so does the charge per delay limit to be imposed.

3. TEST CAMPAIGN TO DEFINE A CHARGE - DISTANCE CORRELATION

These conditions had to be respected in planning the test explosions campaign:

- to exclude, obviously, any damage to the critical objects, but to use charges large enough to be representative of the effects of actual charges: neither scaling up nor scaling down are correct, beyond, say, one order of magnitude, irrespective of the scaling law chosen. Checks have been made of the foreseeable effects of different charges, on the basis of literature data and of data recorded in past test campaigns carried out on the vibration propagation in similar rock masses. A limit charge of 2 kg in the test explosions has been set
- to locate the test explosion points in proximity to the layout of the shaft-tunnel system to be excavated

Upon a careful survey of the site, only two rock exposures were accessible and exploitable for the test explosions, satisfying the aforesaid condition, and are labelled as zone A and zone B in the Figure 1;

- to perform test explosions similar, in principle, to the actual explosions composing a tunnel

shaft excavation round, say, representing a 'production' blast (enlarging an already obtained cavity) and a 'cut' blast (creating the initial opening in the rock face). Upon examination of the morphology of the rock exposures exploitable, it was decided to use zone A for the 'production' test and zone B for the 'cut' test (Figures 3, 4, 5, 6)

– to complete the explosion tests program in few hours, to minimize the interference with the religious functions of the church, with the tourist visits, with the activities of the monks and of the dwellers of the surrounding houses, and with the recreative uses of the facing lake (fishing, sailing and so on)

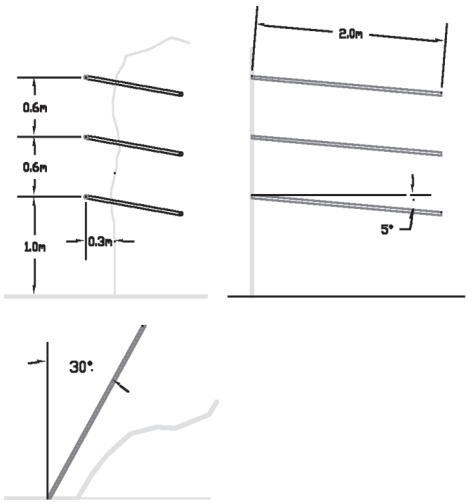

Figure 3. Blasting design for test blast in zone A. Above, on the left, front view with indication of the blast holes and the burden. On the right, vertical section. Below, plan section.

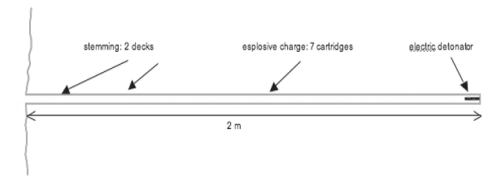

Figure 4. Loading scheme for the blast holes n. 1, 2 and 3 in zone A.

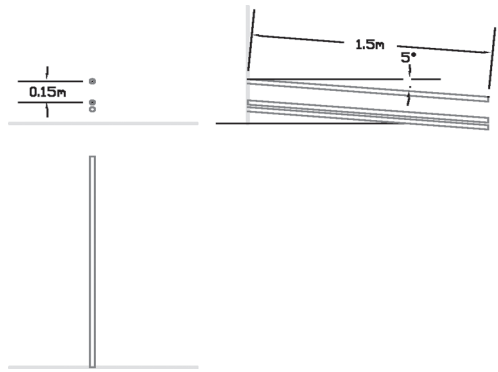

Figure 5. Blasting design for test blast in zone B. Above, on the left, front view with indication of the blast holes and the burden. To be noticed that only the upper hole is charged. On the right, vertical cross section. Below, plan section.

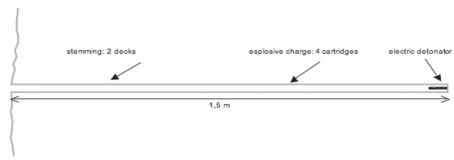

Figure 6. Loading scheme for the blast holes n. 4 in zone B.

That simply means that all preliminary work (topographic survey, marking places, access paths, scaling, drilling, preparation of the seats for the sensors) enjoyed ample time to be completed and checked, while area clearing from unauthorized persons, surveillance of possible accesses, explosive transportation to the test places, charging, circuits check, sensors installation and testing, explosions, post blast safety checks (in particular, the elimination of possible local rock fall danger from the rock walls close to the explosions) were to be completed in a very short time, to fit the allowed temporary window. The main fear, during tests, was obviously represented by fly-rock and by rock stability after the blast. Safe distances were calculated for each test blast, anyhow the local authorities, for safety sake, prohibited access to the whole area to visitors during the tests.

4. RECORDING POINTS

Five vibration velocity recorders and one accelerometer were available. Recording points are shown in Figure 7.

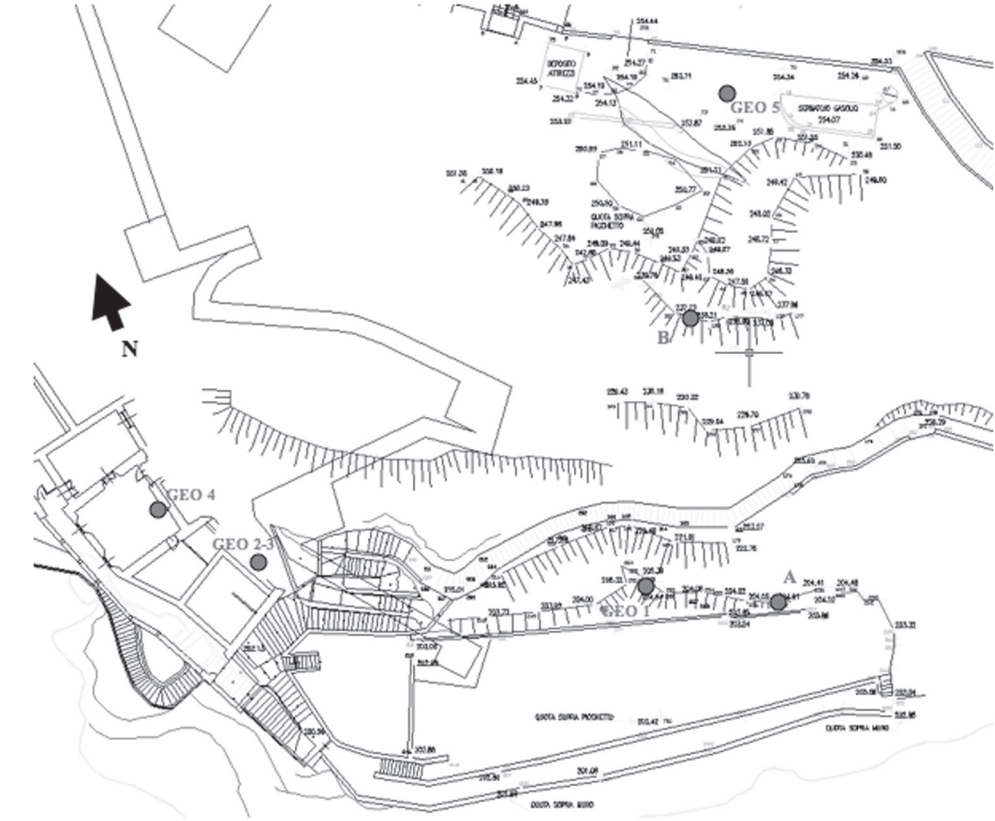

Figure 7. Position of recording devices, indicated as GEOx. The blasting areas are indicated as A and B.

One of the velocity recorders, and the accelerometer, were to be placed at the most sensitive point to be protected, the frescoes hall, position GEO4. Two velocity recorders were fastened with quick setting mortar to the rock wall in a crypt underlying the building closest to the explosions of the hermitage complex; the reason for placing two sensors closely spaced in this position (GEO2 – GEO3) was a conspicuous rock joint showing on the wall: by placing sensors on both sides of the joint, and by comparing the vibrograms recorded, it was possible to have information on the extent of the interruption of mechanical continuity of the rock mass represented by the joint (if a strong difference is recorded, it implies a dangerous situation, briefly, a building resting on separate rock masses). One velocity recorder was fastened with quick setting mortar to the rock in position GEO1, very close, 8.5 m, to test explosion point A, at a scaled distance of only 6 m/kg$^{1/2}$; no significant rock discontinuities show up between the point GEO1 and the explosion point A. The last velocity recorder was placed on the outcropping rock (position GEO5), and simply secured with a heavy sand bag because it was not possible to fix the sensor with quick setting mortar. Distance from explosion point B is 20 m.

Distances between each sensor and each explosion point were carefully measured.

5. EXPLOSION TESTS AND DISTANCE – CHARGE – P.P.V. CORRELATION

Three separate events have been produced and recorded, namely:

- explosion point A: a charge of 2.12 kg (two holes charged with 1.06 kg, simultaneously fires)
- explosion point A :a charge of 1.06 kg (a single

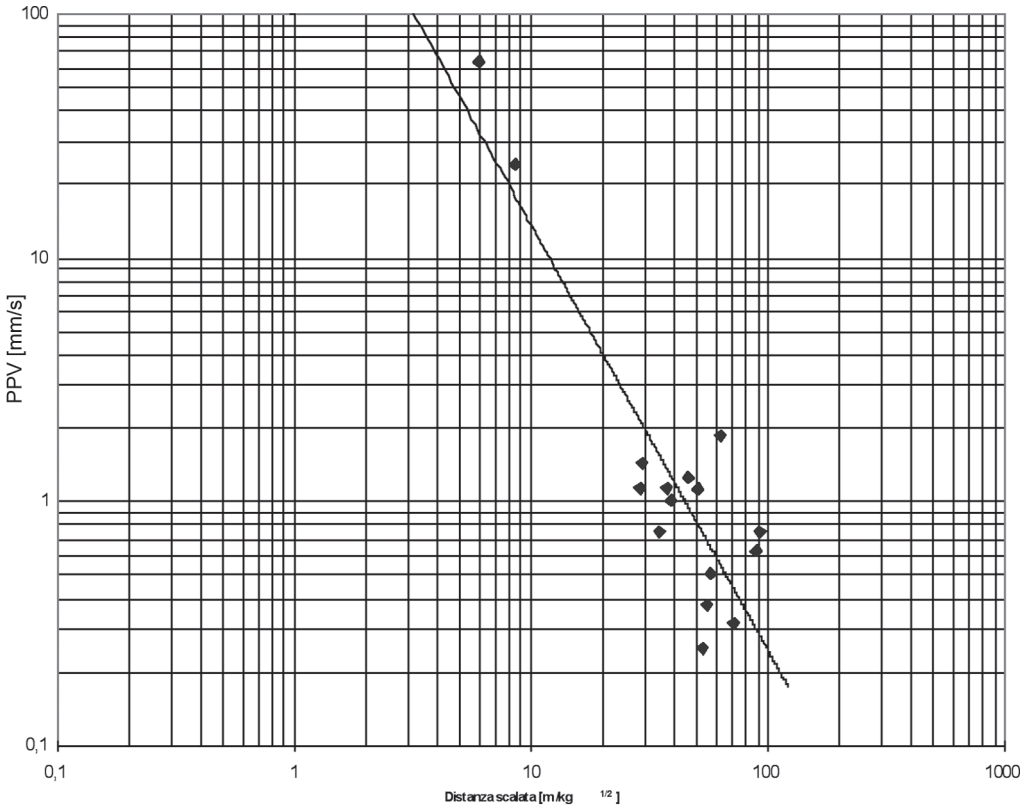

Figure 8. Correlation chart between ppv max and scaled distance. The linear interpolation gives the experimental site law.

hole charged, fired 0.25 s later)

– explosion point B: a single charge of 0.61 kg

Details of charging and holes position are given at point 3.

From recorded vibrograms in the four recording positions the maxima components of p.p.v. have been obtained, and plotted in the p.p.v. – scaled distance (square root) chart, as shown in Figure 8.

A best fit 'site law' has been calculated, of the USBM type:

$$p.p.v. = 744 \cdot \left(\frac{R}{\sqrt{Q}} \right)^{-1.744} \qquad (2)$$

where p.p.v. is in mm/s, R, distance, in m, and Q, cpd, in kg.

But, as expected, the more confined explosions (point B, aimed to represent the effect of the cut in a tunnel or shaft round) produce, at a given scaled distance, more conspicuous disturbance than other explosions (representative points are circled in the Figure 9). Therefore, a conservative calculation of the maximum allowed cpd can not be based on the average 'site law', rather on a 'safe envelope' encompassing the worst cases. Equation found for the safe envelope is (Figure 9):

$$p.p.v. = 2286.1 \cdot \left(\frac{R}{\sqrt{Q}} \right)^{-1.744} \qquad (3)$$

Consigns for the blaster concerning maximum cpd have been calculated, as explained later, on the basis of the latter equation.

As to the distance – main frequency correlation, no reliable indication is obtained from the recorded vibrograms, apart from the tendency of the low frequencies (around 10 Hz) to prevail, which is an expected consequence of the low quality of the

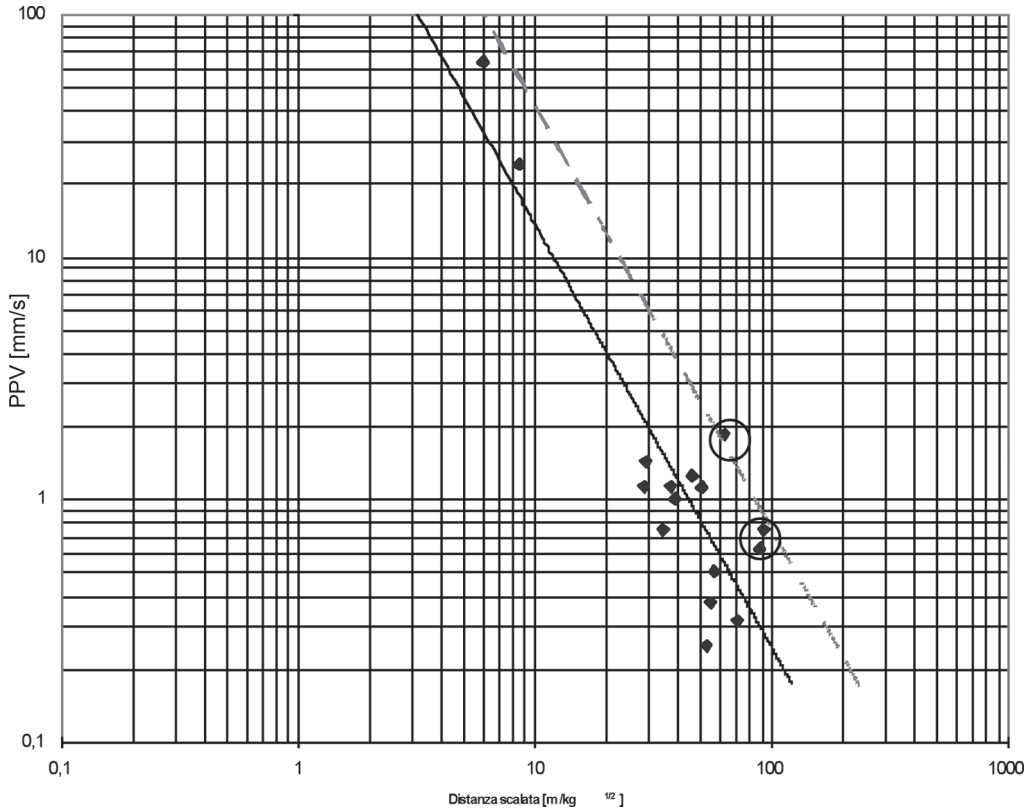

Figure 9. Conservative calculation of the maximum allowed cpd based on a 'safe envelope' encompassing the worst cases.

rock mass crossed. Therefore the p.p.v. limits to be adapted are due to be the lowest foreseen by the regulations.

6. RULES TO BE RESPECTED BY THE CONTRACTOR

Rules are to be given in a form easy to manage by the contractor. To this aim, having in mind that the objects to be protected are the ancient hermitage (p.p.v. limit 3 mm/s), the rock wall (p.p.v. limit 30 mm/s), the houses and some commercial building close to the shaft collar (for which limits respectively of 5 mm/s and 15 mm/s have been set), it seemed more practical to provide the prescriptions as maximum cpd distance graphs, for different tolerable p.p.v. values, than as p.p.v./ scaled distance correlations. The prescriptions are summarized in the graph of Figure 10.

In principle, for each round the distance to each protected object should be calculated, the max allowed cpd read and the lowest adopted to design the round. Actually, the excavation work will be divided in stretches, each one of them characterized by a different cpd limit value. The collar of the shaft will be sunk by mechanical excavation down to some 5 – 6 m, then by drilling and blasting, with cpd limits set in the upper part by the surrounding houses and by the rock wall, and by the hermitage in the lower part. For the tunnel too, the limit charge will be dictated by the rock wall in the portal area, and then by the hermitage.

Anyhow, limit cpd will be in the 0.5 – 1 kg range for a great part of the excavation work (at places, lower than 0.5 kg), and consequently the pull of the rounds very seldom can exceed 1 m (and at the cost of a very high specific drilling).

A permanent recording station will be installed in the frescoes hall to monitor the respect of the p.p.v. limit values during excavation.

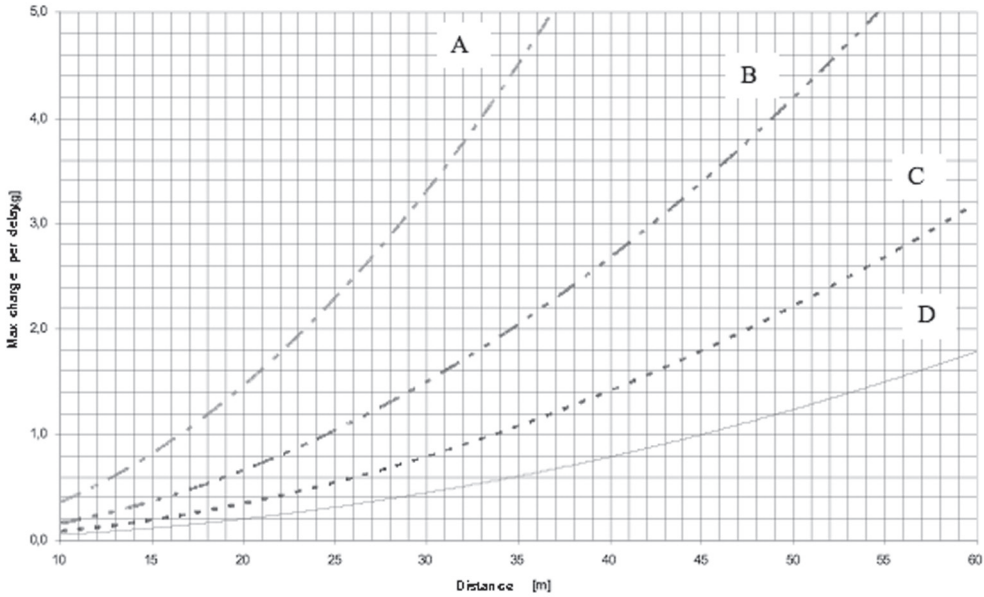

Figure 10. Max charge per delay – distance correlation, according to the different vibration limits proposed. A: ppv max vector sum = 30 mm/s (natural rock slope); B: ppv max vector sum = 15 mm/s (normal building); C: ppv max = 5 mm/s (delicate building); D: ppv max = 3 mm/s (historical building).

7. CONCLUSIONS

The operation has been recently contracted, the works however are still to be launched. The advisability to carry out a constant monitoring of vibrations during the excavation has to be underlined: it should continue up to the end of the works, to permanently evaluate the entity of the phenomenon and, if necessary, intervene on the excavation parameters, on the basis of the results emerged from a refinement of the site law, thanks to the great number of experimental points obtained.

Apart from vibrations by blasting, an evaluation of the background noise linked to the employment of mechanical excavation methods, should be useful, in order to opportunely intervene in case of anomalous troubles.

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Assessment of safe ground and structure vibrations from blasting

M.R. Svinkin
Vibraconsult, Cleveland, Ohio, USA

ABSTRACT: The most important question in assessment of blast effects to structures, people and sensitive devices is determination of the structural response that is the final destination of vibration measurement and control. Measuring ground vibrations is beneficial because it is a traditional procedure for assessment of vibration effects from blasting and such measurement is often the only choice in highly populated areas. However, measurement of ground vibrations does not take into account soil-structure interaction and susceptibility of structures. There is a growing need for measurement of the structural response as the criterion of blasting monitoring and control. The specified safe vibration limit for structural vibrations is considered. Each site is unique because of soil conditions and various structures at sites, but there are some general rules for assessment of vibration effects on structures.

1. INTRODUCTION

Blasting is widely used in quarry and mining production processes and also in construction and demolition operations. Blasting is the beneficial industrial technology which provides the achievement of expected results in a short period of time with relatively low cost, but on the other hand blasts produce undesirable vibrations and sounds. These potential problems can complicate the blast application in urban areas where it is important to establish safe vibration limits, properly assess structural vibrations from blasting, and determine how to prevent or mitigate blast effects on adjacent and remote structures.

For a specified location at some distance from blasting, vibration effects are the results of interaction of three major factors: blast design, geology, and structures. Only blast parameters can be modified to decrease ground and structural vibrations. Geology and the structure condition cannot be changed, but both of them influence the structure response. In assessment of blasting effects, it is essential to correctly determine responses of structures that are final destinations of vibration monitoring and control. Obviously, there is a need to increase involvement of structure vibration measurement in monitoring and control of vibration effects from blasting.

In light of liability of public fear of blasting in general and possible vibration damage lawsuits, private companies and government agencies are often specified to use expensive mechanical excavations instead of blasting. Therefore, it is necessary to emphasize that reliable vibration limits can provide the safety of blasting operations.

This paper shows that the existing vibration limits are very conservative for some cases and inadequate for others. The paper has proposed

the simplified safe vibration limit of structural vibrations and pointed out the necessity to measure structural vibrations from blasting.

2. DAMAGE TO STRUCTURES FROM BLASTING

The U.S. Bureau of Mines (USBM) accumulated results of structural responses and damage produced by ground vibrations from surface mine blasting are shown in Figure 1 which was modified from Siskind (2000). These results were obtained from 718 blasts and 233 documented observations of cracks. Non-damaging blasts are not shown in Figure 1 although some of them produced relatively high level even exceeding 51 mm/s (2 in./s) of ground vibrations.

Analysis of these data indicates different vibration effects on structures depending on the dominant frequency and the peak particle velocity (PPV) of ground vibrations, Svinkin (2004b, 2006a). Data of structural damage depicted in Figure 1 have been arranged in three zones depending on closely grouped damage results. Zone A represents cosmetic cracking (threshold) and other damage in the 3-35 Hz frequency range and in the 12 mm/s (0.5 in/s) - 762 mm/s (30 in/s) velocity range. Zones B and C show only minor and major damage to structures from blasting. For zone B, the frequency range changes from 2 to 5 Hz and the velocity range changes from 33 mm/s (1.3 in/s) to 191 mm/s (7.5 in/s). Zone C is located between 60-450 Hz and 102 mm/s (4 in/s) – 254 mm/s (10 in/s). The levels of vibrations in zones B and C are substantially higher than the USBM vibration limits.

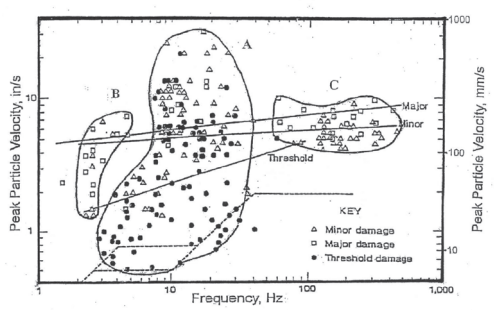

Figure 1. Three zones with closely grouped structure responses and damage summary from ground vibrations generated by blasting, and USBM RI 8507 recommended safe limits – dashed lines. Data were modified from Siskind (2000).

Ground vibrations from blasting may produce direct damage to houses and buildings, trigger structural and soil layer resonant vibrations, and be the cause of soil failure, Svinkin (2004b).

2.1 Direct structural damage

Soil-structure interaction determines the structural responses to ground vibrations that may produce direct damage to structures when frequencies of ground vibrations do not match natural frequencies of structures. This kind of damage might be predominant in zones B and C and partially reflected in zone A (Figure 1). Velocity values of ground vibrations should be substantial for direct structural damage. Thus, the velocity range is 33-191 mm/s (1.3-7.5 in/s) in zone B and 102-254 mm/s (4-10 in/s) in zone C.

It is necessary to point out that there are a number of case histories that demonstrate no structural damage at relatively small distances from blasts though velocities of ground vibrations were higher than the threshold levels for the wide frequency range.

2.2 Resonant structure vibrations

The proximity of the frequency of ground vibrations to one of the building's natural frequencies may generate the condition of resonance in the building. Amplitudes of ground vibrations are substantially less important then a coincidence of frequencies. Increasing of structure vibrations starts after the first cycle of ground vibrations with the dominant frequency near the natural structure frequency. For one-two stories residential houses, a dynamic magnifying factor at resonance was measured in limits of 2-9, Siskind (2000) and Quesne (2001). This factor can be substantially higher for steel and concrete structures. Resonant structural vibrations can be triggered at large distances even more than one mile from a construction site. Structure resonance of horizontal building and wall vibrations and also vertical floor vibrations can result in damage to structures. Resonant horizontal building and house vibrations occur most frequently. Resonant vertical floor vibrations are important when precise and sensitive devices and also fragile objects are installed on the floors. Resonant structural vibrations are predominant in zone A and partially presented in zone B (Figure 1).

2.3 *Resonance of soil layers*

Waves from dynamic sources travel through the soil medium, and they can be amplified as a result of resonance of soil layers. In most cases, analysis of site responses is focused on the motion at the free ground surface. However, resonant effects may occur at any point within a layered soil profile. It is possible to consider two locations with the same soil within the same site excited by the same dynamic source, and these locations could respond quite differently because of the nature and dimensions of surrounding soil layers, Davis and Berrill (1998).

2.4 *Soil failure*

Liquefaction and densification of soils can occur as a result of blast vibration effects. Non-uniform ground and foundation dynamic settlements can be triggered by relatively small ground vibrations in loose soils, and such settlements may happen beyond the zone of densification at various distances from blasting.

3. VIBRATION FEATURES AS DAMAGE CRITERIA

Complex ground and structure vibration responses to blasting create a problem of finding vibration criteria of structural damage. During decades of research performed for the blasting industry, numerous studies have been conducted to analyze vibration effects of blasting on residential houses and other structures in order to find single values of vibration features as the damage criteria for structures.

3.1 *Peak particle acceleration and displacement*

An acceleration of ground vibrations was initially used as a criterion of blasting damage similarly to assessment of structural damage from earthquake. However, this criterion is not satisfactory over a wide frequency range. Leet (1960) showed that the earthquake energy level is substantially higher than that produced by blasting. Nevertheless, acceleration and displacement are important criteria when blasting is carried out near structures with vibration sensitive medical or computerized devices and instruments, Boyle (1990).

3.2 *Energy ratio*

Crandell (1949) considered the energy of the ground vibrations measured near the structures and proposed the energy ration criteria defined as the ratio of the square of acceleration in ft/s^2 and the square of frequency in Hz. This ratio has the dimensions of velocity squared. Crandell showed that the energy ratio less than 3.0 can be used as the safe limit for plaster cracking and the ratio greater than 6.0 designates dangerous vibrations. Some local codes in USA use the energy ratio criteria.

3.3 *Peak particle velocity*

Extensive studies of vibrations from blasting performed by several researchers such as Langefors et al. (1958), Edwards and Northwood (1960), Duvall and Fogelson (1962), Nicholls et al. (1971) and others revealed that structural damage could be well correlated with the Peak Particle Velocity (PPV) of ground vibrations. The PPV is a measure of vibration intensity.

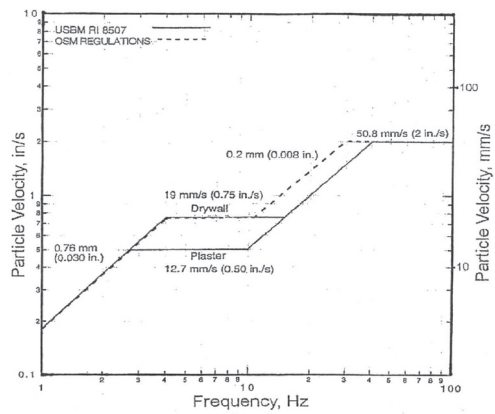

Figure 2. Safe level blasting criteria from USBM RI 8507 and the derivative version, the chart option from OSM surface coal mine regulations. Data were modified from Siskind (2000).

In other research studies, the PPV of structural vibrations were used as the criterion of structural damage from vibrations generated by blasts, Sadovskii (1946).

4. EXISTING VIBRATION REGULATIONS

For the present, there are no general regulations of ground and structure vibrations induced by blasting. Different approaches are put into practice for assessment of tolerable vibration limits.

4.1 USBM - OSM vibration limits

The frequency-based safe vibration limits for cosmetic cracks were developed for 1-2 story residential structures by the USBM – RI 8507, Siskind et al. (1980). These criteria are depicted in Figure 2 and they have the following displacement and velocity values for the four ranges of the dominant frequency: 0.76 mm (0.03 in.) for 1-4 Hz, 19 mm/s (0.75 in./s) for 4-13.7 Hz, 0.2 mm (0.008 in.) for 13.7-40 Hz, and 50.8 mm/s (2.0 in./s) for 40-100 Hz. A limit of 19 mm/s (0.75 in./s) for 4-13.7 Hz is used for drywall while the limit of 12.7 mm/s (0.5 in./s) for 2.5-10 Hz is applied for plaster. According to Dowding (1996), the frequency zones below 4 Hz and above 40 Hz are not well defined regarding the relationship between PPV and frequency and these zones are subjects of future research.

The Office of Surface Mining (OSM) included the derivative version of the USBM safe limits (Figure 2) as the Chart Option into the OSM surface coal mine regulations, OSM (1983). The USBM and OSM criteria for cosmetic cracking threshold developed for 1-2 story residential structures have been included in the Explosive Materials Code, NFPA 495 (2001).

The USBM and OSM criteria were built up on the basis of the two decades research studies of a correlation between ground vibrations and observations of cracking damage in low-rise houses, and these limits are applied for ground vibrations as the criteria of the possible crack formation in structure. Obviously, these vibration limits can be successfully used for similar structures and ground conditions they were developed for.

It is necessary to point out that on one hand these criteria are very restrictive for direct vibration effects on structures and on other hand they cannot protect low-rise structures from appearance of cosmetic cracks under amplification of ground vibrations higher than 4.5x and beyond the 4-12 Hz frequency range.

Other soil-structure interaction requires different vibration limits. For example, Siskind and Stagg (2000) analyzed results of ground vibrations and damage to low-rise residential houses at distances about 1.6 to 6.4 km (1 to 4 miles) from quarry blasting. The specific condition of wave propagation was found at that area: attenuation with distances was less as compared with other similar sites. As a result, the PPV of ground vibrations were relatively high for large distances and charge sizes used. Structural vibrations with duration of about 17 s were superposition of vibrations with various frequencies, including a low frequency of about 8 Hz, which was close to the natural frequency of horizontal house vibrations. Structure responses of 6.1x would produce a global wall strain which is 1.5 times higher than the strain limit for crack formation. Resonant structural vibrations were a possible cause of damage to house walls. On the basis of measured vibrations, the authors of the report have believed that the highest ground vibrations there would likely be under 3 mm/s (0.12 in./s). This criterion is about four times less than the smallest vibration limit of 13 mm/s (0.5 in./s) permitted by the USBM safe criteria.

4.2 British standard

The British Standard BS 7385 (1993) and the USBM criteria are depicted in Figure 3. It can be seen that the BS 7385 considers two lines of the safe limits depending on a type of buildings. Line 1 with the frequency-independent criterion of 51 mm/s (2 in/s) is applied to industrial and heavy commercial buildings. Line 2 with the frequency-based safe limits is similar to the USBM criteria and applied to residential or light commercial type buildings.

It seems the values presented in Figure 3 are small to represent structural vibrations. On the basis of similarity of two standards, it can be assumed that the BS 7385 requires measurement of ground vibrations as the criteria of possible structural damage. If so, the BS 7385 has similar restrictions of its application as the USBM criteria.

4.3 German standard

The German Standard DIN 4150 (1986) and the USBM criteria are depicted in Figure 4. DIN 4150 suggests three lines of time-dependent vibration limits for different types of structures. Line 1 is applied for buildings used for commercial purposes, industrial buildings and buildings of similar design. Line 2 is applied for dwellings and buildings of similar design and/or use. Line 3 is used for structures that because of their particular sensitivity to vibration, do not correspond to those listed in lines 1 and 2, and are of great intrinsic values (e.g. building that are under a preservation order). Lines 2 and 3 are located below the USBM vibration limits. Line 1 is closed to the upper boundary of the USBM vibration limits in the 4-100 Hz frequency range.

Comments are needed regarding structure particular sensitivity to vibrations. Such terminology is appropriate for devices and technology sensitive to vibration, but structure sensitivity is different. Resonant vibrations are dangerous for any kind of structures: from building under preservation order to industrial buildings. Dynamic settlements also can be devastated for any kind of buildings. However, industrial buildings have much higher resistance to direct vibration effects than 1-2 story residential houses.

The basis for the guideline level of DIN 4150 is unknown, AASHTO (1996). Obviously, these criteria intended to minimize human perceptions and complaints. This approach would be sensible if construction operations constantly affect houses and even at night. However, if blasts are performed 1-2 times per day, such an approach is unreasonable.

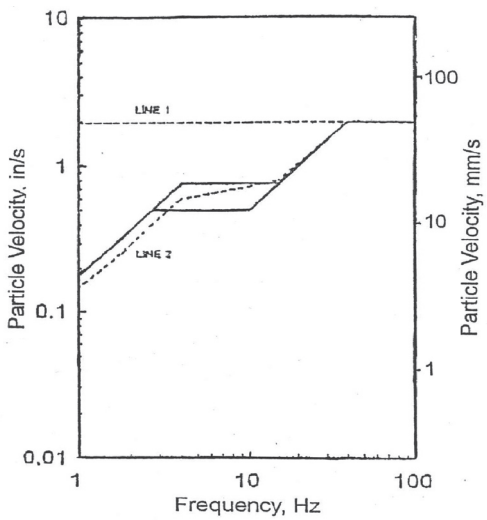

Figure 3. Vibration guidelines – USBM RI 8507 (solid line) compared to BS 7385 (dashed line). Line 1: reinforced or framed structures, industrial and heavy commercial buildings; line 2: un-reinforced or light frame structures, residential or light commercial type buildings. From AASHTO Designation R 8-96.

5. A SIMPLIFIED SAFE CRITERION

Ground vibrations are the major area of vibration measurements during blasting. Measurement of ground vibrations is beneficial for some reasons. First, it is a traditional procedure for assessment of vibration effects from blasting and the vibration measurement feedback is usually used for correction of a blast procedure; second, at the time of vibration monitoring, one transducer can be held at the same location on the ground and employed for the reference vibration record; third, it is beneficial for indication of possible resonance of soil layers; and forth, it is important for evaluation of dynamic settlement damage hazards. Measuring ground vibrations is often the only choice in a highly populated area.

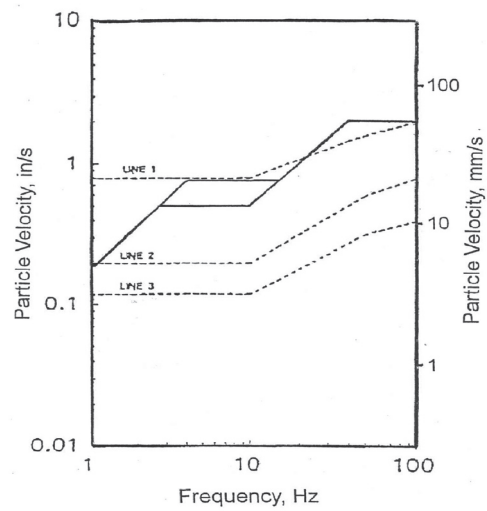

Figure 4. Vibration guidelines – USBM RI 8507 (solid line) compared to DIN 4150 (dashed line). Line 1: buildings used for commercial purposes, industrial buildings and buildings of similar design. Line 2: dwellings and buildings of similar design and/or use. Line 3: structures that because of their particular sensitivity to vibration do not correspond to those listed in loines 1 and 2, and are of great intrinsic values (e.g. buildings that are under a preservation order). From ASSHTO Designation: R 8-96.

Low-rise houses are the most typical structures in urban and rural areas and therefore such residential structures received a major attention during blast operations. At the same time blasting may affect various super and underground structures. According to the existing practice, safe limits are applied for ground vibrations as the criteria of possible structural damage, Siskind et al. (1980). The safe vibration criteria are used for indirect assessment of blasting vibration effects on different structures even though the USBM safe criteria are based on structural responses and

actual observations of cracking damage in 1-2 story residential houses. Data presented in Figure 1 indicate that threshold damage to structures could occur for in the wide 13 mm/s (0.5 in./s) - 279 mm/s (11 in/s) velocity range of ground vibrations. It means that for certain cases the cracking limit can be substantially higher than 13 mm/s (0.5 in./s). However, there are no studies of a correlation between ground vibrations from blasting and damage to the whole range of superstructures and underground structures undergoing vibrations.

In reality, various building or house structures and room contents will have different values of amplitude and frequency generated by the same ground vibrations. Besides, the existing criteria provide no distinction of type, age and stress history of structures and do not take into account building configuration. These are some of the major disadvantages of the criteria. Furthermore, the application of USBM-OSM limits to different super and underground structures is incorrect. AASHTO (1996) stated the application of the USBM limits to markedly different types of structures is common and inaccurate.

It is important to clarify what threshold of cosmetic cracking can be used for multi-story residential, commercial, and industrial buildings, facilities, mass concrete, buried pipes and structures, etc. Each site is unique because of soil conditions and various structures at sites. Therefore, specifications for blasting operations have to be specified at each site, and sometimes preliminary tests should be done to determine tolerable levels of structural vibrations. It is necessary to make direct measurement of structural vibrations accompanied by observation of the results of dynamic effects.

For multi-story residential, commercial and industrial buildings, the frequency-independent simplified safe limit of 51 mm/s (2 in/s) can be chosen for the PPV of structural, not ground, vibrations, Svinkin (2003, 2004b, 2006b). This criterion automatically takes into account soil-structure interaction for the whole building frequency range. The proposed criterion does not exclude higher allowable vibration levels. There are two reasons which confirm truthfulness and expediency of this criterion:

- First, in the middle of 1940s, the safe vibration limits of 30-50 mm/s (1.18-1.97 in/s) for sound structures were discovered at the Moscow Institute of Physics of the Earth, Sadovskii (1946). During the Second World War, these limits were found on the basis of analysis of structural responses to various types of blasts: in the air, on the ground, and under the ground. These limits were successfully used for years in former USSR
- Second, according to Siskind (2000), the PPV of 51 mm/s (2 in./s) is the highest vibration level generated inside homes by walking, jumping, slamming doors, etc. Without doubts, massive concrete structures have much greater cracking thresholds

It is easy to demonstrate compatibility of the new simplified safe criterion and the existing vibration limits. To evaluate tolerable structural vibrations, the smallest vibration limit of 13 mm/s (0.5 in./s) from the USBM vibration limits has to be multiplied by 4.5, and their product of 57 mm/s (2.25 in./s) is higher than the simplified criterion of 51 mm/s (2 in./s). It is important that the limit of 51 mm/s (2 in./s) for structural vibrations can be applied for assessment of vibration effects on 1-2 story houses as well. Furthermore, the simplified criterion is not contradictory to the British Standard because this standard is similar to the USBM vibration limits. The same conclusions can be drawn regarding the German Standard.

There are some general rules for measurement of structural vibrations. Horizontal building vibrations are most important for measurements. Therefore, transducers should be installed on a windowsill at the upper floor and a foundation for vibration measurements in the perpendicular direction to the building wall. Such measurement is necessary to detect resonant structural vibrations. Vertical floor vibrations should be measured when precise and sensitive devices and also fragile objects are installed on the floors. The pre-construction condition survey of structures surrounding a construction site may determine certain locations for vibration monitoring and control.

6. A NEED FOR MEASUREMENT OF STRUCTURAL VIBRATIONS

The structural response is an indication of vibration effects on structures, and there is a growing need for measurement of the structural response as the criterion of vibration monitoring and control. The Office of Surface Mining (USA) funded research

project for evaluation of structural responses to blasting, Crum (1997) and Aimone-Martin et al. (2003). Structure response measurements are the best way to show if a structure is typical of those studied by the USBM and others to develop the safe-level criteria, Siskind (2000). Measurements of structural vibrations were made in different projects for determining vibration effects on historic buildings, curing concrete, sensitive devices and equipment, e.g. Elliott and Goumans (2004), Lucca (2004), and Svinkin (2004a and 2006b).

Furthermore, direct measurement of structural responses to ground vibrations from blasting is more accurate and has the substantial advantages in comparison with ground vibration measurement because structure responses ensure the flexibility of implicitly considering the variety of soil deposits and the condition of structures. Such an approach provides measurement of structural response after soil-structure interaction, takes into account distinction of type, configuration, age and stress history of structures in the wide frequency range. As a result of these advantages, tolerable limits of structural vibrations can be substantially increased.

Vibration limits for blasting within or adjacent to old mass concrete have a special interest because the concrete is a material with high resistance to impact loads. According to Siskind (2000), the PPV of 254 mm/s (10 in/s) of ground vibrations is the threshold for cracking of mass concrete. On the other hand Oriard (2002) showed in one case history that the 100 year old concrete received high-frequency vibration intensities in the range of 1000-1150 mm/s (40-45 in/s) on a top of the concrete wall and much higher PPVs at rock-concrete contacts and at mass concrete blasting perimeters. Oriard (2000) has warned that the experiences for blasting within concrete and nearby rock are different, and vibration criteria used for high-frequency intensities from close-in blasts cannot be applied for lower frequency vibrations at remote locations. At the same time, it necessary to point out that the specified vibration limit of 51 mm/s (2 in/s) for structural vibrations presented in this paper can be applied for lower frequency vibrations at remote locations.

Oriard (2002) has also underlined some of the lessons learned from case histories: high particle velocities are a common occurrence on construction sites and those concrete structures, even old ones, routinely tolerate high vibration levels without incurring damage; restrictive vibration limits are not compatible with high rates of production and project designers must select an appropriate compromise between the two.

7. CRITERIA OF SOIL FAILURE

It is necessary to point out that all mentioned above vibration limits can be used for assessment of direct vibration effects on structures and resonant structural vibrations. However, such limits have nothing to do at all with soil failure. There are no federal, state, and local regulations of the critical vibration levels of ground vibrations which may trigger dynamic settlements as results of blast liquefaction and densification.

Liquefaction can occur in saturated sands, silts, and quick clays and also in dry and loose sands and silts, Poulos et al. (1985). Some blasting studies have presented the PPV values as the blasting vibration threshold for liquefaction in sand soils. Charlie et al. (1992) reported the results of a series of different explosive charges detonated at a depth of 3 m in a dense, saturated, alluvial sand deposit. Liquefaction was induced at the depth of explosives locations for the PPV that exceeded 160 mm/s (6.3 in/s). In another case history performed as a part of the Canadian Liquefaction Experiment, single and multiple explosive charges were detonated in mine tailings consisted of loose saturated sands, Al-Qasimi et al. (2005). It was found that the condition of zero effective stress was induced at a depth of 6 m when the PPV exceeded 650 mm/s for single detonation and 130 mm/s for twelve detonations with millisecond delay.

Blasting densification is used for improving loose and saturated sands to receive satisfied soil conditions. Initiation sequences are important for the control of vibration effects on adjacent structures. There is a procedure to calculate a maximum radius of ground surface settlements greater than 1 cm, Dowding (1996). Densification of sands is expected at short distances from blasting, but surface settlements extend beyond the zone of densification for account of joint influence of static and dynamic loads.

Ground and foundation settlements as a result of relatively small ground vibrations in loose sands may occur at various distances from the source. Unlike liquefaction, dynamic settlements from densification can be triggered by small ground

vibrations. Unfortunately, there are only two documented cases of displaying vibratory settlement in the specified types of sand soils. Lacy and Gould (1985) found that the peak particle velocity of 2.5 mm/s could be considered as the threshold of possible significant settlements at sites with narrow-graded single sized clean sand deposits. Claugh and Chameau (1980) have revealed that acceleration higher than 0.05 g can trigger dynamic settlement in loose sands with rubble and broken rock.

8. CONCLUSIONS

The existing vibration criteria should be used as guidelines, and structure vibration responses have to be taken into account in a choice of the safe vibration limits.

Sensible choice should be provided between measurement of ground and structural vibrations. The most important question in assessment of blasting effects is determination of the structural response that is a final goal of vibration monitoring and control. There is a growing need for measurement of structural vibrations. For multi-story residential, commercial and industrial buildings, the frequency-independent safe limit of 51 mm/s (2 in/s) can be chosen for the PPV of structural vibrations. This simplified criterion provides a reasonable indication of structural damage from direct vibration effects and resonant vibrations.

Measurement of structural vibrations accompanied by observation of the results of dynamic effects is beneficial for determination of vibration limits which can combine production blasts with prevention of cosmetic cracking and other damage to structures.

Despite of the potential economic and time saving advantages, some owners and contractors try to avoid blasting because of fear of alleged claims and high costs of settlements and attorney assistance. Well founded vibration limits of structural vibrations can help in resolving this problem.

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Blast induced micrometer response of cracks in concrete block structures

C. H. Dowding
Professor of Civil and Environmental Engineering, Northwestern University

ABSTRACT: Blasting and climatological response of cracks are compared for structures constructed of concrete block typical of home construction in Europe. While much has been written comparing micrometer crack response of wood frame structures typical of residential construction in North America, little has been written concerning response of structures more typical of European construction. Responses of cracks in four different structures are compared with the following locations and (wall materials): Wales, UK (Concrete Block), Florida, USA (Concrete Block – 2 cases), Wisconsin, USA (Concrete Block and Stone). Time histories of micrometer response of cracks to both blast induced dynamic effects as well as long term climatological effects are compared. These comparisons show that even for concrete block construction without wood, climatological and home heating effects have greater influence on crack response than blast induced ground motions with a wide range in intensity.

1. INTRODUCTION

This paper compares vibration and climatological crack & structural response of four residential structures with exterior walls constructed of concrete blocks. These cases are important because they allow a determination of the degree to which cracks in concrete block structures respond to climatological effects. It has been surmised that construction of homes with concrete blocks are less prone to climatological crack response than homes constructed of wooden frames, which are subject to shrinkage and swelling of the wood frame from changes in humidity. The four cases involve two in Florida, one in Wales and another in Wisconsin. Vibration and climatological crack response was measured in the Floridian and Wisconsin homes. While vibration crack response of cracks was not measured in the Welsh case history, before and after blast measurement of the crack opening allows an indirect conclusion concerning long term change in crack width. In this paper each structure is introduced in Figure 1. Each case study is then described in detail. After the four cases have been introduced, they are compared to evaluate differences and similarities in the responses of cracks to vibratory and long term climatological effects.

2. MIAMI, FLORIDA; (LOUIS 2000)

The concrete masonry unit (CMU) house shown in the top left in Figure 1 was located on mine property and thus could be subjected to ground motions that exceeded normal control limits. Its

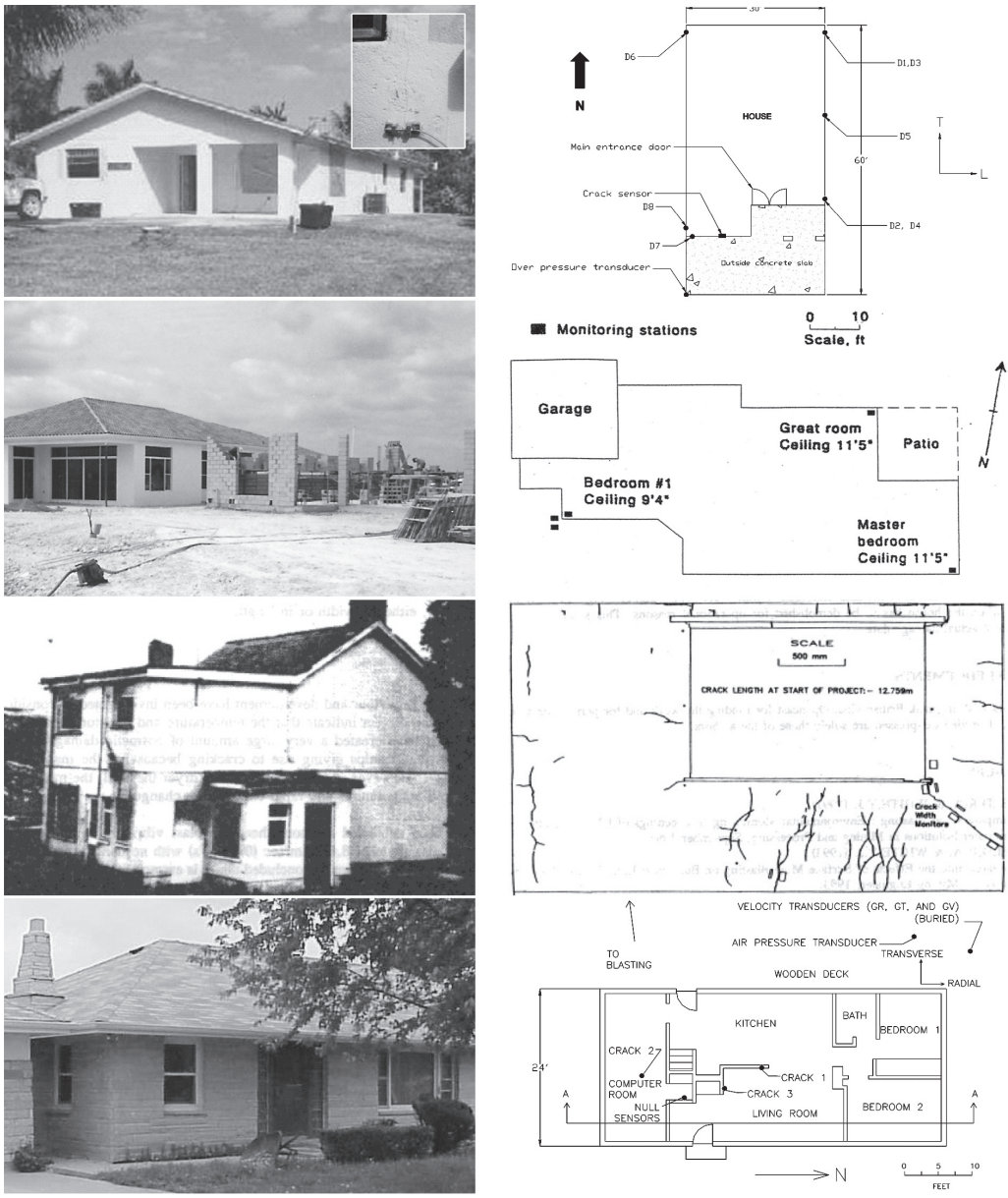

Figure 1. Photographs (left) and crack locations (right) of block structures from (beginning at the top) Miami Florida; Ft Lauderdale, FL; Gilfach Iago (From White et al. 1993), Wales, UK, and Franklin, WI.

response was monitored with a hybrid autonomous crack monitoring system (ACM), as well as two multi channel vibration monitoring systems, both of which were constructed by GeoSonics. Like the hybrid described in the 2001 surface coal mining studies (Aimone-Martin et al. 2003) this combination recorded vibratory time histories of crack & structure response, ground motions and air over pressures as well as well as long term, climatological crack response.

This single story ranch house was constructed with external concrete CMU walls and was founded

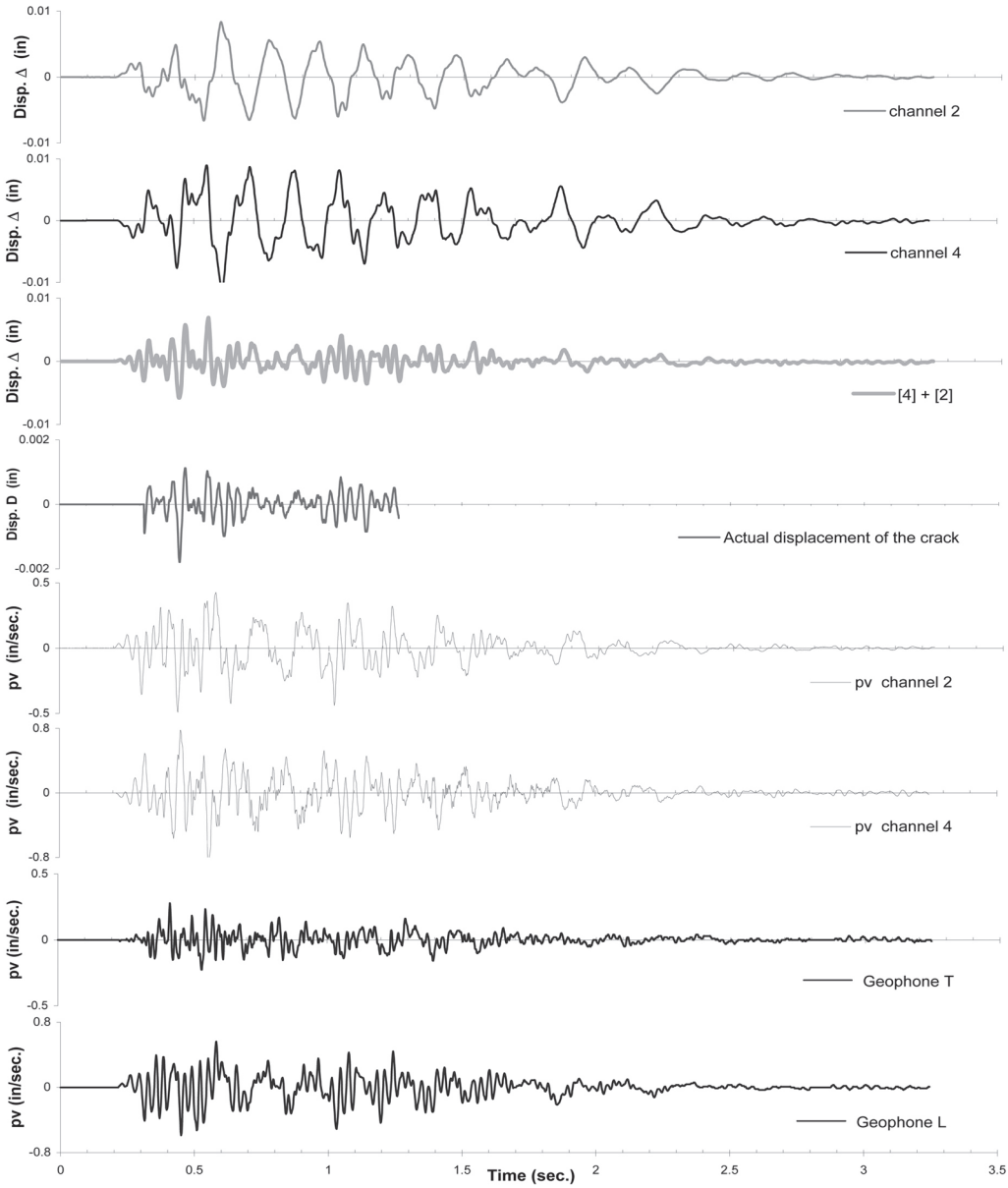

Figure 2. Time histories produced by 17.8 mm/s (0.7 ips) Miami FL blast (from top to bottom) A) structural displacement response (channel 2 & 4) and their difference (4+2) compared to crack response (actual…); B) structural velocity response (pv ch. 2 & 4) compared to ground motions (Geophone T & L).

on a concrete slab as are almost all homes in southern Florida. The external CMU walls were covered with a coat of stucco to produce a uniform surface. Internal walls, ceiling joists and roof trusses were constructed of wood. Internal walls were covered with gypsum drywall. Locations of the transducers are shown in the plan view of the house on the top right in Figure 1. An external crack in the stucco over CMU below the south facing window shown in the inset in the house photograph

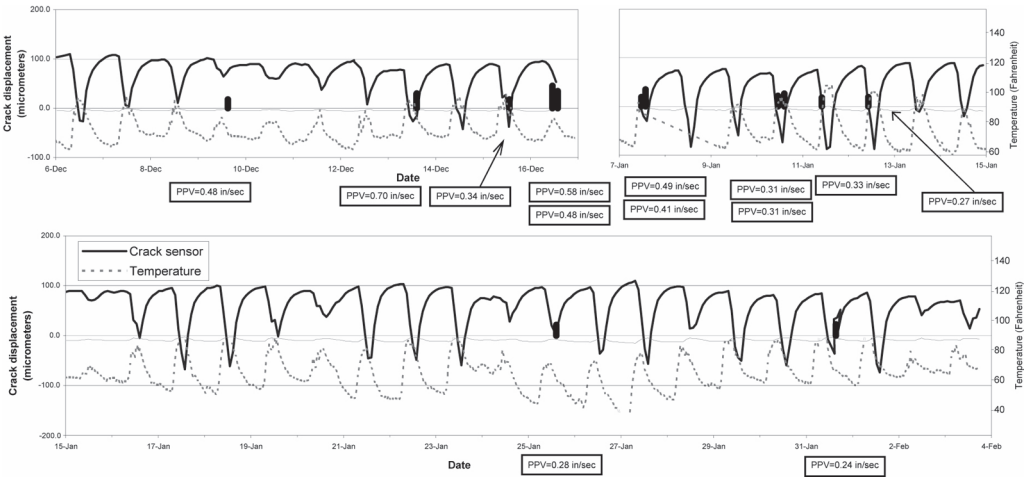

Figure 3. Comparison of vibratory crack responses (solid vertical bars) with cyclic long term responses (solid line) shows that even motions exceeding regulatory limits cannot produce crack responses as large as those produced by temperature change. Comparison of long term response with temperature variation (dotted line) shows the high correlation with temperature change.

was monitored with a Kaman eddy current micro meter displacement sensor. Velocity transducers 2 and 4 were attached perpendicularly (low and high respectively) to the N-S wall in the SE corner of the structure, as shown in the house plan in Figure 1 (top right). Thus they measure super structure motions in a direction parallel to the wall containing the crack.

Crack and wall motions in the plane of (parallel to) the wall containing the crack for the 16 December event are shown in Figure 2. Measured crack response, the 4th time history from the top of the figure correlates well in terms of frequency content with the calculated relative displacement between the top (channel 4; D4) and bottom (channel 2; D2) shown in the time history third from the top.

Climatologically induced crack response from mid November to early February in Figure 3 shows the heavy influence of the direct solar heating on the south facing wall. This graph compares blast-induced crack response (solid vertical bars) with the long term time history of climatologically induced crack response (thin horizontal solid line). The long term crack response pattern can be compared with the variation of external temperature (dotted line) as well as the response of the null sensor (vanishingly thin solid line). The null sensor is another Kaman sensor placed across an adjacent, uncracked section of the south wall to measure sensor and wall material response. Despite the high degree of solar heating, the null sensor response was small (compared to temperature and crack response). Comparison of the time history of the crack response with that of the temperature in Figure 3 shows less crack response on days with more constant temperatures, which is most likely a result of cloudiness and lower temperature induced strains from direct sun exposure (insolation) on the south facing wall containing the crack.

Close-by quarry blasting produced ground motions with peak particle velocities (PPV) as high as 17.8 mm/s (0.7 ips) in the direction parallel to the wall containing the crack as shown by the PPV values in the boxes in Figure 3. Peak crack responses to these events are shown as bars, which extend upward from the 0.0 crack displacement line, whose height is proportional to the 0 to peak crack response. This comparison shows that even motions that are 140% of the frequency independent control limit of 12.7 mm/s in the United States were unable to produce crack responses greater than 0.61 (61%) of those induced by typical daily temperature changes.

3. FT. LAUDERDALE, FLORIDA; (SISKI-ND ET AL. 1996)

A single story structure like that in the photograph

second from the top in Figure 1 was constructed with external CMU walls, and was founded on a concrete slab like that in Miami. Details of the block construction can be seen on the right side of the same photograph. Critical corners of the outer walls are constructed of cast-in-place reinforced columns and a continuous reinforced concrete beam is poured on top of the outer walls. Interior walls, ceiling joists and roof framing are constructed of wood or equivalent steel 'C' steel members. The exterior walls and surfaces are covered with stucco to produce a uniform surface.

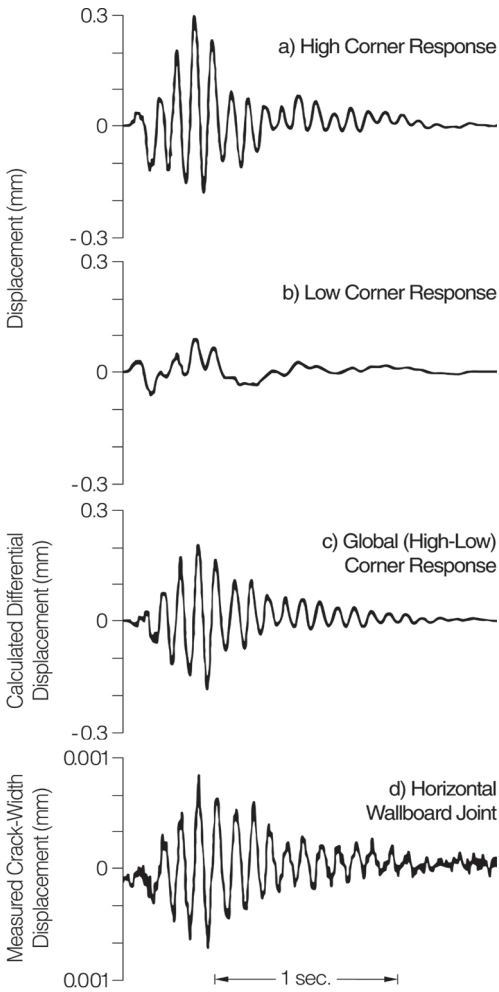

Figure 4. Time histories produced by 11 mm/s (0.43 ips) Ft Lauderdale blast (from top to bottom of Figure 2): structural displacement response (upper -a- and lower -b- corner) and their difference (c, high minus low) compared to the response of dry wall joint (d - bottom).

Kaman crack sensors were placed across an interior, taped dry wall joint on the north wall and on the exterior patio ceiling across an exterior stucco crack between two metal mesh sheets on which the stucco was applied. Geometry and sensor (monitor) locations for this structure are shown in the plan to the right of the photograph in Figure1. Taped drywall joints are interfaces between adjacent sheets of drywall that are spanned only by paper tape covered with a 1 to 2 mm thick plaster coat to smooth the interface. Field measurements verify that the joints are more responsive than wall materials (Dowding 1996). Velocity transducers were placed at the upper and lower corners of the NE and SW corners to measure relative displacement of the walls.

Structural response time histories from the shot producing the highest PPVs, 11 mm/s (0.43 ips), are compared to the time history of the taped horizontal dry wall joint in Figure 4. Response of the horizontal wallboard joint (bottom of figure) on the north wall is compared with the north-south 'displacement' responses at the high and low corners (top of figure) as well as the relative displacement between bottom and velocity transducers. This differential displacement produces a relative displacement time history that matches that of the drywall joint.

Climatological joint response was difficult to capture with this instrument array as it could not be operated autonomously and required human intervention. As a result, climatological response of the two monitored joints was only obtained for a single, two-day period. During that two-day interval the interior dry wall joint and exterior stucco crack displaced 11 and 18 μm (440 & 720 μ in.) respectively with only a 5 ºF (3 ºC) change in exterior temperature. By linear extrapolation, during a normal daily temperature change of 20 ºF (11 ºC), the interior joint and exterior crack would change width by 44 and 72 μm.

Blasting in this case study was needed to simultaneously 1) excavate poorly cemented clastic limestone for foundation pads for the homes to form 2) adjacent lakes. This activity produced PPVs that ranged from 2.3 mm/s sec (0.09 ips) at a 50 Hz dominant frequency to 11 mm/s (0.44 ips) at a 20 Hz dominant frequency. These motions produced maximum tape joint and stucco crack responses of 10 μm (40 μ in.). Thus ground motions nearly equal to the frequency independent control limit of 12.7 mm/s would produce crack responses only a quarter

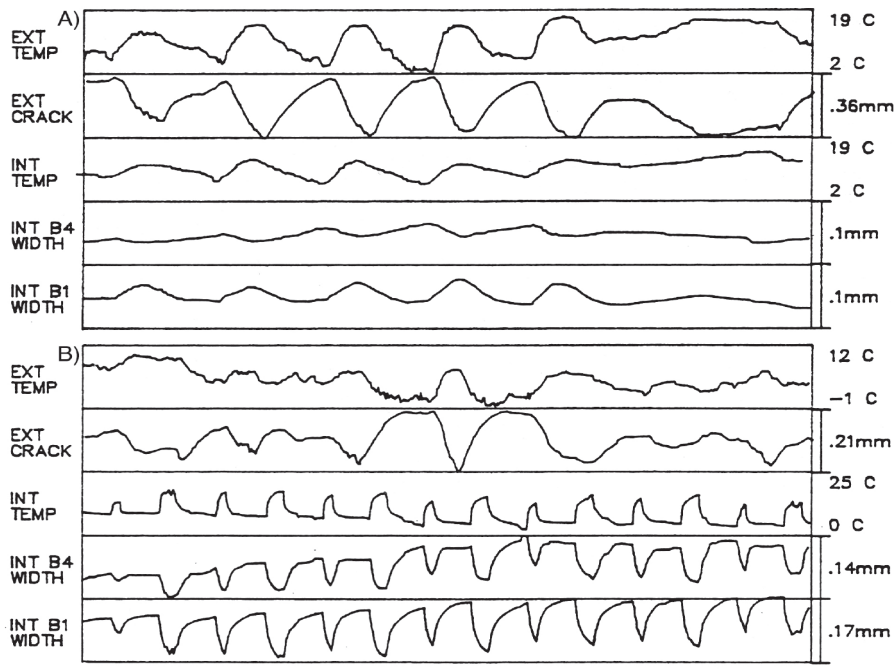

Figure 5. One week's worth of long term change in width of 3 cracks compared with temperature for the Wales house similar to that shown in Figure 3 for the Miami, FL house. Response with no internal heat at the top A) and that with internal heat twice a day at bottom, B). Crack B1 is in an external block wall (White et al, 1993a).

that might be produced by changing temperature during a typical day – on the north and shady side of the house.

4. GILFACH IAGO, WALES, UK; (WHITE ET AL. 1993 A&B)

The two-story house second from the bottom in Figure 1 was fitted with LVDTs and a continuous blast monitoring system (White et al. 1993a) to record ground motions and structural response generated by near-by surface coal mine blasting. Vibration and LVDT responses were measured separately and thus the system differs from an Autonomous Crack Monitoring system; however the LVDT measurement of crack response can be employed for comparison with the other case studies to assess possible similarities and differences.

Cracks were fitted with LVDTs in two rooms in the newer rear extension of the elderly stone cottage because of the poor condition of the original portion. The extension was a "double skin concrete block construction with external rendering and internal plaster" (White et al. 1993). Internal walls were constructed of 'plaster board'. External walls were founded upon a trench filled with concrete and brick. Lack of a structural connection between the two portions exacerbated differential movement and resulted in a large, external active crack between the extension and original structure.

Typical long term crack responses are compared for a week with external and internal room temperatures in Figure 5a and b with and without active room heaters respectively. Response of three cracks is compared: one external crack and two internal. The external crack was that between the old and new portions. Internal crack B 1 is on an external block wall and internal crack B 4 is on an internal 'plasterboard' wall. Wall B 1 is shown in on the right hand side of Figure 1 (second from the bottom), with its cracked condition along with the location of the crack monitors. The crack at the corner of the window extended into the concrete block. There is no description of the frame for the

internal wall, which is likely to be wood given the presence of plasterboard, and is assumed to be similar to gypsum dry wall in the US.

Interior heating affects interior cracks on both plasterboard interior and block exterior walls. Without heaters response of the external crack between the addition and original structure was absolutely large and larger than that of the internal cracks showing a daily peak to peak response of 0.36 mm or 360 μm (14,400 μ in.) The cracks within the addition responded far less and more typically: 27 and 45 μm for the plasterboard and block walls respectively. Operation of the heaters twice per day changed the periodicity of responses of the internal cracks to match the heater activity but not that for the crack between the new and old portions. In addition the larger temperature swings produced by the heater induced larger responses of the cracks inside the addition. The large daily swings of the crack response between the two sections of the structure are probably the result of numerous differences in the structures; foundation depth, materials, framing, etc.

LVDT readings were obtained every 20 minutes and immediately after each coal mine blast and other dynamic events. This surveillance scheme is similar to modified Level I operation of ACM, where long term climatological response is measured, but dynamic response is not. However, changes from the last reading (as recent as 20 minute ago) could be measured. During surveillance over 600 blasts occurred and their effect is summarized in White et al. (1993b). During this period of observation 'the lowest PPV [peak particle velocity] blast which almost certainly gave rise to damage was 24.1 mm/s despite the fact that the house was in poor condition'. A change in the width of a crack in plaster board (drywall) above a door in wall B4 was produced by slamming the door below.

5. FRANKLIN, WISCONSIN; (LOUIS 2000, MCKENNA 2002)

As shown in the photograph at the bottom of Figure 1, this ranch style house's exterior load bearing walls are constructed of stone cemented to a concrete masonry load bearing wall. Interior walls are constructed with typical wood framing. The interior surfaces of the exterior walls are covered with gypsum drywall. A full CMU basement, with a walkout entrance, serves as the foundation.

Kaman crack sensors have been placed across the three interior cracks located in the plan view on the bottom of the right hand side of Figure 1. Cracks 1 and 3 were located at the corners of openings between rooms, with that between the living room and kitchen (beneath crack 1) some 3 doors wide. The ceiling crack, 2, is in the center of the study where a ranch style house's typical center wall support would be located.

Discussion in this article will focus on crack 3 as it responded most to lower frequency excitation. Velocity response of an exterior wall was not measured as it was for the other three case studies. Instead the dynamic response of the crack was employed to estimate the natural frequency of the superstructure. Unfortunately, space does not permit presentation of the vibratory and long term climatological response time histories. They can be found at the ACM web page at www.iti.northwestern.edu/ACM in the pdf version of the theses by either Louis (2000) or McKenna (2002).

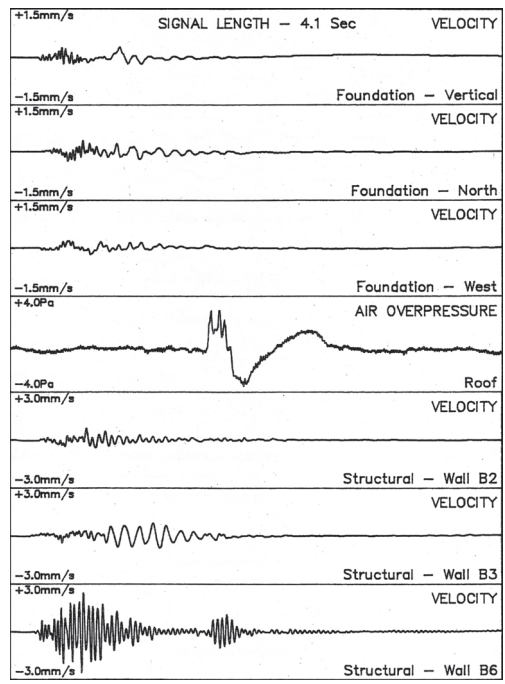

Figure 6. Comparison of typical low level velocity excitation (top four plots) and wall response time histories (bottom three plots) at Gilfach Iago reveal a variety of wall responses (White et al. 1993a).

Surface quarry blasting some 450 m (1500 ft) away, produced peak particle velocities of 0.8 to 3.3 mm/s (0.03 to 0.13 ips) in the direction of the wall containing crack 3 during the two month period of observation summarized by Louis (2000). The largest of these motions induced maximum crack responses of 9 μm, which were only 40% of those induced during intensive heating in November. A full year of the observation of the three cracks shows that through one season the cracks opened and/or closed some 300 μm (Dowding, 2007).

6. COMPARISON OF RESPONSES OF BLOCK STRUCTURES

It has been surmised that research on residential structures in the United States constructed of wooden frames is not directly applicable to structures constructed of cement blocks as in Europe. Responses of block structures presented in this paper can be employed to investigate differences that may exist. In addition, response of block structures reported herein can be compared to other case studies of wood frame residential structures, which are more typical of residential structures in the US.

Table 1 compares the attributes of the four case histories in this paper to investigate similarities and differences between the response properties of the Welsh and US houses all of which are constructed of block walls. This table contains comparisons of dynamic response properties, dynamic crack responses and long term, climatological crack responses.

Dynamic structural response properties in Table 1 were obtained from free vibration response like that at the ends of response time histories like that in Figures 2 and 6. Estimates of natural frequency and damping from time history data (some of which is presented here) in Table 1 show that the single story Florida structures have natural frequencies that bracket (7 – 10 Hz) that for the Welsh structure (9 Hz). Since one would expect the natural frequency of a 2-story structure to be less than that of a single story structure, this might imply that the Welsh 2 story structure is slightly stiffer than a typical US structure. However, it appears that responses of the Welsh structure were measured at the mid wall rather than the corners and thus reflected more wall-like responses. Therefore more response motions at the corners of European block structures are needed. Similar calculations can be made to determine damping constants, which are 5 and 6% for the Floridian and Welsh structures respectively. Thus they have similar damping constants.

Dynamic crack response comparisons are more difficult to make, since the crack response data for the Welsh house were collected before it was possible to autonomously collect dynamic crack response. However, White et al. (1993b) indicate that no permanent crack offsets from blasting alone were observed in the Welsh case despite excitation with PPVs up to 9 mm/s during the low level excitation phase and 24.1 ips during the intense, close-in phase. This observation is consistent with observations of structures in the US.

Finally and most importantly the response of cracks to climatological effects is compared on the far right of Table 1. It appears that crack response is similar for the Welsh and Floridian homes. First compare responses of cracks on exterior block walls: the Miami exterior crack and the interior Welsh crack on wall B1. The crack on wall B1 penetrated the entire wall thickness and thus its behavior reflects behavior of the exterior wall, not just the interior surface coat. If the maximum daily peak to peak (p to p) crack responses shown in Figures 3 & 5 are divided by the p to p changes in temperature, the temperature sensitivity is 2.9 to 10.6 μm/°C for the B1 crack and 9 μm/°C for the Miami crack. The range in the Welsh crack response is a function of the heat source. The heated test for the Welsh crack employed local heat blowers and the Miami crack was affected by direct sunshine heating. Thus both involved localized heat sources. The Ft Lauderdale crack response was that of a stucco ceiling crack between two sheets of wire mesh supports, which does not involve localized heating.

Crack response is compared to peak particle velocity in the direction of the wall containing the crack in Figure 7 for the Milwaukee and Miami cases. As can be seen, the Milwaukee case involved typical ground motions near quarries (0.8 to 3.3 mm/s). On the other hand the Miami cases involved unusually high PPVs 17.5 mm/s because it was owned by the quarry and was located on their property. There exists a rough (visual or optical) correlation between PPV and crack response (left graphs), but since excitation frequency and time history (number of pulses, exact timing, etc) are not included in the PPV magnitude, the correlation coefficient (R) can be low. In this case correlation

Table 1. Comparison of block structures and their response to climatological and blast effects.

						Max Crack Response			Crack Temp Sensitivity	
Case Study	Building	Natural Frequency	Damping	Excitation Frequency	Particle Velocity	Crack Type	Blasting p to p	Daily p to p	Δ Temp Daily	Δ Crack / Δ Temp
		(Hz)	(%)	(Hz)	(mm/s)		(μm)	(μm)	(°C)	(μm/°C)
Miami, FL	1 Story SOG	7	5	6 to 38	4 to 18	Block Wall Exterior	46.0	150.0	16.6	9.0
Ft. Lauderdale, FL	1 Story SOG	10		10 to 60	2 to 11	Stucco Ceiling Exterior	10.0	72.0	11.1	6.4
Wales	2 Story SOG	9	6	10 to 33	1 to 9 R	Block Wall Interior	NR	36.0 120.0	12.3[1] 11.3[2]	2.9[1] 10.6[2]
Milwaukee, WI (#3)[3]	1+ Story BMT	~9		6 to 39	1 o 3	Wall Board Interior	9.0	12.0		

SOG = Slab On Grade, Block = Concrete Block, BMT = Basement, R = Resultant

[1] Unheated [2] Heated [3] Louis, 2000

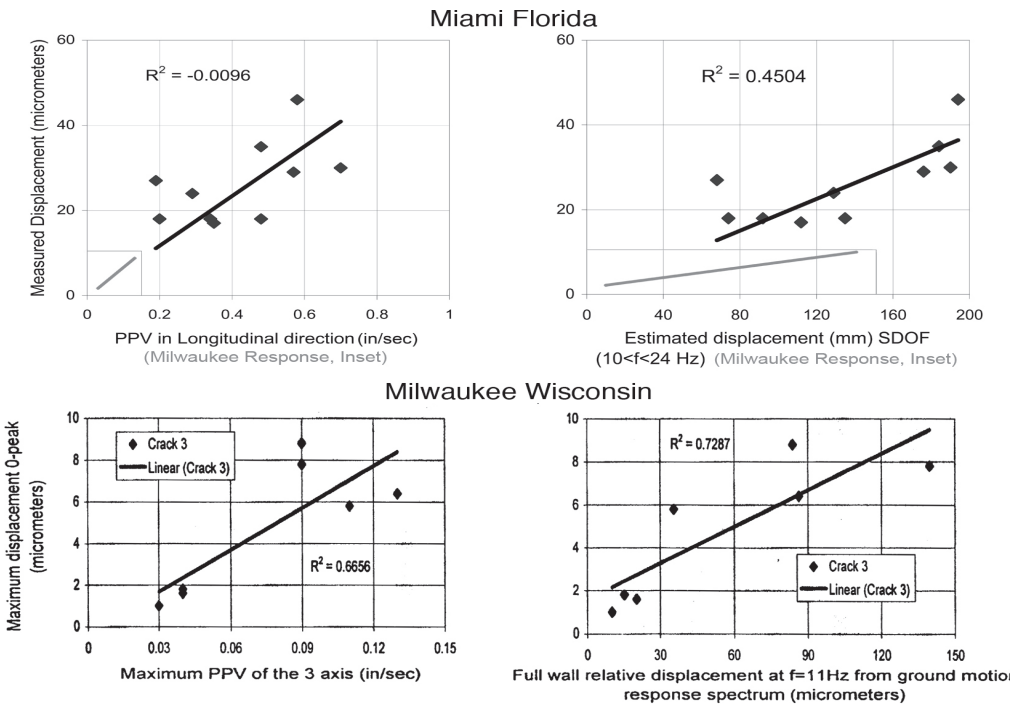

Figure 7. Comparison of crack response with peak particle velocity PPV (on left) and single degree of freedom response SDOF (on right) for the Miami case (top) and the Milwaukee case (bottom) shows rough proportionality with PPV but a higher correlation with SDOF response because of the inclusion of excitation frequency. (PPV in ips where 1 ips = 25.4 mm/s).

can be improved through comparison of crack response with the single degree of freedom response spectrum of the ground motions as shown in the right hand graphs (Dowding & McKenna 2005). The smaller Milwaukee excitation and responses are included in the Miami graphs in the insets at the lower left of both Miami graphs.

Comparison of crack response in a range of structures and building types reveals that for each crack there is a rough proportionality between ground motion descriptors and dynamic crack response. However it is different for each crack. This variation is logical since cracks are the result of structural details, and will vary from structure to structure and from cracking mechanism to cracking mechanism. Examples of this variation in the ra¬tio of PPV with ground motion descriptor can be seen in Dowding & McKenna (2005).

7. CONCLUSIONS

- Cracks in concrete block structures do respond to changes in temperature
- Temperature induced crack responses of block structures are large, some 3 to 15 times greater than response to ground motions with PPVs of 3 mm/s that typically are of concern to occupants
- Thermal response sensitivity of cracks in external block walls is similar for both the Welsh and Floridian houses

8. ACKNOWLEDGEMENTS

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Comparison of different methods of measuring and calculating blast vibrations in rock masses

B. Müller, J. Hausmann & H. Niedzwiedz
Movement and Blasting Consulting, Leipzig, Germany

ABSTRACT: On the one hand the measurement instruments for peak particle velocity recording with referring frequency as well as the required evaluation software have advanced significantly in recent years. On the other hand different evaluation methods of charge weight-distance relations were presented in the technical literature and engineer standards, which use various factors, e.g.:

- charge weight per delay (kg per delay)
- scaled distance (kg/m^n)
- specific consumption of explosives (kg/m^3)
- SRSD-square-root-scaled-distance ($m/kg^{1/2}$)
- superpositioning-model
- momentum model
- variable statistic evaluation methods

In spite of all positive efforts to enhance the vibration measurements and the charge weight-distance relations in regard to their declaration and forecast precision, the following findings remain:

- The recorded peak particle velocities and frequencies are surrogates for the vibration evaluation in each kind of construction
- As a result of the missing interpretation possibilities of peak particle velocity data with regard to fissure appearance in constructions there are no standardized reference values nor limits in the vibration engineer standards of the different countries

Therefore a systematic, critical comparison of the single proceedings of formulating charge weight-distance relations is taken in order to enhance the precision and statistical reliability of the calculation method and the vibration forecast respectively. For that purpose the authoritative physically proved blasting technical, geometric and underground conditioned influencing variables are shown. In this context systematic measurement errors, which occur by using geophones, are presented and a required method for the use of statistically proved data is recommended. As a result of the comparisons a modern vibration forecast is described, which exhibits a maintainable data variance.

The recording of strain-states during vibration influence is an alternative to peak particle velocity measurements. The stress-strain-state measurement under dynamic influences is a direct variable for the calculation of the created tensions compared to the peak particle velocity. Strain measurements are more precise than peak particle velocity measurements and can be interpreted more accurately in order to evaluate possible fissuring in constructions. Six years of experience in doing systematic strain measurements under different blasting technical conditions will be presented. A linear relation exists between the peak particle velocity and the strain value related to the measurement point. Finally, a consistent recommendation of permitted reference values and limits for blast vibrations in buildings and structures basing on classified construction types, the building ground as well as the measurerands strain and peak particle velocity will be declaimed.

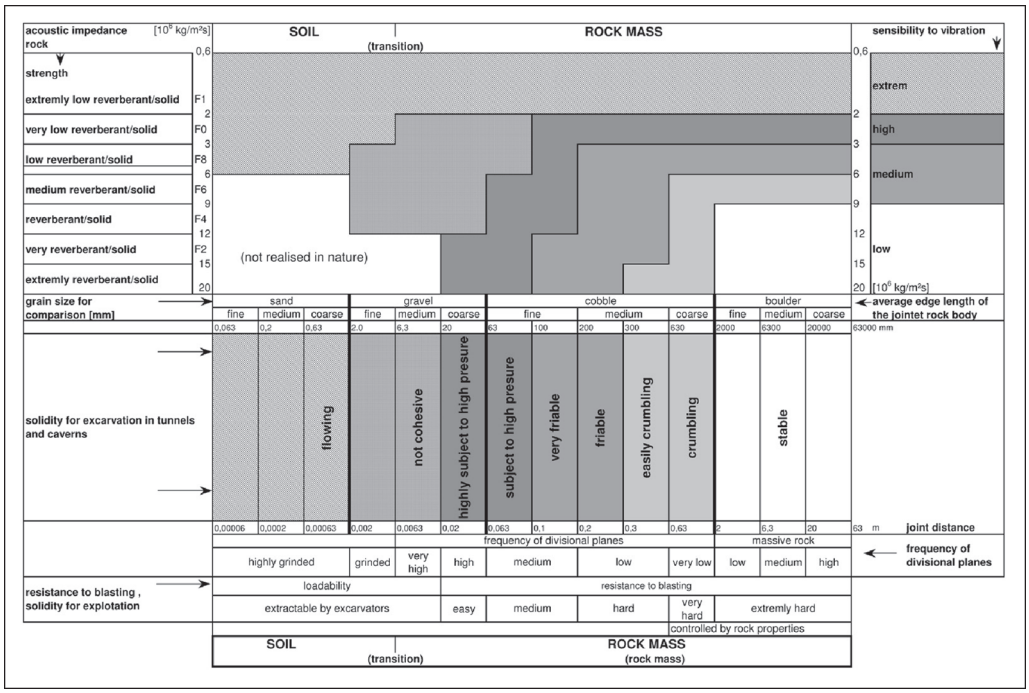

Figure 1. Recommended classification of rocks and rock masses for objective graduation of the resistance to blasting (above ground), the solidity for excavation (under ground) and the sensibility to vibration of rock masses using the acoustic impedance and the frequency of the divisional planes in the rock mass (Müller 2006).

1. INTRODUCTION

In doing blasting operations vibrations occur inevitably as unwanted side-effects in the surrounding area. These dynamic influences have been recorded for years by measuring the peak particle acceleration, the peak particle velocity or the peak particle propagation. As a result of the frequency range of 1-315 Hz, the peak particle velocity measurement with the referring frequency as a surrogate for vibrations has been implemented in blasting work. Therefore high grade measurement instruments and evaluation software were developed and applied worldwide. In the meantime dynamic strain sensors, basing on Fibre Brag gratings (FBG), which can be used for more precise recording of the direct stress-strain-state value during vibration influence, are offered. Although the measurement technique has developed significantly, the recording, evaluation, forecast and interpretation of the vibration measurements are handled quite differently, resulting in unsatisfied declarations. Worldwide, there is no standardised concept of reference values nor limits for the manifold buildings and structures as well as comparable standards. On the other hand different methods and models for the purpose of evaluation of charge weight-distance relations are presented in the technical literature, which should enhance the accuracy of vibration prediction and forecast. Some of these methods have been investigated with regard to measurement errors, modelling, statistical reliability of the relation and the declaration accuracy within a research project supported by the Deutsche Bundesstiftung Umwelt (German Federal Foundation of Environment). First results will be presented in this paper.

2. WAY OF MEASURING AND MEASURMENT ERRORS BY USING GEOPHONES

The systematically taken peak particle velocity measurements and the long-time experiences in soil dynamics resulted in the finding, that the

Table 1. Evaluation of the influence of the geophone stabilisation at the similar measuring point with the similar dynamic momentum.

direction of the momentum	geophone set up in the common way using 3 spikes(1)	geophone loaded with 5 kg (2)	error resp. decrease of the value (2) : (1) %
	(peak particle velocity mm/s)		
x	0,513	0,230	55,17
y	0,195	0,151	22,56
z	0,147	0,118	19,73

incoming resp. energised peak particle velocity and frequency at the immission point is influenced by the underground conditions at the location of the geophone. In this complex of problems tests were taken, initiating a similar dynamic momentum to the underground. The acoustic impedance of the related soil or rock as well as the peak particle velocity could be recorded at the measuring points. The data set showed a significant coherence of the peak particle velocity, which was enforced by the same momentum, to the acoustic impedance (Baumann & Müller 2000).With increasing acoustic impedance of the under ground, the peak particle velocity, induced by the same dynamic momentum, decreases. This means for practical circumstances of technical measurement, that if the under ground or building ground of a building shows a very low acoustic impedance and e.g. is formed of a cohesive and rigid soil, the peak particle velocity will be enhanced two or three times in comparison to a rocky under ground. Generally it can be concluded, that the sensibility to vibration of the underground depends on the acoustic impedance. For this reason the sensibility to vibration of the underground was implemented as an additional attribute of the resistance to blasting of rock masses.

Hence, the variance of peak particle velocity data can be influenced by the underground at the measuring points. Open air measurements are a popular method of vibration monitoring. The result of the evaluation of different data sets consistently shows the effect, that open air measurements always exhibit higher values in comparison to measurements, which were taken on a solid foundation or basement floor in a building in similar distance to the blast location.

In some countries it is prescribed to couple the geophones with the ground by using a mounting. In order to investigate the influences in this way of setting up different tests were taken. The recorded peak particle velocities, induced by constant dynamic momentum with or without stabilisation, are listed in Table 1.

The results show a distinct tendency. The geophones, which were stabilised by additional weight or rigid mounting at the measuring point, present more precise data. Small and light geophones with three or four spikes falsify the value by movement. It is recommended to mount the geophones firmly or to couple them with the vibrating underground according to these principles. For the evaluation it is often of interest, which vibration direction exhibits the highest amplitude and how is this related to the ignition sequence. Several tests were taken in the laboratory, which show unambiguously the analogy between the direction of highest incitation and the maximum value of the component (Figure 2). Graphs showing the similarity between the maximum value of the peak particle velocity and the highest dynamic influence are plotted in Figure 2. The setting up of the geophones has to be orientated to the main round direction of the blast momentum at the immission point or building accordingly for the purpose of a correct evaluation of the single y-y-z components.

The vibration propagation at the immission point does not occur constantly but inhomogeneous and anisotrophic corresponding to the rock mass conditions, the chosen ignition sequence and charge arrangement.

To recapitulate, it can be declared that the following influences lead to substantial variances of the expected peak particle velocities at one or several measuring points:

- the underground conditions
- the coupling of the geophones with the underground

– the exact orientation of the x-y-z components
– the anisotrophic and inhomogeneous propagation conditions at the blasting system and the corresponding ignition sequence

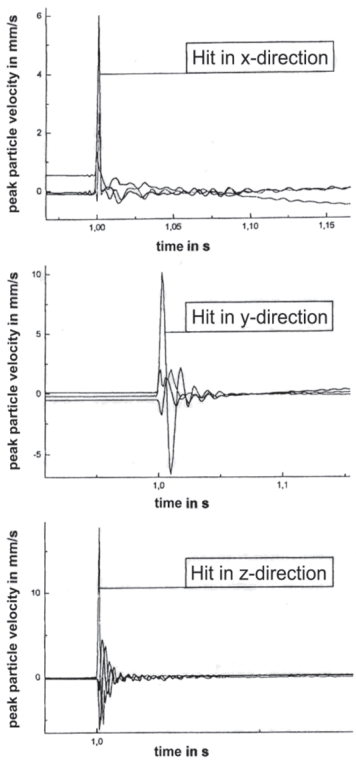

Figure 2. Single plots of the y-y-z directions of a 3 component geophone, induced by constant hits; the highest peaks accord to the relative hit direction.

3. MAIN VIBRATION INFLUENCING FACTORS AND MODELLING FOR EVALUATION

3.1 Available relations basing on significant vibration influencing parameters

The first accepted empiric relation for evaluation of blast vibrations was developed 1958 by Koch, basing on systematic peak particle velocity measurements in dependence of the location of the geophones to the blasting point.

$$v_i = k \cdot L_B^b \cdot r^{-m} \quad (1)$$

Symbol explication:

v_i = peak particle velocity (mm/s)

k, b, m = empirically determined dimensionless factors resp. exponents, which may change their values according to the geological-geomechanical conditions and the applied drilling, blasting and ignition techniques

L_B = charge weight per delay resp. charge weight per simultaneous ignited delay of a blasting system no matter of the position (for dimension correction it has to be divided over 1 kg)

r = distance between blasting point and measuring point (for dimension correction it has to be divided over 1 m)

Koch's essential finding of the considerable influence of the charge weight and the distance to vibrations are unquestioned to this day and obtains as a fundamental declaration.

Numerous authors used the influence factors:

– distance r (blasting point – measuring point) and
– charge weight per delay (L_B)

For further evaluation and development of the following relations (Blair 1999, Dowding 2000, Heinze 1993, Hustrulid 1999, Lüdeling & Hinzen 1986, Persson, Holmberg & Lee 1994, Siskind 2000):

for crystalline rocks

$$v_i = 206 \cdot L_B^{0,8} \cdot r^{-1,3} \quad (2)$$

for siliceous rocks, general

$$v_i = 897 \cdot L_B^{0,68} \cdot r^{-1,51} \quad (3)$$

for sedimentary rocks

$$v_i = 969 \cdot L_B^{0,6} \cdot r^{-1,5} \quad (4)$$

$$v_i = 16 \frac{\sqrt{0,01 \cdot L_B}}{(0,01 \cdot r)^{1,4786 + 0,1314 \cdot \lg(0,01 \cdot r)}} \quad (5)$$

$$v_i = 4273 \cdot L_B \cdot r^{-n} \quad (6)$$

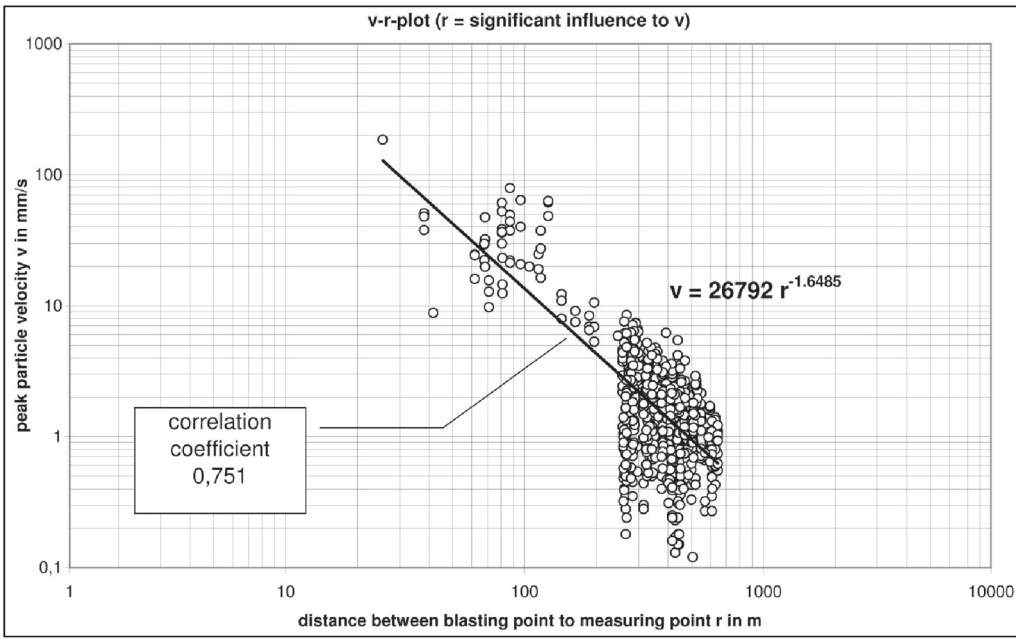

Figure 3. Influence of the distance between the blasting point to the measuring point with a vast variance in a quarry.

$$v_i = 900 \cdot \left(\frac{r}{\sqrt{L_B}} \right)^{-1,6} \quad (7)$$

$$v_i = \frac{r}{\sqrt{L_B}} \quad (8)$$

$$v_i = 182 \left(r / L_B^{0,5} \right)^{-1,82} \quad (9)$$

$$v_i = 187 \left(r / L_B^{0,5} \right)^{-1,18} \quad (10)$$

$$v_i = 4273\, Ds^{-2,08} \qquad Ds = \sqrt{\frac{ft}{lb}} \quad (11)$$

The selected charge weight-distance or scaled range relations, which are often proved by statistical investigations, base on Koch's unquestioned model representation, which is pointing out, that the peak particle velocity increases due to the increase of the charge weight per delay and the decrease of the distance. From the influence factors L_B and r the most obvious influence to the vibrations is showed by the distance of the blasting point to the measuring point (Figure 3).

The charge weight per delay L_B does not exhibit a statistically proven and verifiable coherence to the peak particle velocity (Figure 4). The ± consistent dependencies and the analogical calculation results of the peak particle velocity, using the forecast relations (2) till (11), can always be reduced to the predominant influence of the factor r.

The application of the different forecast formulas partially occurs quite uncriticised without comprehension of the applied drilling, blasting nor ignition techniques, the explosive characteristics, the geomechanical conditions of the under ground, the coupling of the geophones at the measuring point and/or other measurement categories.

3.2 Model description for vibration forecast

The previous forecast should be checked accurately according to the following reasons:

– The vibration forecasts are often developed by statistical investigations of recorded data. The comprehended factors resp. exponents obtain for specific measuring objects, undergrounds,

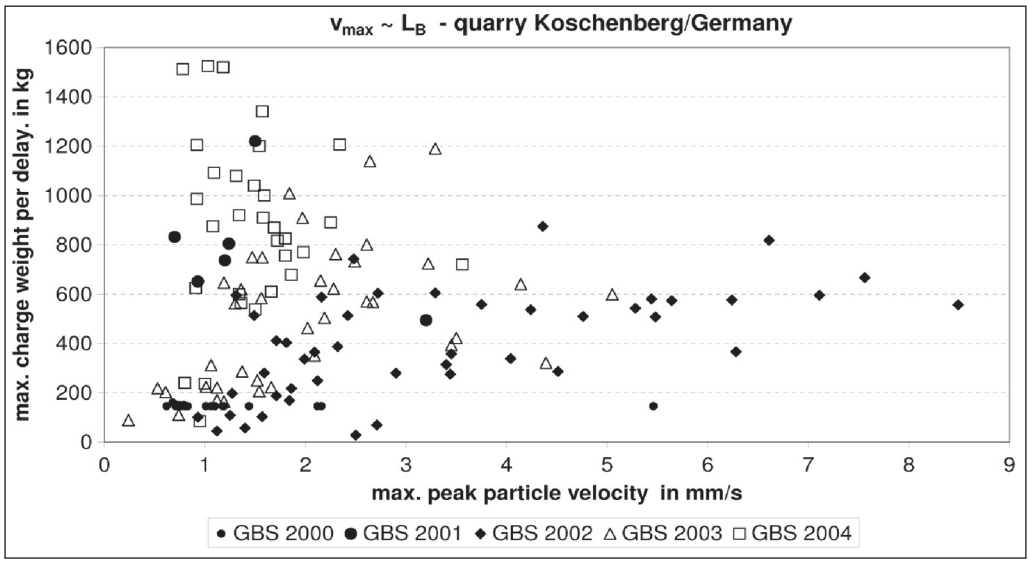

Figure 4. Relation between the charge weight per delay and the peak particle velocity in the meta-greywacke quarry Koschenberg/Germany (Fischer et al. 2005).

Table 2. Results of selected correlation coefficients from statistical calculations of different vibration influencing parameters.

statistical investigation v_{max} ⇒	correlation coefficient R^2	notes
r (Bild 3)	≥ 0,7	biggest influence to vibrations
$L_B \cdot r^{-n}$ (Bild 4)	≤ 0,55	the charge weight per delay downgrades the correlation
$W_B \cdot r^{-n}$(Bild 5)	≥ 0,7	the charge weight of a bore hole does not change the correlation
$W_B \cdot c_d \cdot r^{-n}$ (Bild 6)	> 0,8 ... 0,99	($W_B \cdot c_d$) = momentum, upgrades the correlation
$\frac{W_B \cdot c_d^2 \cdot r^{-n}}{2}$	> 0,84 ... 0,99	$\frac{W_B \cdot c_d^2}{2}$ = energetic factor of the explosives, upgrades the correlation

rock masses etc. without a general evaluation of the empiric character of the relations

– The transferability and generality of the relations (2) till (11) to others or comparable objects is dubious or unacceptable due to the manifold influence factors, if the drilling, blasting and ignition parameters apart from the distance change significantly
– The extraction in the quarries or the proceeding at rock construction sites is restricted by the declaration or specification of a charge weight per delay and can lead to the abandonment of a plant due to economic reasons or vibration problems
– On the one hand, the availability of very different, modern explosives for full borehole charging advanced in recent years. On the other hand, ignition techniques were developed to a high technical level, so that a vast variety of possibilities of delay techniques exist

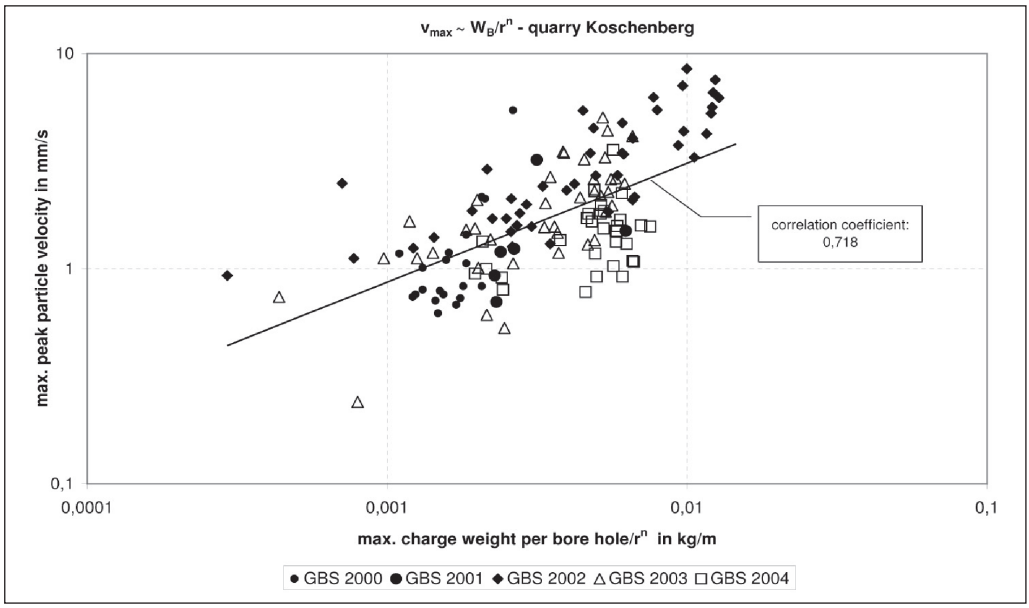

Figure 5. Relation of the peak particle velocity to the value of ratio maximum charge weight of a bore hole $W_B \cdot r^{-n}$; the blastings were ignited according to the principles of the momentum theory (Fischer et al. 2005).

$$v_{max} = R_M \cdot \left(W_B \cdot c_d \cdot r^{-n}\right)^m \quad \text{or} \quad \frac{W_B \cdot c_d^2}{2} \qquad (12)$$

- v_{max}: maximum peak particle velocity (mm/s)
- R_M: rock mass coefficient of statistical correlation, influenced by the specific blast effect; corrects the dimension
- W_B: = energetic effect of the charge weight to the rock mass (Joule)
- c_d: = momentum (kg/ms) = impuls energy to the rock mass
- r: distance between the blast system to the measuring point (m), applied as attenuation factor
- n, m: statistically evaluated exponents

On the base of these examinations and the application of the momentum theory as well as the consequent energetic interpretation to the detonative explosive reaction of a blasting system, the following parameters form momentum generation resp. energetic effect were statistically investigated concerning to their influences to vibration resp. the surrogate peak particle velocity (Fischer et al. 2005, Müller & Böhnke 2003, Müller 2005/2006) (Table 2).

Symbol explication for Table 2:

v_{max} = maximum peak particle velocity of one of the three oscillation planes x-y-z (mm/s)

r = distance between blasting point and measuring point (m)

L_B = charge weight per delay (kg)

W_B = charge weight of a borehole (kg) – the blasting system has to be ignited according to principles of the momentum theory (Fischer et al. 2005, Müller 2005/2006)

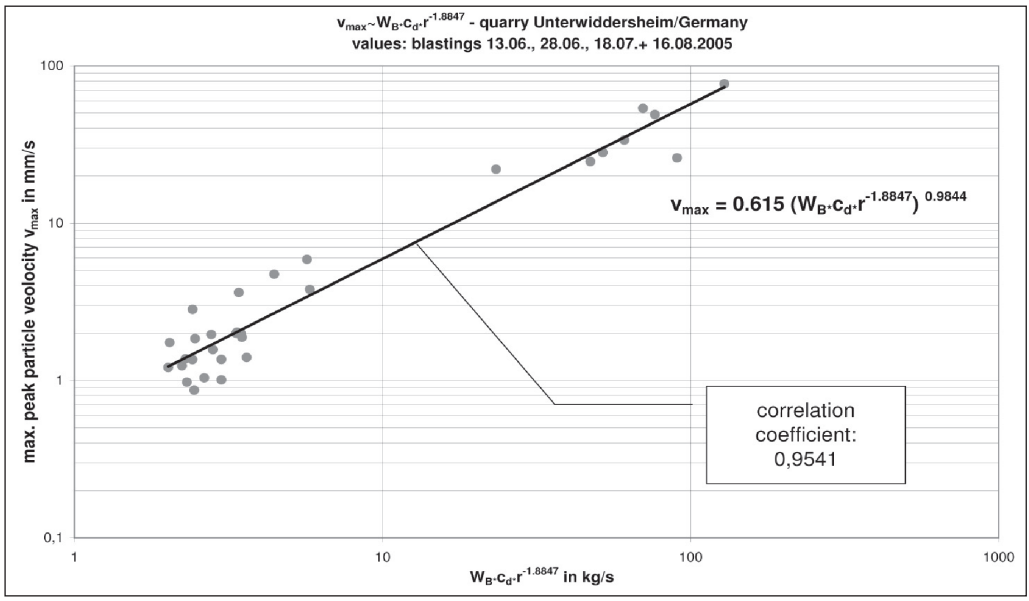

Figure 6. Basalt quarry Unterwiddersheim/Germany $W_B \cdot c_d \Rightarrow 0{,}9$.

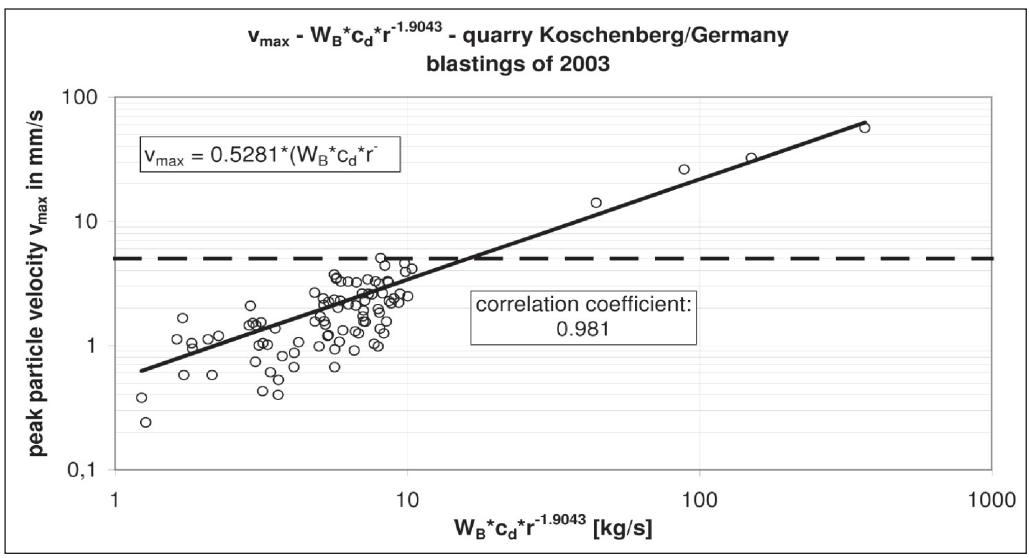

Figure 7. Momentum-distance relation of blastings in the year 2003; meta-greywacke quarry Koschen-berg/Germany.

c_d = detonation velocity of used explosives in situ (m/s)

For the statistical calculations consistent measurerand variables and input parameters were used, which had been recorded over a longer period by investigating blastings under normal production conditions in a basalt and a meta-greywacke quarry.

The results document the way how to generalise and modelize a relation for vibrations:

This model (12) has been applied successfully for vibration measurement evaluation and forecast

Table 3. Correlation coefficients and other statistical results for evaluation resp. comparison of vibration forecast relations.

relation	basalt quarry correlation coefficient (low number of data)	meta-greywacke quarry
correlation coefficient (high number of data)		
(2)	0,824	0,695
(3)	0,826	0,686
(5)	0,535	0,811
(7)	0,759	0,624
(8)	0,501	0,141
(9)	0,508	0,621
(10)	0,663	0,680
(12) momentum	0,846	0,985
(12) energy	0,851	0,987

(Fischer et al. 2005, Müller & Böhnke 2003, Müller 2005/2006) (Figures 6 and 7). With accordant proceeding the relation is:

- statistically proved
- right dimensioned
- applied within the variance of the input parameters W_B, c_d, r
- the peak particle velocity has to be recorded close to the blasting point and in far-off objects

Due to our results a generalised relation for vibration problems, like the exemplary listed formulas (2) till (11), cannot be formulated. The relation has to be developed and calculated according to the presented model (12) for each object or quarry, basing on a profound data set (Müller 2005/2006).

4. COMPARISON OF DIFFERENT PEAK PARTICLE VELOCITY-CHARGE WEIGHT-DISTANCE RELATIONS

The selected forecast relations for peak particle velocity evaluation and recording (2) till (12) were compared with each other by statistical investigation concerning to their proved declaration and forecast calculation accuracies. To guarantee an objective comparison, *consistent* parameters and measurement results of a basalt quarry with a hart blastable and a meta-greywacke quarry with a medium blastable rock mass were selected.

The used data sets, partly documented in the Figures 6 and 7, were recorded systematically in the close-up and far-off range, attending the details described in the chapters 2 and 3. The non linear relations require an aligned statistical regression and correlation analysis:

$$\hat{n}_i = v_i - \hat{v}_i \tag{13}$$

$$RSS = \sum_{i=1}^{n} \left(\hat{u}_1 - \ddot{\hat{u}}_i\right)^2 \tag{14}$$

$$TSS = \sum_{i=1}^{n} \left(v_1 - \ddot{v}_i\right)^2 \tag{15}$$

$$R^2 = 1 - \frac{RSS}{TSS} \tag{16}$$

The calculation results are listed in Table 3:

The results of the comparison exhibit a wide spreading correlation coefficient between 0,141 till 0,987. Of any calculations just the model (12) is acceptable also under conditions of a low number of data resp. blastings. The other functions exhibit large fluctuations, which merely can reach an acceptable value of the correlation coefficient >0,8, if they are based on a good data set. Hence, if the boundary conditions are attended, the vibration forecast model described in chapter 3.2 is recently the most applicable for the purpose of:

- statistically proved and
- physically substantiated

vibration evaluation. Above ground and underground blast vibrations can be influenced, decreased and forecasted with the help of the momentum- resp. energy-distance relation.

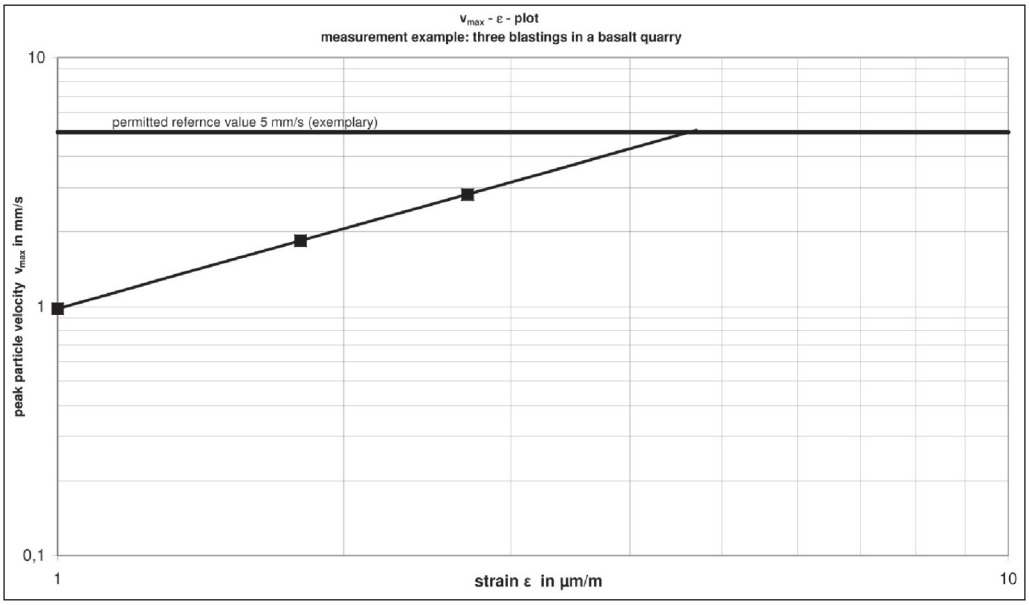

Figure 8. Coherence between strain and Peak particle velocity at a measuring point for blastings in a basalt quarry (5 to 7,2 t emulsion explosives per blasting).

Table 4. recommendation of a specification of building and structures in construction types due to the lowest crack tension ε_F.

Construction type	Used construction material	Minimum crack tension of the building material ε_F	Examples
1	mortar, glass, bricking	0,05 till 0,1 mm/m	residential houses, commercial property
2	concrete, wood, natural stone, steel	> 0,1 mm/m	pylons, embankment dams, reinforced concrete bridges, steel bridges

5. PEAK PARTICLE VELOCITY AND STRAIN

Our company has used several strain sensors for stress-strain-state measurements induced by explosive reactions for more then six years. The direct strain variable shows a linear coherence to the peak particle velocity at a measuring point (Figure 8). The amplitudes of the graphs of a stress-strain-measurement exhibit analogical development like the peak particle velocity (Baumann & Müller 2000, Müller & Böhnke 2003). However, the oscillation of the dynamic strain sensor are more stable and less sensitive to er-rors, so that more precise measurement data can be recorded.

The advantages of the FBG-sensors and their application are:

- accurate and direct coupling with the measuring point
- constant temperature recording
- simultaneous and high-resolution measurement of stress- and strain-states
- only one calibration needed
- long-term stability, no influence of high frequency effects or electromagnetic waves
- maintenance-free
- online data transfer
- more definite according to the real effects to a building

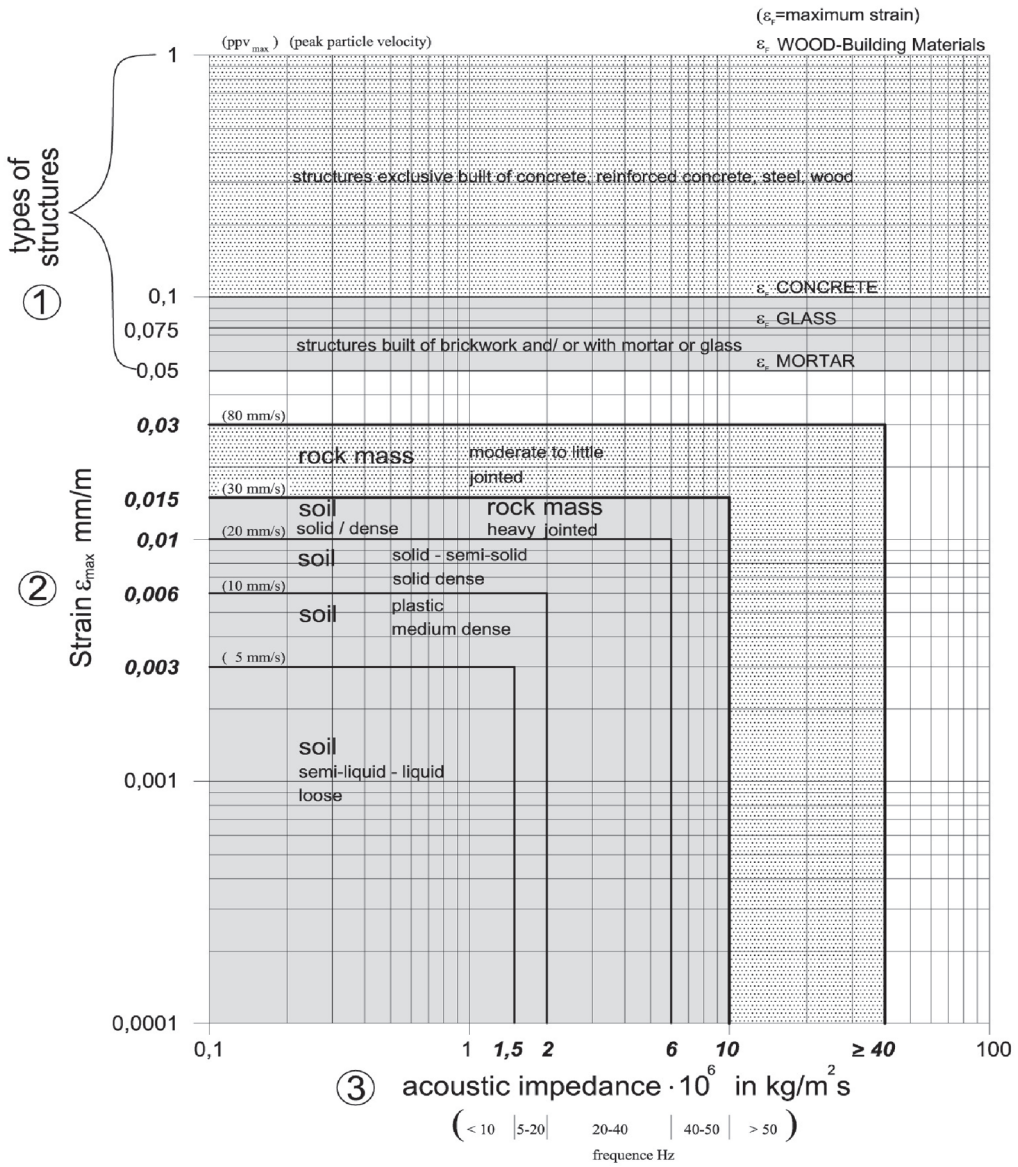

Figure 9. Recommendation of a specification of permitted reference values for short-term dynamic influences based on strain and peak particle velocity.

As a result of the numerous measurements of strains caused by dynamic effects since 2000 a recommendation of a specification of permitted reference values of strain states due to short-term vibrations of all types of constructions has been designed. During the construction of buildings and structures the strain formation is crucial.

The building materials crack or fissures occur, if the crack tension is reached resp. overstepped. Therefore it is recommended to classify civil structures with regard to the used construction materials (Table 4). Two construction types are assigned to the implemented building material due to the lowest crack tension ε_F.

According to the value of the lowest crack tension, the maximum permitted direct strain value has been defined for 30 % of the dynamic short-term effect of the crack tension (Figure 9).

For further assignment of the reference values a special influence was accorded for the under ground and building ground at the site of the building and structure, appropriate to the presented results in chapter 2 and 3.

The permitted strain reference values are compared with the surrogate peak particle velocity, which may be used simultaneously (Figure 9). The permitted reference values plotted in figure 9 are applied for:

- the foundation resp. foundation level of the construction
- short-term dynamic effects >1 Hz
- buildings and structures of all kind of use

and give consideration to the modern, objective requirements for vibration evaluation.

6. CONCLUSIONS

Occurring possible measurement errors by doing vibration measurements with geophones have been discussed and ways of advanced data recording by doing peak particle velocity measurements have been pointed out. For selected empiric charge weight-distance relations, which base on similar data sets of different blastings in two quarries, statistical investigation were taken. As a result it was shown, that the predominant majority of the offered relations does not accord the high requirements of forecast and prediction needed for reduction of vibration.

The model representation of the momentum- resp. energy-distance relation is recently the most applicable for proved declarations of vibration forecast and influence. Strain measurements can substitute peak particle velocity measurements due to their linear coherence with them. Within the actual investigations, which are supported by the Deutsche Stiftung Umwelt (German Foundation of Environment), the found findings will be enhanced and generalised requirements as well as measurement techniques will be tested in different rock masses.

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Comparison of vibration levels from drill & blast work and mechanical excavation of tunnels with roadheader

F. Egger, Ch. Heiss & P. Moser
Chair of Mining Engineering, Department Mineral Resources & Petroleum Engineering, University of Leoben, Austria

ABSTRACT: Excavation by drill & blast work as well as mechanical excavation with roadheaders causes an impact of energy at the tunnel face that generates ground vibrations. These vibrations are transmitted through the rock mass as seismic body and surface waves and may cause damage to the surrounding rock mass of an underground opening. To quantify the impact energy of seismic waves to the surrounding rock mass of an underground opening generated by drifting activities in general and especially to compare different excavation methods based on ground vibration measurements, the following factors were defined by investigations within the research-project TUNCONSTRUCT, which is co-financed by the European Commission under the 6th framework program:

- the energy consumption of the excavation method E_{spec} (energy consumption of the cutting head, energy of the explosives)
- the radiated seismic energy for a unit element at certain distances along the sidewalls ($E_{spec,rad}$)
- the seismic efficiency factor ($\eta_{SE}= E_{rad}/E_{spec}$)
- the impact quantification number (IQN)

1. INTRODUCTION

Within the research-project TUNCONSTRUCT the chair of Mining Engineering and Mineral Economics at the University of Leoben has been making investigations about the excavation damage zone (EDZ). Seismic measurements were carried out in several tunnel sites and underground mines to get information about the influence of ground vibrations to this zone.

The present investigation comprises more than 400 in situ measurements that were performed in three underground mines for drill and blast (D&B) development and in two underground mines, two tunnel sites and a test rig for roadheader development.

For D&B development ground vibrations measurements for full rounds as well as for single shots were carried out. All roadheader developments were done by the means of an ALPINE MINER ATM 105. The measurements were performed by the use of triaxial geophones, which were either fixed in boreholes or on the roadway surface. Following the measurements the dominating frequencies of the ground vibrations and a number of statistical parameters were determined from the observed data.

The main objective of these measurements and data analysis was the determination of input

Table 1. Main input parameters for modelling the energy impact of ground vibrations due to D&B work.

Parameter	Index	Unit	Location Mittersill	Breitenau	Erzberg
Depth of drift	OV	[m]	775	800	60
Depth of geophone	DG	[m]	2.6-3.0	1.9-2.1	0.35-0.55
Mean distance of the geophones from blast	D_{mean}	[m]	60.2	30.1	14.3
Drift cross section	CS	[m^2]	22.5	23.4	12
Rock density	ρ	[kg/m^3]	2958	2700	2703
Uniaxial compressive strength	UCS	[MPa]	91.2	136	35
Brazilian tensile strength	BTS	[MPa]	10.9	8.7	7.6
Young's Modulus	E	[MPa]	19361	102000	18136
Rock Mass Rating (Bieniawski)	RMR	[-]	55	41	55

parameters for different rock and impact conditions to define a general function for the propagation of ground vibrations. The investigation program can be structured into four main steps:

- data acquisition due to several field measurements
- data analysis
- development of a general function for the propagation of ground vibrations
- quantification of the impact of energy due to ground vibrations based on the factors mentioned above.

2. VIBRATION MEASUREMENTS

2.1 Vibration measurements in roadways developed by drill and blast work

Ground vibration measurements in drifts excavated by D&B work were carried out in two different underground mines for full rounds and within a blast research program in one mine for single shots. The main input parameters for modelling the energy impact of ground vibrations to the surrounding rock mass of the drifted roadway are summarised in Table 1.

2.1.1 Magnesite Mine Breitenau (Austria)

The cross section of the roadway in which the measurements were performed was 23.4m^2 and the overburden was of about 800m. The explosives used for the roadway excavation were Gelatin Donerit 1 and Emulgit LWC A1. A total of 54 boreholes were drilled for each round of shots. The drifting advance per round was 4.2m. The measurements were performed with geophones fixed in boreholes. The maximum of the recorded resulting ground vibrations was about 90 mm/s at a distance of 15.6m from the source.

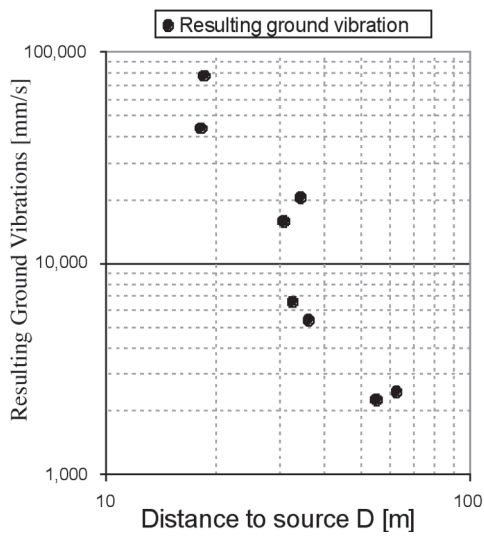

Figure 1. Resulting ground vibrations (Magnesite Mine Breitenau).

The measurements were taken one level above the drill and blast excavation. The arrangement of the geophones for the first full round is shown in Figure 2.

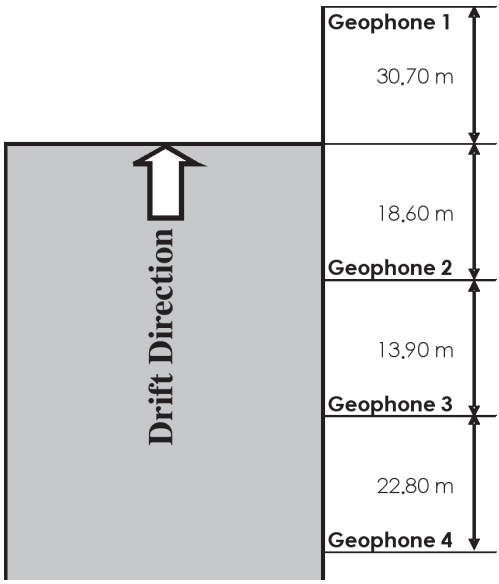

Figure 2. Arrangement of borehole geophones for full round #1 (Magnesite Mine Breitenau).

2.1.2 Tungsten Mine Mittersill (Austria)

Ground vibration measurements were performed during a drill and blast development at level TS775 at the Tungsten Mine Mittersill. The cross section of the roadway was 22.5m² and the overburden was about 775m. The explosives used for the roadway excavation were Austrogel G1 and Wandex. A total of 65 boreholes were drilled for each round of shots. The drifting advance per round was 3.8m. The measurements were taken with geophones fixed in boreholes and the VIBRAS 3004 measurement system. The maximum of the recorded resulting ground vibrations was 124mm/s at a distance of 14.5m from the source.

2.1.3 Iron Ore Mine Erzberg (Austria)

The ground vibration measurements at the iron ore mine Erzberg were performed within a blast research programme with RocKracker™. RocKracker is a new, propellant-based explosive for breaking oversize boulders without the safety concerns of noxious gases or explosives and detonators. RocKracker uses a composite propellant and electric, or non-electric shocktube thermal, igniter to generate gas pressure in the hole bottom and fracture the boulder (cp.: www.rocktek.com).

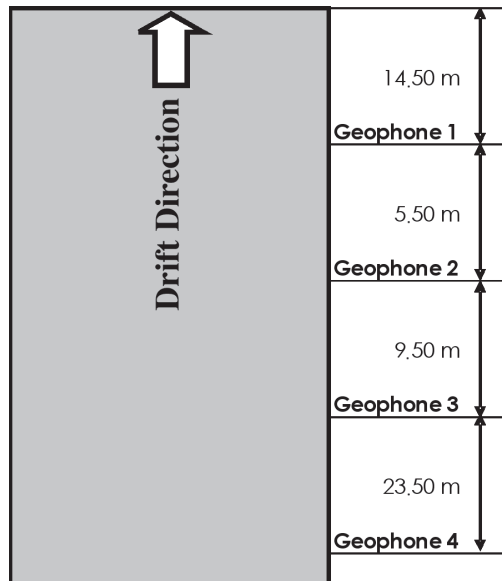

Figure 3. Arrangement of borehole geophones for full round #1 (Tungsten Mine Mittersill).

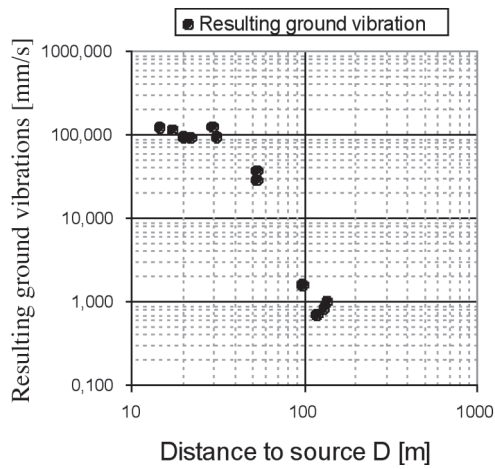

Figure 4. Resulting ground vibrations (Wolfram Mine Mittersill)

The diagram in Figure 6 shows the resulting ground vibrations for the different single shots related to

the distance of the source and in the diagram of Figure 7 the resulting ground vibrations related to the scaled distance of the source are shown. The scaling of the distance by dividing the distance with the square root of the mass of the explosives leads to a very good normalization of the results of the single shots with different charges.

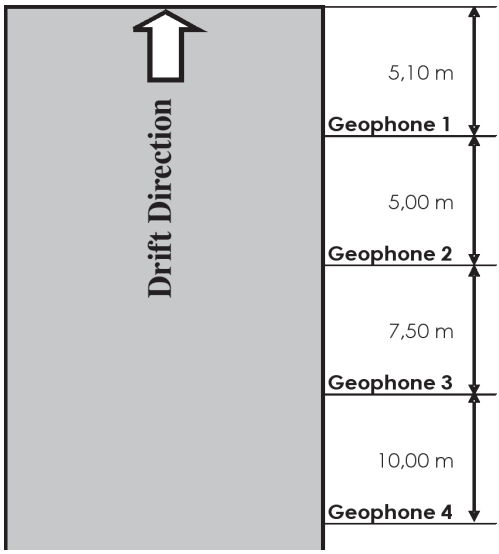

Figure 5. General arrangement of borehole geophones (Iron Ore Mine Erzberg).

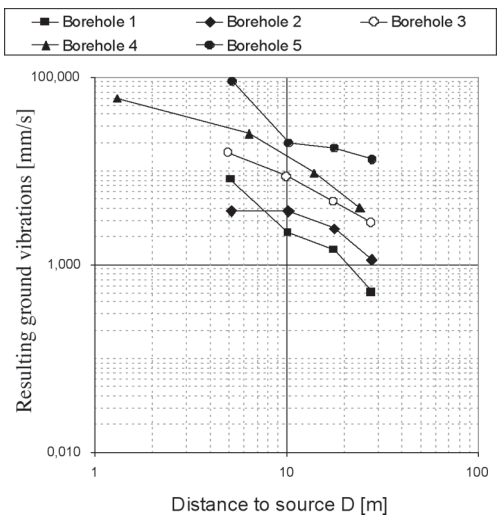

Figure 6. Resulting ground vibrations (Iron Ore Mine Erzberg).

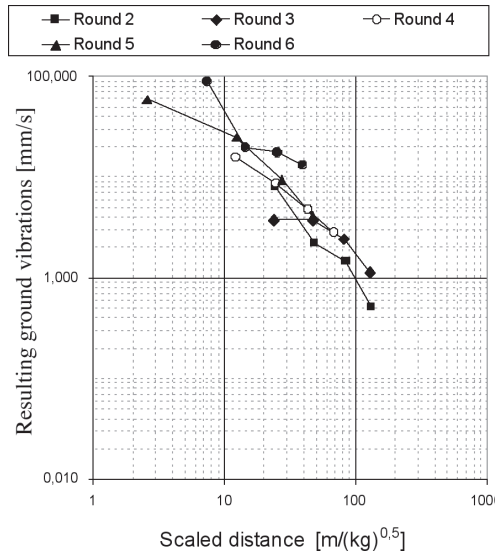

Figure 7. Resulting ground vibrations – scaled distance diagram (Iron Ore Mine Erzberg).

2.2 Vibration measurements in roadways developed with roadheader

To obtain data of drifts developed with roadheader, measurements were performed in four different underground mines and tunnel sites. All drifts were excavated by the means of an ALPINE MINER ATM 105 roadheader manufactured by Sandvik the former VOEST-APINE Bergtechnik Zeltweg. This roadheader is an extremely powerful boom-type roadheader of the 100 tons class and has proven its unique transverse cutting technology in hard rock applications worldwide.

The main input parameters for modelling the energy impact of ground vibrations to the surrounding rock mass of the drifted roadway are summarised in Table 2.

2.2.1 Pozzano Road Tunnel in Sorrent (Italy)

The tunnel is located in a sequence of thickly bedded dolomitic limestone. The average thickness of the layers amounts to about 0,5m. Due to its location in a tectonically active zone the rock was cracked with a varying intensity. The state of the tunnel face was highly variable, ranging from massive to destabilized within a short distance. A total of six measurements were carried out using the VIBRAS 3004 measuring system with four geophones.

Table 2. Main input parameters for modelling the energy impact of ground vibrations due to excavation by the means of roadheader.

Parameter	Index	Unit	Location					Boundaries	
			Sorrent	Erzberg	Montreal	Cullinan	VAB	min	max
Depth of drift	OV	[m]	250	50	20	717	2,5	2,5	717
Depth Geophone	DG	[m]	0,1	0,1	2	0,9-3,5	0,3	0,1	2
Mean distance of the geophones from blast	D_{mean}	[m]	14	17,3	15,6	11,8	7,8	7,8	17,3
Drift cross section	CS	[m^2]	42	28,8	46	25	33,1	25	46
Rock density	ρ	[kg/m^3]	2720	2630	2640	2698	2250	2250	2720
Uniaxial compressive strength	UCS	[MPa]	188	152	94,4	79,4	32,3	32,3	188
Brazilian tensile strength	BTS	[MPa]	7,7	11	6,4	6,6	3,1	3,1	11
Young's Modulus	E	[MPa]	47083	22550	10374	13704	10047	10047	47083
Mean frequency	f_{mean}	[1/s]	48	74	236	299	7	7	299
Rock Mass Rating (Bieniawski)	RMR	[-]	54	46	64	55	89	44	90
Mean power	P_{mean}	[kW]	215	183	153	150	88	88	225
Mean cut	Cut_{mean}	[mm]	134	152	138	142	143	134	152
Sump in depth	SD	[mm]	750	630	650	650	650	630	750
Mean slewing speed	v_s	[m/s]	0,2	0,2	0,2	0,2	0,2	0,2	0,2

The first geophone was placed close to the roadheader (1m), the remaining 3 geophones were placed 5m, 10m and 20m behind the roadheader along the tunnel axis. To get comparable results all geophones had the same orientation (x-direction outwards). The geophones were placed in a distance of 0.5m close to the sidewall. The horizontal positioning was made by means of a level on each geophone. Figure 9 shows the maximum resulting ground vibrations versus distance from source.

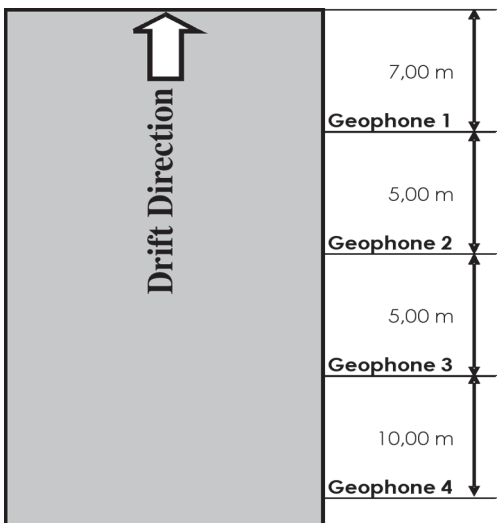

Figure 8. General arrangement of geophones (Pozzano Road Tunnel Sorrento/ Italy).

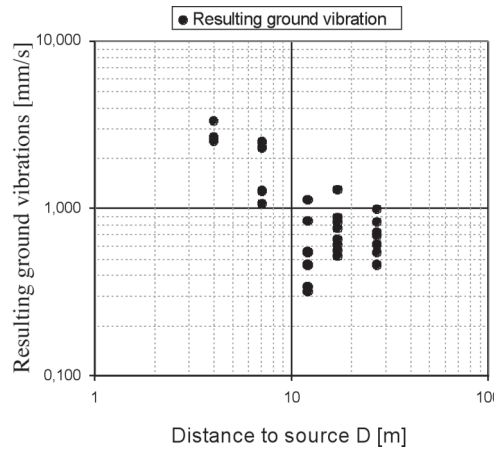

Figure 9. Resulting ground vibrations (Pozzano Road Tunnel Sorrento/ Italy).

The highest recorded resulting ground vibration of all test measurements close to the roadheader was 3.36 mm/s. 5, 10 and 20 meters behind the roadheader the peak particle velocity decreased abruptly below 1 mm/s to a minimum of 0.32 mm/s.

2.2.2 Iron Ore Mine Erzberg (Austria)

The test site was located in an old drift which was used to serve the Erzberg iron deposits and situated about 60m away from the open air in a breakaway of the main gallery. The roof support was ensured by bolting and jet grouting. Six individual vibration

measurements were carried out. The duration of the measurements was 1 to 1.6 seconds per measurement. The first geophone was placed close to the roadheader (1m), the other 3 geophones were placed 5m, 10m and 20m behind the roadheader. The values of ground vibrations during cutting process were very low. The maximum resulting ground vibration of all test measurements close to the roadheader was 0.93 mm/s. 5m, 10m and 20m behind the roadheader there was an abrupt decrease of the resulting ground vibrations to less than 0.1 mm/s, as can be seen in Figure 11.

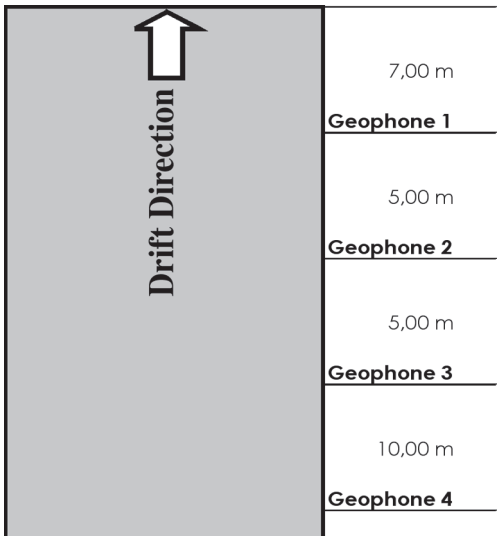

Figure 10. General arrangement of geophones (Iron Ore Mine Erzberg / Austria).

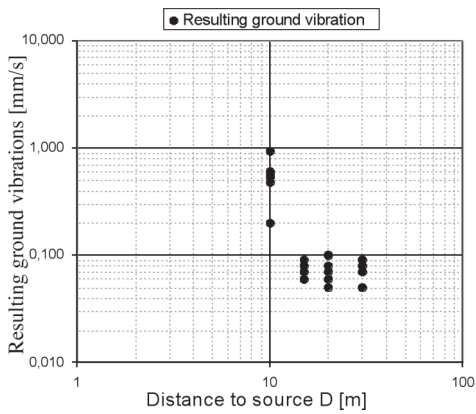

Figure 11. Resulting ground vibrations (Iron Ore Mine Erzberg / Austria).

2.2.3 Montreal Metro Tunnel (Canada)

The Montreal Metro Tunnel was excavated over a length of 1200m with a cross section of 46m² by means of a roadheader. The main rock type in the study area was a shaly fossiliferous limestone with numerous calcite veins in it. The persistence of these veins was in the range between a few millimetres and a few meters. Layers of shale are deposited with a thickness between 30 and 155 centimetres. A total of 87 measurements using triaxial-geophones fixed in boreholes and 6 measurements using the VIBRAS geophones were taken. The resulting ground vibrations showed maximum values of 1.6mm/s at a distance of 2.7m. The measuring range was from 2.7m to 50m from the source of impact. Figure 13 shows the maximum resulting ground vibrations versus distance from source.

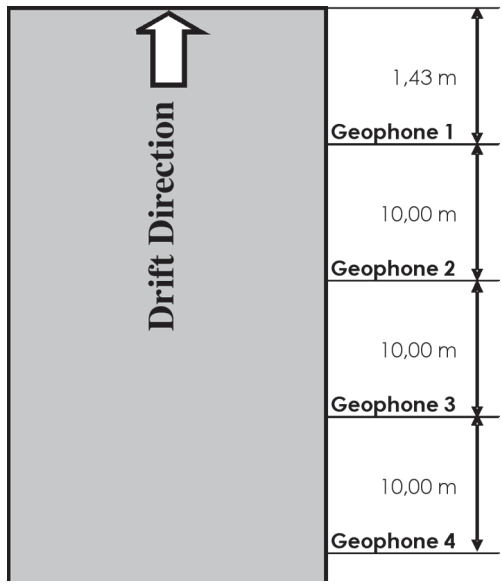

Figure 12. General arrangement of geophones (Montreal Metro Tunnel / Canada).

2.2.4 Cullinan Diamond Mine (South Africa)

Ground vibration measurements were carried out while the roadheader was cutting in T64 south from South towards North. A total of 24 measurements using triaxial-geophones were taken in Tunnel T66 South to determine ground vibrations caused by roadheader action. In total 118 resultant ground vibrations for the maximum peak value were

calculated. All measurements were taken between 5,0m and 25,3m distance to the roadheader boom. The exact distance between the geophone and the boom is not known as the cutting head position is permanently changing during the cut. In general, the uncertainty decreases with the distance of geophone from the tunnel face. Figure 8 shows the maximum resulting ground vibrations versus distance from source.

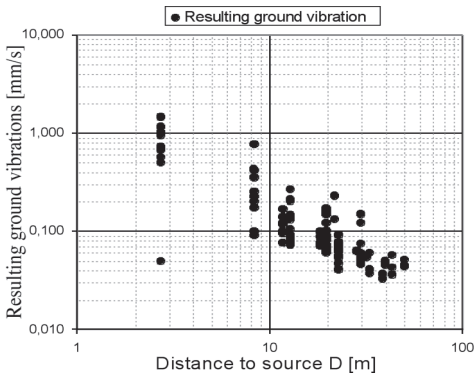

Figure 13. Resulting ground vibrations (Montreal Metro Tunnel / Canada).

3. ANALYSIS AND MODELLING

The large number of field tests provided a large range of data which offered the opportunity to carry out deep investigations and to furnish worthy information about the impact of energy. The results of these analysis showed that the impact of energy depends on a number of parameters, mainly on the energy input of the excavation method, its seismic efficiency and the absorption coefficient. However, it remains crucial to keep in mind that in spite of an extensive scale of data, reality can never be strictly described. Hence approximated data must be considered with caution.

Based on the measuring results the determination of a general function for the propagation of ground vibrations caused by different excavation methods along the sidewall of a roadway considering different geometrical, rock and rock mass conditions was a main objective of the investigation program. On the basis of this function a model which describes the impact of energy was set up to quantify the cumulative impact during 1m of axial roadway development or during the excavation of 1m^3 of rock.

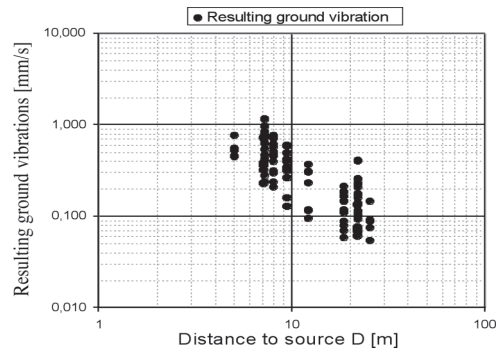

Figure 14. Resulting ground vibrations (Cullinan Diamond Mine / South Africa).

3.1 Propagation function of ground vibrations

To determine a general function for the propagation of ground vibrations a number of basic equations were defined which approximate the resulting ground vibrations at the source of impact and at any

Table 3. General arrangement of geophones (Cullinan Diamond Mine / South Africa).

Geophone No.	Borehole	Installation depths	Height above level	Distance to the cutting head on 2004/12/01	Distance to the cutting head on 2004/12/02
		[m]	[m]	[m]	[m]
1	T 66 S 03 A W	0.9	2.5	21.9	25.3
2	T 66 South 03 W	1.2	2.2	18.6	21.8
3	T 66 South 05 W	1.3	1.4	9.4	12.1
4	T 66 South 06 W	1.2	1.7	8	7.1
5	T 66 South 06 W	3.1	1.7	7.2	5

distance to the source of impact. Therefore several assumptions had to be made to simulate the complex process of drifting and the complex function of the propagation of seismic waves in the rock mass.

The developed function of seismic wave propagation in hard rock is based on the general equation for propagation of ground motions which is used in geophysics and composed of a power function - to consider the wave type (depending on geometrical parameters) - and an exponential function - to consider absorption (mainly depending on internal friction, when a loss of energy occurs and vibration energy is partly transformed into heat and fracture energy):

$$v_D = v_0 \left(\frac{D_0}{D} \right)^n \cdot e^{\alpha \cdot (D_0 - D)} \qquad (1)$$

with

v_D vibration in the distance D [mm/s]
v_0 vibration in the distance D_0 [mm/s]
D_0 distance where v_0 is known [m]
D distance from the source [m]
n wave form factor [1]
α absorption coefficient [1/mm]

3.1.1 The absorption coefficient

The absorption coefficient was determined by performing a dimensional analysis and a regression analysis using the measurement results of four drifts developed by means of roadheader. There the correlation coefficient which was reached in the least squares analysis was 0.69.

The final form of the approximation equation for the absorption coefficient is

$$\alpha = 8{,}15 \cdot 10^{16} \cdot \left(\frac{RMR}{100} \right)^{-27{,}41} \cdot UCS^{-0{,}011} \cdot E^{-3{,}86} \cdot OV^{-2{,}12} \cdot \rho^{31{,}281} \cdot \varphi^{1{,}64} \qquad (2)$$

with

α absorption coefficient [1/mm]
RMR rock mass rating [1]
UCS uniaxial compressive strength [MPa]
E Young´s modulus [MPa]
OV overburden [m]
ρ density [kg/m³]
ϕ frequency [1/s]

The absorption coefficient varies for different rock types from 10-7 m-1 to 10-1 m-1, as can be seen in Figure 15.

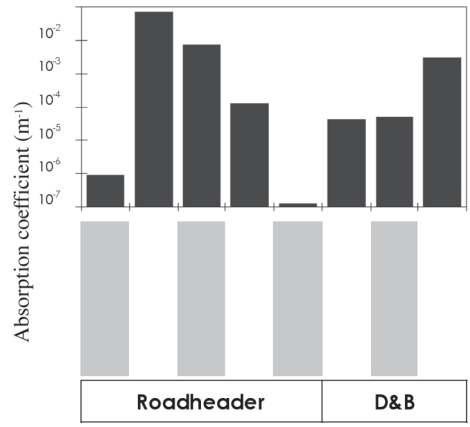

Figure 15. Determined absorption coefficients for investigated sites.

3.1.2 The wave form factor

The wave form factor for the investigated sites varied from 0.6 to 1.0. The parameters which determined the wave form factor were the:

- depth of geophone in the sidewall (DG)
- wave form factor coefficient (h)

The final equation for the wave form factor was:

$$n_D = \frac{2 \cdot \arctan\left(h \cdot \sqrt{DG}\right)}{\pi} \qquad (3)$$

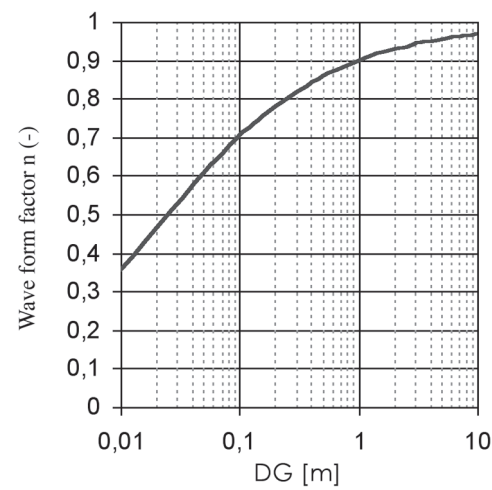

Figure 16. Variation of the wave form factor.

3.2 *Quantification of the impact of energy due to ground vibrations*

The last step of the investigation program was the quantification of the impact of energy. As a consequence of the complex form of the propagation function an approximation calculation was performed to quantify the impact of energy in five characteristics related either to 1m roadway development or to 1m³ of excavated rock:

- the energy consumption of the excavation method E_{spec} (energy consumption of the cutting head, energy of the explosives)
- the radiated seismic energy ($E_{spec,rad}$) for one unit element at certain distances in the sidewall
- the seismic efficiency factor ηSE (quotient of the cumulated radiated seismic energy and the energy consumption of the excavation method E_{rad} / E_{spec})
- the impact quantification number IQN, which is related to the cumulated movement of the unit element

3.2.1 *The energy consumption (E_{spec})*

The highest energy consumption (energy consumption of the cutting head, energy of the explosives) was determined at the Pozzano Road Tunnel site as can be seen in Figure 17. All determined specific energies were higher for roadheader development than for drill and blast development.

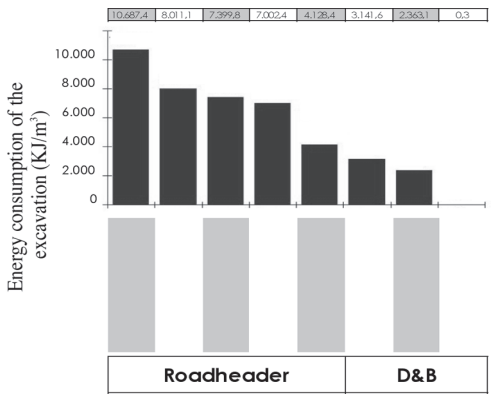

Figure 17. Specific energy consumptions of the investigated roadway drifts.

$$E_{spec} = \frac{E_{EM}}{VO} \quad (4)$$

with

E_{spec} specific energy consumption of the excavation [kJ/m³]

E_{EM} energy consumption of the excavation step [kJ]

VO average excavated volume per excavation step [m³]

3.2.2 *The radiated seismic energy for a unit element (Espec,rad)*

The simulation of the roadheader drifting sites showed that the main part of the seismic energy of a unit element close to the sidewall of the roadway (< 2m) was radiated within the first meters (2m - 10m). For drill and blast development these distances were generally higher, but mainly depending on the absorption coefficient and the wave form factor. The radiated seismic energy ($E_{spec,rad}$) caused per meter roadway development for a unit element of the density ρ and a volume of 1m³ is calculated by:

$$E_{spec,rad} = \sum_{k=1}^{K} E_{sec,rad,af,k} + \sum_{k=1}^{K} E_{spec,rad,be,k} \quad (5)$$

with

$E_{spec,rad}$ radiated seismic energy for a unit element [kJ]

k segment index used in the approximation calculation [1]

K number of segments [1]

$E_{spec,rad,af,k}$ radiated seismic energy for a unit element after the excavation has passed [kJ]

$E_{spec,rad,be,k}$ radiated seismic energy for a unit element before the excavation has passed [kJ]

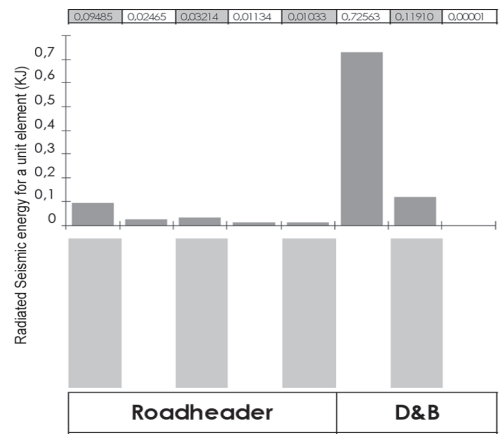

Figure 18. Radiated seismic energy for a unit element.

The radiated seismic energy for a unit element in a depth of 0.3m in the sidewall was lower for roadheader drifting, whereas for Pozzano and Breitenau, the energy differed less than 20%. The lowest radiated seismic energy was determined for the single shots using RocKracker, as it is a very soft explosive.

3.2.3 The seismic efficiency factor (ηSE)

The seismic efficiency factor shows the portion of the input energy which is transformed into seismic energy. The seismic efficiencies for full round of shots were much higher than for roadheader development, which can be seen in Figure 19. The seismic efficiency factor is calculated by:

$$\eta_{SE} = \frac{E_{rad}}{E_{spec,EM}} \quad (6)$$

with

ηSE seismic energy factor [1]

E_{rad} cumulated radiated seismic energy [kJ]

$E_{spec,EM}$ specific seismic energy of the excavation method [kJ]

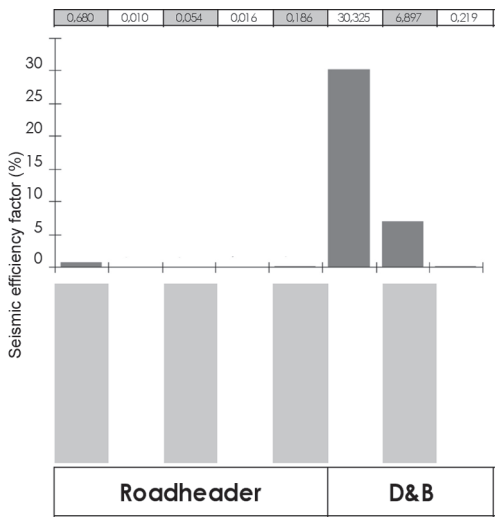

Figure 19. Seismic efficiency factor of the investigated sites.

3.2.4 The impact quantification number (IQN)

The impact quantification number gives an indication of the cumulative oscillation movement of a unit element of rock for the whole excavation process, e.g. initially the element is ahead of the excavation, then the element is in the vicinity of the excavation and ultimately it is in distance from the excavation face. It is calculated by

$$IQN = \left(\sum_{k=1}^{K} T_{k,af} \cdot RGV_{k,af} \cdot \frac{1}{2 \cdot \pi}\right) \cdot \left(\sum_{k=1}^{K} T_{k,be} \cdot RGV_{k,be} \cdot \frac{1}{2 \cdot \pi}\right) \quad (7)$$

With

IQN impact quantification number [m]

T_k exposed time of the segment k during the excavation process [s]

RGV_k resulting ground vibration at the distance k [m/s]

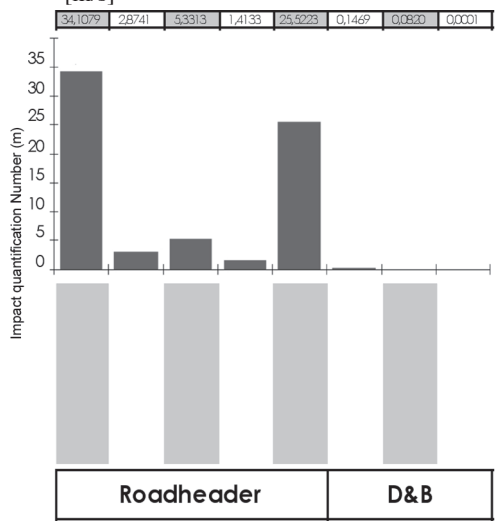

Figure 20. Impact quantification number for the test sites per m^3 excavated rock.

4. CONCLUSIONS

The comparison of ground vibration measurements of different excavation methods within the EU-Project TUNCONSTRUCT has shown three significant differences between excavation by the means of roadheader and excavation by D&B:

1) The specific energy consumption of mechanical excavation by the means of roadheader was in general higher than excavation by D&B. The energy consumption values of the excavation varies from 10.687 kJ/m^3 (Sorrento) to 7.002 kJ/m^3 (Cullinan) for roadheader excavation and from 3.141 kJ/m^3 (Mittersill) to 2.363 kJ/m^3 (Breitenau)

for D&B full rounds.

2) The impact quantification number was 10 to 200 times higher (but at low amplitudes) for roadheader development than for D&B development, because in the case of roadheader excavation a unit element is continuously subjected to oscillations as a result of the cutting drum, whereas in D&B the unit element is only subjected to ground vibrations during blasting time. This means that in the case of a roadheader development the duration of excitation is about 10^5 times higher than that of D&B development.

3) The radiated seismic energy and as a consequence the seismic efficiency factor was higher for D&B development than for roadheader development.

The high specific energy consumption and the low radiated seismic energy of roadheader development compared to D&B development can be explained by a smaller block size of the material excavated by a roadheader.

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Critical investigation into the drilling and blasting work done at the Mormont quarry in order to reduce blast vibrations

G. Reichholf
Coordinator raw material management, manufacturing services central Europe, Holcim (Schweiz) AG / Switzerland

C. Brocard & M. Honorat
Head of raw material extraction / quarry manager - both Holcim Eclépens, Holcim (Schweiz) AG / Switzerland

ABSTRACT: Holcim Eclépens produces about 650,000 tons of cement per year. To supply the plant with sufficient raw material in 2005 about 670,000 tons of limestone were extracted at the Mormont quarry by applying drilling and blasting. About 200,000 tons were mined at the Marniére marl quarry by using an excavator.

For many years residents of Eclépens, a village near the Mormont quarry, have been complaining about ongoing blast vibrations. Holcim responded with a guarantee to keep future blast vibrations below 3mm/s, which is distinctly below the legal limit. In 2005 Holcim decided to increase the raw material extraction by 30%. As the people of Eclépens feared an increase of blast vibrations, Holcim set up the working pro-gram 'concept carrière 2005' to critically analyse the future impact of that extraction increase.

Today, after more than 160 production blasts, only three single blasts exceeded the 3 mm/s limit. The majority of the blast vibrations are distinctly below 2 mm/s.

1. INTRODUCTION

Holcim is one of the world's leading suppliers of cement and aggregates (crushed stone, sand and gravel), it also produces ready-mix concrete and asphalt. Holcim (Schweiz) AG - cement production division - owns a cement plant in the western part of Switzerland. Today, the cement plant at Eclépens produces about 670,000 tons of cement.

To supply the plant with sufficient raw material year per year about 650,000 tons of lime-stone and about 200,000 tons of marl have to be extracted in the quarries around Eclépens. The marl quarry, Marniere, is around 2 km away from the next village and is mined by using an excavator. Holcim Eclépens does not have any problems with its neighbors. Unfortunately, this is not the case for the limestone quarry, Mor-mont (Figure 1). This quarry is also situated nearby the village of Eclépens but the material is extracted by applying drilling and blasting.

People from Eclépens have been complaining about ongoing blast vibrations for many years. To show the village understanding for their complaints Holcim Eclépens guaranteed that the future blast vibrations would be below 3mm/s. Although this value was distinctly lower than the legal limit, the residents of Eclépens reacted strongly when the limit was reached once. The result of that self restriction was that in some areas of the Mormont quarry the mined bench height had to be reduced down to 4 m only to reach the guaranteed blast vibrations level.

In 2005 Holcim Eclépens decided to increase the raw material extraction by 30%. When this strategy was communicated the people of Eclépens

feared that the impact on the village due to blast vibrations would increase, too. In spring 2005 Holcim Eclépens set up the working program ‘concept carrière 2005’ to ana-lyze critically the impact of the extraction increase on the performance of the Mormont quarry. Therefore, Holcim Eclépens and the Department of Mining Engineering of the University of Leoben/ Austria started a project to prepare a new mining strategy.

Figure 1. Mormont quarry of Holcim Eclépens (left: village of Eclépens).

2. GENERAL DESCRIPTION OF THE PROJECT WORK

The project work was split up into three parts:

- For a period of 6 years (2000 to 2005) an analysis of the historical blast data entered into a home-made database was carried out. This database comprised blast data such as the geometry of a blast, type and amount of explosives used, recorded blast vibrations, etc
- In a second step, a detailed analysis (geometry, drilling & blasting, vibrations) of the actual drill & blasting work done in the Mormont quarry was done. This analysis covered nine typical production blasts and defined the status quo of the drilling and blasting work of Eclépens
- In a third step, the test results of the nine blast-rounds carried out were critically compared with the data derived from the home-made database. On the basis of that comparison the future testing pro-gram was defined. In order to guarantee a high benefit for all participants, the attention was directed to a highly flexible testing program (e.g. learning from mistakes, etc.)

3. DATA MONITORING DURING THE TEST BLASTS

3.1 In general

During the test blasts a detailed monitoring program was in operation. This comprised the analysis of the blast-round geometry, an assessment of the quality of the drilling as well as of the charging procedure.

3.2 Monitoring before blasting

The blast geometry (burden, spacing, bench height) was measured by using a theodolite with reflector-less distance measurements. The quality of the drilling was assessed by using the data acquisition system ‘Boretrak’ which enabled the visualization of the direction and the inclination of the drilled boreholes along the total borehole length.

Both, the surveying data and the Boretrak data were finally transferred into the mining software Surpac. Within Surpac a modeling of both the face geometry and the borehole planes was done. As a result, pro-files across the bench face at every borehole position were determined. From these the burden variation along the drill holes could be derived. By comparing the face geometry of two subsequent blasts, an esti-mation of the back-break of every round could be determined, too.

Finally, from each blast several pictures were taken to describe the geology and other characteristics of the bench face.

3.3 Monitoring after blasting

There are several ways to assess the quality of a blast result. Bearing the complaints of the villagers of Eclépens in mind Holcim concentrated only on measuring the blast vibrations of each blast at various spots in and around the quarry. For that purpose between six and nine, three axial geophones of the Bartec ME 2002 type were used.

Each blast was filmed to analyze the movement of the bench face and to describe the characteristic posi-tion of the muck pile and the back break.

As Holcim Eclépens does not own a tool to measure the blast fragmentation on the spot the parameter of blast fragmentation could not be assessed. This is also valid for the velocity of detonation (VOD), which enables the control of the quality of the explosive detonation. For safety reasons Holcim Eclépens uses a detonating cord

in all boreholes. This guarantees a full detonation of the explosives along the boreholes, but disables the data - trap system to measure the VOD of the explosive in the borehole.

4 ANALYSIS OF THE HISTORICAL DATA

4.1 Recorded data and data calculated additionally

The blasting data of the years 2000 to summer 2005 was stored in a home-made database and covered around 1,000 blasts for that period. The following parameters were taken up:

- date, time and place (bench, section) of blast
- geometry of the blast (spacing, burden, bench height) and size
- number of boreholes drilled, borehole inclination and length
- type and amount of explosives used
- position of the geophones G1 to G3

An attempt was also made to gather additional data which may help with the ongoing analysis of the his-torical data. This data includes:

- the distance between the former blast and the position of the geophones (G1 - G3) in Eclépens
- specific explosive consumption per blast and explosives per delay
- season of the year
- orientation of the bench face towards Eclépens, etc

All the available data was put into a statistic program and several correlation analyses were carried out. Starting with a huge number of data it turned out that the quality of the data was obviously not as good as expected. For example, the names of the benches and the place of the blast were renamed, etc. Due to that it was necessary to reduce the number of blasts to 870 or 86%.

4.2 Conclusion of historical data analysis

Although Eclépens had stored a lot of blast data of previous years, the detailed analyses showed that the quality of the data is often not meaningful. This is due to a number of reasons:

- people entered wrong or incomplete data into the database
- names of the geophones and their positions were entered by different people in the database and with that the names were confused, or
- the geometry of hundreds of blasts was always identical and with that estimated

The analysis clearly showed that the recorded blast vibrations cannot be related to the type and amount of explosives, geometry of the blast, explosives per delay, etc. Clear correlations could only be found be-tween data that logically depends on each other (e.g. length of borhole vs. length of used detonating cord, etc.). Furthermore, on the bases of the analysis it can be stated that:

- the low quality of the data allows no serious and valid statement about the former drilling and blasting work done at the Mormont quarry. This is strongly valid for the data recorded before summer 2004. In summer 2004 the former quarry manager retired and a new manager was put in charge. From that time on the quality of the data tended to increase
- blast vibrations seem to be independent from the amount of explosives per detonator. For example bench 1 and bench 3 are both 25 m high, but the median amount of explosives per detonator of bench 1 is with around 50 kg around 30% lower than that of bench 3 (70 kg). Interestingly, the median blast vibrations are with 1.60 - 1.65 mm/s nearly identical
- blasts of the upper benches (bench 5 and bench 5+) produce with a median explosive amount of 25 kg per detonator vibrations up to 1.90 m/s (median values). In contrast, blasts blasted at bench 2 (25 m) produce with a median explosives amount of 40 kg the lowest vibrations (1.29 mm/s)
- problems with blast vibrations started in 2002; at that time Eclépens shifted its mining activities to-wards the upper levels of the quarry

5. CONTROL OF THE ACTUAL DRILL & BLAST WORK

5.1 Geometry and drill work

Nine blasts were recorded in detail to describe the actual drilling and blasting work done at Eclépens. It was found, that the actual blast geometry slightly differed from the planned values. This is shown in Table 1.

In general, the comparison between the planned and the actual blasting geometry showed a rather

Table 1. Comparison of planned and actual blast parameters.

Comparison of planned / actual blast geometry		
	planned	actual
Burden [m]	3	2.8 - 3
Side spacing top [m]	3/3.5	3/3.5
Side spacing bottom [m]	3/3.5	2 - 4
Hole diameter [mm]	89	89
Bench height [m]	13 / 25	11-12.5 / 22-24
Borehole inclination [°]	80	80
Sub drilling [m]	1	1-2.5

good result. Slight differences were found in the burden of the blast-round: the burden was a little overesti-mated by the quarry manager. The reason for that small difference is most due to the low back break. As the remaining parts of the boreholes were most times visible the quarry manager could determine the bur-den of the blast easily.

The bench height was overestimated with a median variation of + 1 m. The reason here is that Eclépens does not use an electronic measuring device to control the bench height; they only use a measuring tape to determine the bench height.

A big difference was found for the planned sub drilling. Although sub drilling was planned to be 1 m, the measured additional sub drilling was around 1 to 1.5 m. One reason was the high production rate at that time. Eclépens had to survey, drill and fill the boreholes with explosives before the muck pile of the for-mer blast was entirely removed in front of the bench face. To be on the safe side Eclépens tends to drill a little bit deeper than planned. Another reason was that Eclépens in general overestimated the bench height.

The side spacing on the top of the borehole meets the planned size and the side spacing in the bottom of the borehole deviate from the planned size. The reason for that deviation is based on the way in which Eclépens orientates the drilling machine with respect to the blast direction. To orientate the drilling equipment Eclépens adjusts it to a specific point on the other side of the valley. As it is often foggy at Eclépens the strategy is not really reliable.

Finally, it can be stated that the planed borehole inclination fits with 80° quite well (median variation: +/- 0.5°).

5.2 Blast work

The typical explosive column for a production blast with a bench height of around 12 m at “Mormont” consists of two types of explosives (data from manufacturer).

- Tovex S (bottom charge):
  density: 1.3 g/cm^3
  VOD about 4’000 m/s
  weight: 3.1 kg / cartr. (65 x 800 mm)
- Tovex T (column charge):
  density: 1.1 g/cm^3
  VOD about 3’700 m/s
  weight: 2.8 kg / cartr. (65 x 800 mm)

The explosive column of the standard blasts is designed as follows:

- two cartridges of Tovex S and two cartridges of Tovex T form the bottom charge
- then 1.5 m of intermediate stemming alternate several times with 3 cartridges of Tovex T until the planned stemming length of 2 to 3 m is reached
- drill dust is used as stemming material

Finally, the detonation is started with an electrical detonator on the bottom of the borehole. The detonator is connected to a detonating cord with 12 g/m or more.

5.3 *Conclusion*

The analysis shows that Eclépens has burden and spacing of a blast under control. Only the determination of the bench height causes problems and with that leads to an inappropriate sub drilling. To solve that problem a pocket laser system was bought which allows Eclépens to measure the bench height pragmati-cally. Finally, the orientation of the drilling equipment with respect to the blast direction has to be re- thought.

Regarding the explosive column design it can be stated that the borehole filling degree of a standard blast is with around 30 - 40 % extremely low. Eclépens has to increase the drilling pattern to increase the bore-hole filling degree as well; otherwise the question arises why Eclépens drills a borehole to fill 60% of it up with drill dust.

6. EVALUATION OF BLAST VIBRATIONS

6.1 *In general*

Vibrations generated by blasting transmit a seismic wave through the rock mass. The energy of the propagating wave generated by an explosive charge diminishes while spreading out from the borehole. In nature, the rock mass consists of different types of rocks and contains joints, cavities, etc. Due to that the propagation of a wave is of high complexity. To describe the systematic of a geometric absorption of the wave, the focus of the explanation is directed to a homogenous material.

In a homogeneous and elastic medium, the amplitude drops due to geometric absorption. This damping characterizes the type of wave. For example, the amplitude of a surface wave drops by a factor of $1/(\text{distance})^{½}$, whereas the amplitude of a body wave is damped by a factor of 1/distance (Jimeno 1995). As the type of wave dominates the ground movement several hundred metres away from the blast, it is important to know the type of expanding wave. Be it a body or a surface wave, the exponent of a trend line helps to describe the damping behavior of a wave.

6.2 *Results*

The analysis of the latest historical data (summer 2004 to spring 2005) showed that the higher a blast is situated in the quarry the more the blast tends to create vibrations. To verify that conclusion two geo-phone lines between the quarry and Eclépens were installed. After putting up the two geophone lines 33 blasts were recorded to verify the wave propagation behavior of the rock mass between the Mormont quarry and Eclépens.

The permanently installed geophones G1 and G2 were the starting points of the geophone lines 1 and 2 (figure 2). Each geophone line consisted of four geophones. These geophones were installed in line at places in the forest where they could be connected to the bed rock. These places were prepared with a hammer to get a horizontal surface and their position was determined by GPS. Each day the quarry blasted, the geophone lines were installed in the morning and dismantled in the evening.

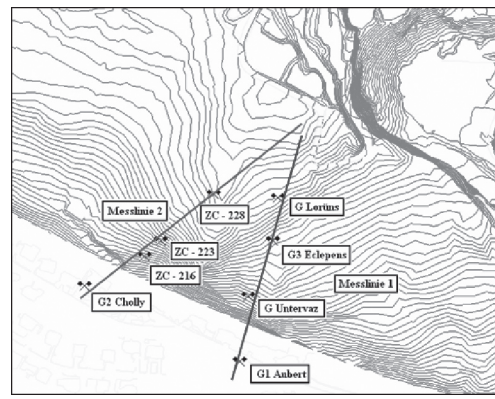

Figure 2. Geophone lines (line 1: blue and line 2: green).

Figure 3 gives an overview of the recorded vibrations with respect to the distance and the amount of ex-plosives per delay.

As it can be seen, the blast vibrations of all blasts recorded for geophone line 1 have the tendency to be slightly higher than that of line 2. The correlation coefficient of both geophone lines are with 0.6 (line 1) and 0.64 (line 2) more or less the same. In contrast, the exponent of both trend lines differs (0.78 vs. 1.21).

To understand the wave propagation behavior of the rock mass in detail, an analysis of the wave propaga-tion with respect to the bench number was carried out. Figure 4 shows the recorded blast vibrations of geophone line 1 with respect to the bench number.

The analysis clearly shows that the recorded data can be separated into two blast vibration

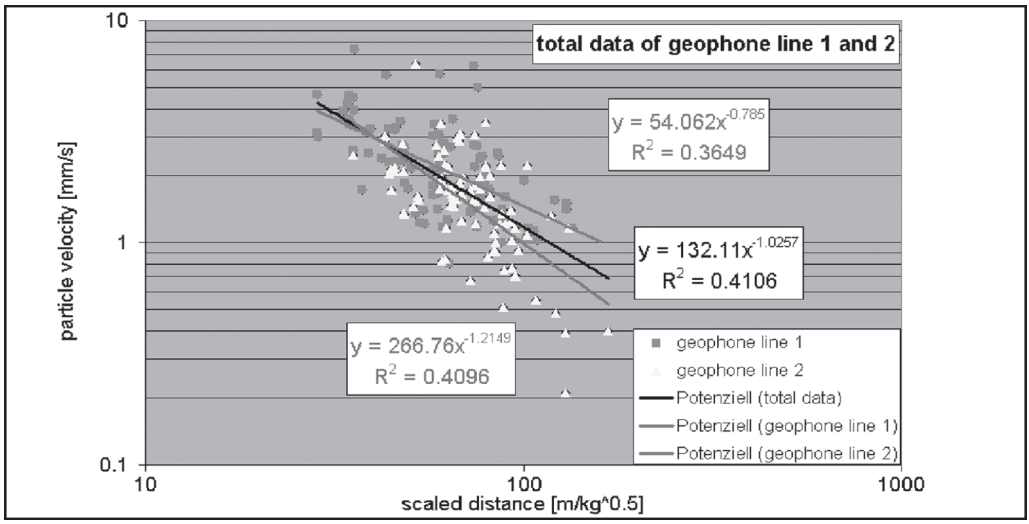

Figure 3. Blast vibration data vs. geophones line 1 and 2.

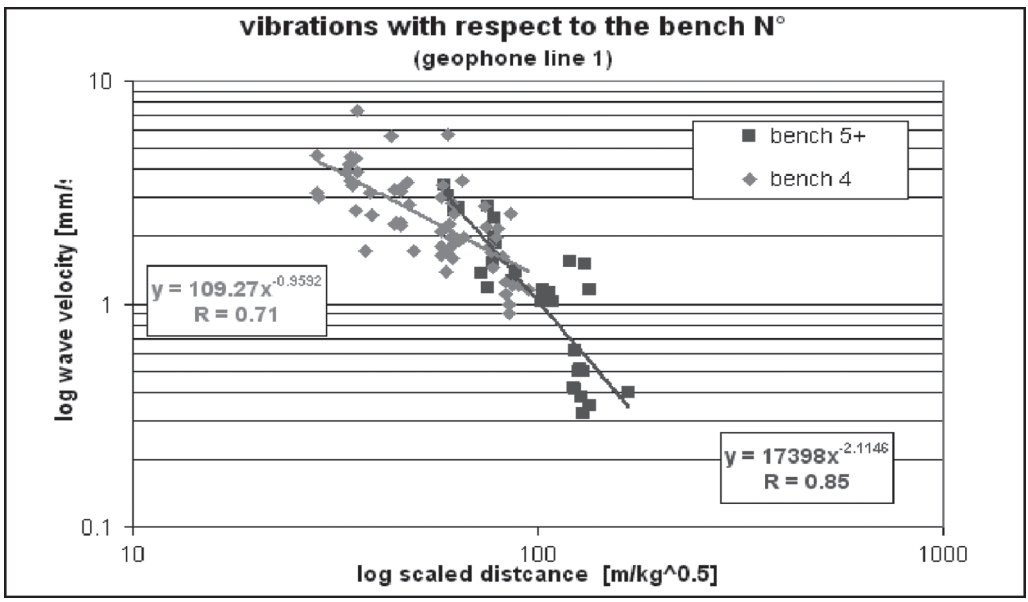

Figure 4. Blast vibrations with respect to bench number (geophone line 1).

'families'. The data of family 1 describes blast vibrations results of bench 4 (red). The correlation coefficient is 0.71. The exponent of the trend line is - 0.96 and the multiplier of the equation is about 110.

Although the correlation coefficient is with 0.85 also high, the recorded data of family 2 shows a totally different characteristic. The exponent of the trend line is with -2.12 distinctly higher and the multiplier has a power range of about 160 (109 to 17'398).

Due to that analysis it is obvious that at the Mormont quarry two different wave propagation phenomena exist. Vibrations generated at bench 5+ can be characterized by a high exponent of the trend line. The rock mass here damp the vibrations quite well, but the starting point is with 17,398 too high to reduce the blast vibrations. The opposite is the case for blasts of bench 4. The rock mass shows with an exponent of -- 0.96 a low damping behavior, but due to the low starting point of 109 the generated vibrations are more problematic.

7. CONCLUSION AND RECOMMENDATIONS

7.1 Conclusion

The working program 'concept carrière 2005' was of importance for Holcim Eclépens to understand bet-ter the blast vibration problematic of the Mormont quarry. The extensive data analysis mentioned above, lead to the following conclusion. The vibration problems Holcim Eclépens is suffering from are based heavily on the rock mass. Minor influence can be attributed to the blasting work done by the team, the bench orientation, the size of the blasts, the type of explosives used, etc.

7.2 Recommendations

The following recommendations were drawn and in the mean time realized by Holcim Eclépens:

- All blasts have to be measured and surveyed
- drill vertical boreholes to have a higher drilling quality (borehole length, orientation)
- reduce sub drilling to 0 - 0.8 m (more sub drilling is not needed)
- decrease bench height of all benches to 12 - 13 m (which will have an additional positive impact on the quality management of the raw material)
- increase number of blasts per year by changing the ignition system towards non electric detonators
- try to increase blasting pattern especially at the lower benches
- while having higher benches apply decked charges by using up to 4 detonators per borehole
- finally, use gravel (8-15 mm) instead of drilling dust for the stemming

Today, after more than 160 production blasts, Holcim Eclépens realized most of the recommendations given. As more than 98% of all production blasts created blast vibrations below 3 mm/s it can be stated that Holcim Eclépens did an excellent job.

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Ground vibration and air over pressure study during deep hole blasting for safe guarding chrome ore plant and dwellings – an Indian case study

V. M. S. R. Murthy & D. Kumar
Department of Mining Engineering, Indian School of Mines University, Dhanbad, India

A. Agrawal
Director, Mines, FACOR, Bhadrak, India

ABSTRACT: The Ostapal Chromite Mine, located in Orissa (India), is being operated for winning chromite ore using drilling and blasting technique. The mine is located in a flat terrain with important structures (Chrome ore plant-COB Plant) and dwellings lying within 100 m distance. A safe blasting methodology was desired to suggest the maximum explodable charge per delay (MCPD) for limiting the peak particle velocity (PPV) of ground vibration within safe and permissible limits. To meet the above stated objective 20 trial blasts were conducted and ground vibration and air overpressures were monitored with the help of 3 sets of Minimate/ Minimate plus seismographs. The initiating systems used were electric delay detonators, detonating cord and NONEL. The efficacy of each system was studied and the maximum permissible charge per delay was suggested.

Considering the required charge per hole of 15 to 20 kg for breaking rock effectively and the allowable PPV at the COB plant and the dwellings to be 25 mm/s and 15 mm/s, the safe distance upto which blasting can be carried out has been arrived. The study reveals that using conventional initiation systems the plant can be retained at the existing place till the quarry face reaches within a distance of 60 m with the maximum charge per delay limited to 17 kg. However using NONEL systems the plant can be retained at the existing place till the quarry reaches within a distance of 50 m with the maximum charge per delay limited to 17 kg (same as conventional). It is evident from the above investigation and analysis that use of NONEL is beneficial in general and when the distance between plant and the mine reduces below 60m in particular.

1. INTRODUCTION

Ostapal Chromite Mine of M/s Ferro Alloys Corporation Limited (FACOR) is located at village Gurajarga-Ostapal of Sukinda valley, Orissa, India. The mine lease area of the project is 72.843 Ha. The mine was opened in the year 1986. Table 1 depicts salient features of the mine. Mechanized extraction, involving deep hole drilling using 110 mm diameter crawler mounted drills with 5 to 6m high benches and hydraulic excavators of 1m^3 bucket capacity for loading onto 10 t tippers, is being practiced at Ostapal Chromite Mine (FACOR).

2. OBJECTIVE

The workings of open-pit mine falls very near to the COB plant and a residential colony of the Company. It is felt necessary to control blasting operations from three view points, namely, ground vibration, air over pressure and fly rock. The objective of the study was to suggest the maximum explodable charge per delay (MCPD) for limiting the Peak Particle Velocity (PPV) of ground vibration within permissible limits for both the Conventional and NONEL system of blasting for the safety of structures described above.

Table 1. Salient feature of the mine.

Name of the Mine	Ostapal Chromite Mines
Owner	M/s Ferroy Alloy Corpon. Ltd
Location	Ostapal of Sukinda valley, Orissa, India
Topo sheet no.	73G/16 & 73G/12
Latitude	21° 03' 29" to 21° 04' 03" N
Departure	85° 47' 13" to 85° 47' 44" E
Elevation	135 m RL to 158 m RL
Topography	Almost flat terrain with gentle slope
Leasehold area	72.843 Ha
Lithology	Laterite derived from ultramafics chemical weathering of ultramafics

3. METHODOLOGY

To meet the above stated objective field reconnaissance was done to identify the different formations and fragile locations such as benches facing human habitat and plant location. Accordingly a total of 20 trial blasts comprising of both conventional and non electric delay, were conducted and ground vibration and air overpressure were monitored in two phases of study with the help of 3 sets of Minimate/ Minimate plus seismographs.

The mine is being worked by mechanized drilling and blasting method with 6 to 7 m high benches. Details of drilling and blasting operations are described below:

3.1 Drilling & blasting

Crawler mounted DTH drills of 110 mm are being used to drill blast holes. The holes are made 15° to 20° inclined from vertical for maintaining uniform burden all along the hole apart from reducing the fly rock and toe formation due to effective utilization of free face. The fragmentation is also expected to be high in inclined holes. All these are achieved at the cost of a very marginal decrease in penetration rate.

Blasting was carried out using explosives, namely, Indoprime (as primer) and prilled ammonium nitrate (AN) with fuel oil (FO) (as column charge). About 13 - 21 kg of explosive was used per hole depending on bench height, rock formation and available bench width.

3.2 Initiation systems

– Conventional: detonating cord, detonating re lays (25ms), electric short delay detonators (0-9 nos)
– NONEL: Handydet having a delay interval of 200 ms down-the-hole delay and 25 ms surface delay between the holes, 25 ms/42ms between the rows

4. SUMMARY OF MONITORED BLASTS

During trial blasts the parameters recorded include:

– blast geometry
– rock properties
– peak and resultant peak particle velocity in the three orthogonal directions (mm/sec)
– dominant frequencies
– air overpressure
– wave form
– maximum charge per delay, per hole and per round
– distance of vibration recording instruments from the blast site
– initiation sequence and tie-ins
– delay periods
– visual observations of the distance of flying fragments
– charge distribution in holes and other relevant data

From the visual observation of blasts and inspection of site after blasting it was observed that the distance traveled by flying fragments varied from 20 to 60 m. Table 2 depicts the summary of trial blast parameters.

Part of mine plan showing the location of some of the trial blasts and vibration, air overpressure recording stations are shown in Figure 1(a & b).

Table 2. Summary of trial blast parameters.

Parameters	Value
Number of trial blasts:	20 (full scale spread over two phases of study)
Number of Blast Monitoring stations:	60
Explosives used:	Indoprime (as primer) and Prilled Ammonium Nitrate (AN) with Fuel Oil (FO) (as column charge)
Powder factor achieved:	2.19 to 3.78 m^3/kg of explosive
Depth of blast holes:	4.0 to 6.5 m
True burden:	2.5 to 3.0 m
True spacing:	3.0 to 3.5 m
Charge per round:	136 to 505 kg
Maximum charge per delay:	13 to 58 kg
The number of blast holes	per round: 9 to 36

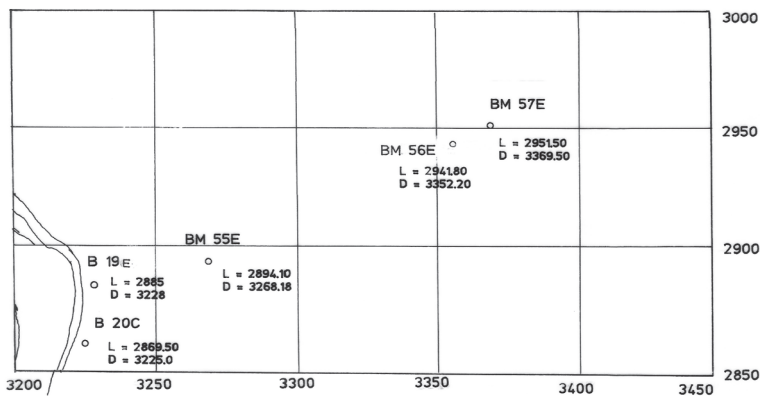


Figure 1a. B: Blast locations, BM: Blast monitoring stations, C: Using conventional system of initiations, E: NONEL system of initiation.

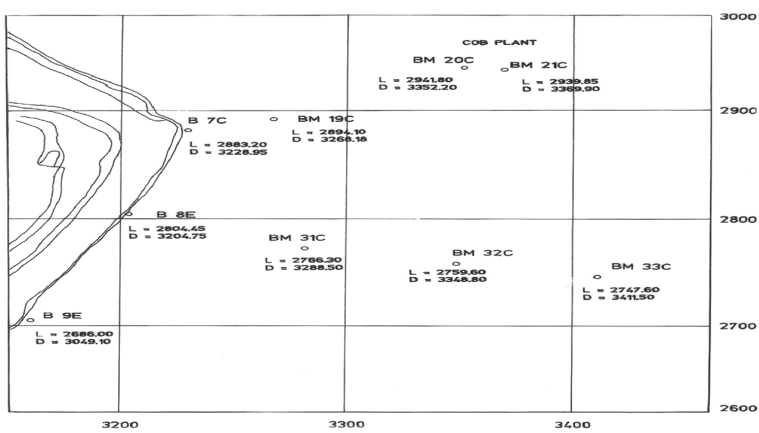


Figure 1b.

The initiation patterns and tie-ins are shown in Figure 2. The analyses of experimental blasts are presented in Table 3.

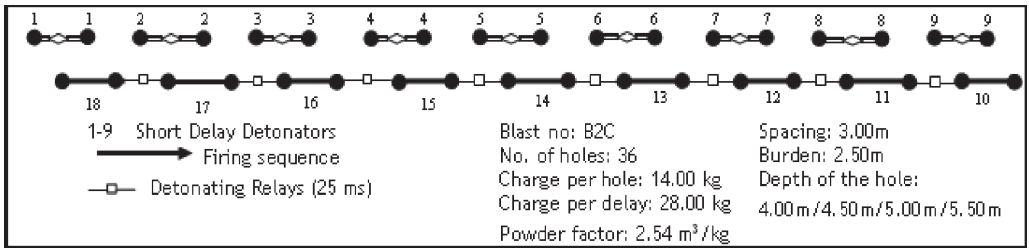


Figure 2a. In-line pattern conventional initiation systems (detonating fuse, detonating relays, and electric delay detonators).

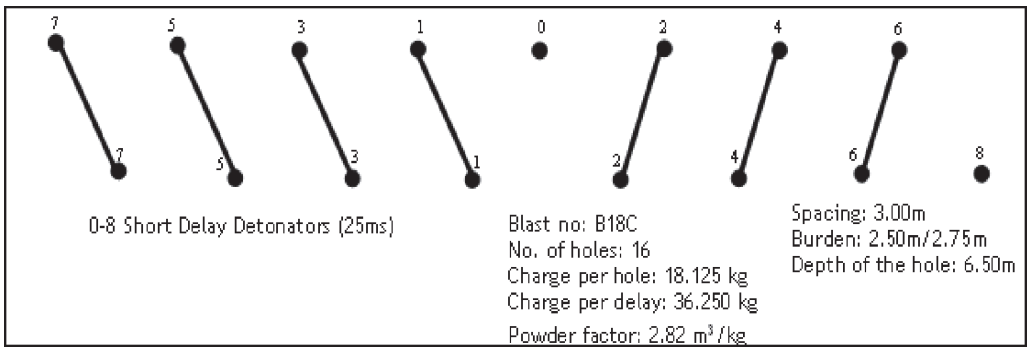


Figure 2b. V –pattern conventional initiation systems (detonating fuse, detonating relays, and electric delay detonators)

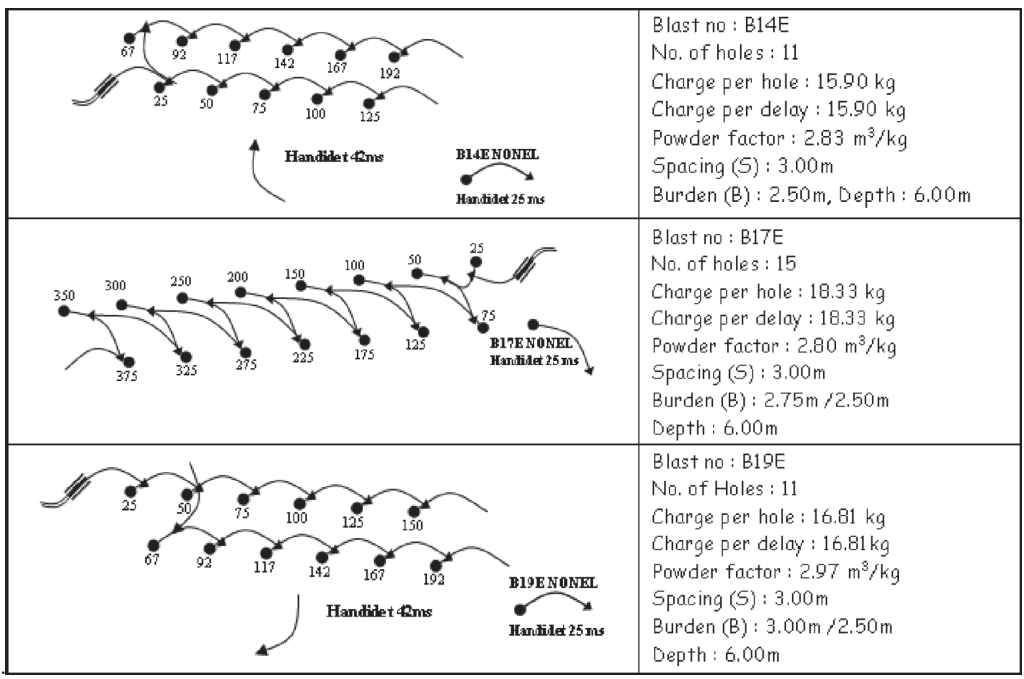


Figure 2c. In-line pattern & V pattern NONEL initiation systems (in-hole delays and trunk line delays).

Table 3. Analysis of blast monitoring data.

Parameter	Range	Class Interval	No. of events	Remarks
W Max.charge per delay (kg)	15-58	0-20	18	Majority of the events (46)utilized less than 40 kg of explosive per delay
		20-40	28	
		40-60	6	
PPV,(peak vector sum) Peak Particle Velocity (mm/s)	1.11-43.5	5 or below	36	44 events have shown a PPV less than 10 mm/s
		5-10	8	
		10-20	4	
		20-30	3	
		Above 30	1	
Time of occurrence of Peak vector sum (PVS) (ms)	1 to 288 ms	< 50	3	Majority of events (41) recorded pvs within a duration of 300 ms.
		50-100	11	
		100-150	9	
		150-200	5	
		200-300	13	
		Above 300	11	
Distance (m)	41-274	Blast site to monitoring station		
Frequency (Hz)	Transverse, Vertical, and Longitudinal			More than 75 % events recorded less than 25Hz Z_c (Zero crossing) frequency. This has a bearing on fixing allowable PPV.
	< 8	8-25	>25	
	60	58	38	
AOP (Air over pressure), dBL				All the events are below the safe limits of 140 dBL

5. GROUND VIBRATIONS & AIR OVER-PRESSURE MONITORING

When an explosive charge detonates in the blast hole, intense strain waves are transmitted to the surrounding rock. The energy carried by these strain waves known as strain energy, fragments the rock medium due to different breakage mechanisms such as crushing, radial cracking and reflection breakage in the presence of a free face. Combined, the crushed zone and radial fracture zone encompass a volume of permanently deformed rock. When the strain wave intensity diminishes to the level where no permanent deformation occurs in the rock mass (i.e. beyond the fragmentation zone), strain waves propagate through the medium in the form of elastic waves, oscillating the particles through which they travel. These waves in the elastic zone are known as ground vibrations, which closely conform to visco-elastic behavior (Ricker 1977).

In India DGMS suggested standards of blast vibrations in their circular No.7 of 1997 are being implemented. Accordingly, the permissible peak particle velocity (PPV) at the foundation level of structures in mining area is detailed in Table 4.

The analysis of ground vibration records as presented in Table 3 reveals that the zero crossing frequencies of the ground vibration in the Ostapal Chromite Mine varied from 3 to 65 Hz. It may be noted that frequencies less than 8 Hz were recorded in 60 observations, 58 observations were between 8 – 25 Hz and 38 observations were having a frequency more than 25 Hz. The zero-crossing frequencies recorded were utilized for fixing the

Table 4. Permissible peak particle velocity as per DGMS in India (1997).

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

Figure 3a.

Figure 3b.

Figure 3c.

Figure 3a. Monitoring of blast vibration and air overpressure with Minimate 077 Seismographs at COB plant.

Figure 3b. Monitoring of blast vibration and air overpressure with Minimate 077 Seismographs outside the employee quarters near mine site.

Figure 3c. Tie-ins pattern using Handidet NONEL during trial blasts.

allowable peak particle velocity at the structures. Considering the COB plant and the dwellings nearby as the main structures to be protected, the maximum allowable peak particle velocity (PPV) is fixed at 25 mm/s and 15 mm/s respectively based on the trial blasts and the response of structures during blasting. Accordingly Table 5 & 6 suggests maximum charges per delay in conventional initiating system (DF + EDD +DR) and NONEL initiation systems.

Figure 3 presents the plates showing tie-ins patterns, ground vibration monitoring stations etc.

Figure 4a.

Figure 4b.

Figure 4a. Rock fragmentation with conventional system of initiation (detonating fuse, detonating relays and EDD;

Figure 4b. Post blast rock fragmentation with NONEL system of Initiation.

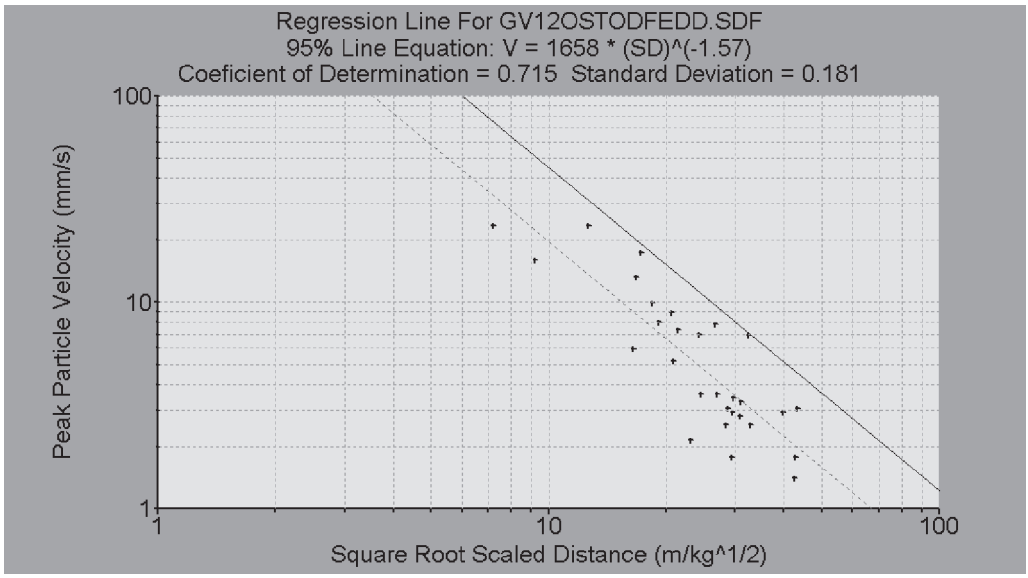
Figure 4 shows the post blast fragmentations using different initiation systems and formations.

5.1 Empirical relationship between ground vibration, distance and explosive charge

Applying the method of least square regression analysis, an empirical equation is established relating resultant peak particle velocity, V (mm/sec) of the ground, the distance of blasting site from the point of monitoring R (m) and the maximum charge per delay W (kg) (Figure 5). The derived empirical ground vibration predictor equation for the Ostapal Chromite Mine considering the entire conventional systems of experimental blasts is given below (Eq. 1):

$$V_{conv} = 1658^{*} \left(\frac{R}{\sqrt{W}}\right)^{-1.57} \quad (1)$$

Coefficient of determination (r^2) = 0.715



(DF: Detonating Fuse, EDD: Electronic Delay Detonators, DR: Detonating Relays)

Figure 5. Ground vibration predictor equation using Conventional-initiating systems.

The derived empirical ground vibration predictor equation considering the entire NONEL initiated experimental blasts is given below (Eq. 2) (Figure 6).

$$V_{NONEL} = 700 * \left(\frac{R}{\sqrt{W}}\right)^{-1.35} \quad (2)$$

Coefficient of determination = 0.518

While deriving the ground vibration predictor equations it was observed that the cube root scaling has yielded better correlation than square root scaling. Therefore, a comparative analysis has been carried out between cube root scaling and square root scaling in predicting the maximum charge delay with varying distance (Figure 7). It is seen from this analysis that the suggested maximum charge per delay is marginally low with cube root scaling for distances up to 75 m. Beyond 75 m the cube root scaling overestimates the charges. Since the workings are lying at distances more than 75 m from important structures (COB Plant & Dwellings) it is recommended to use square root scaling.

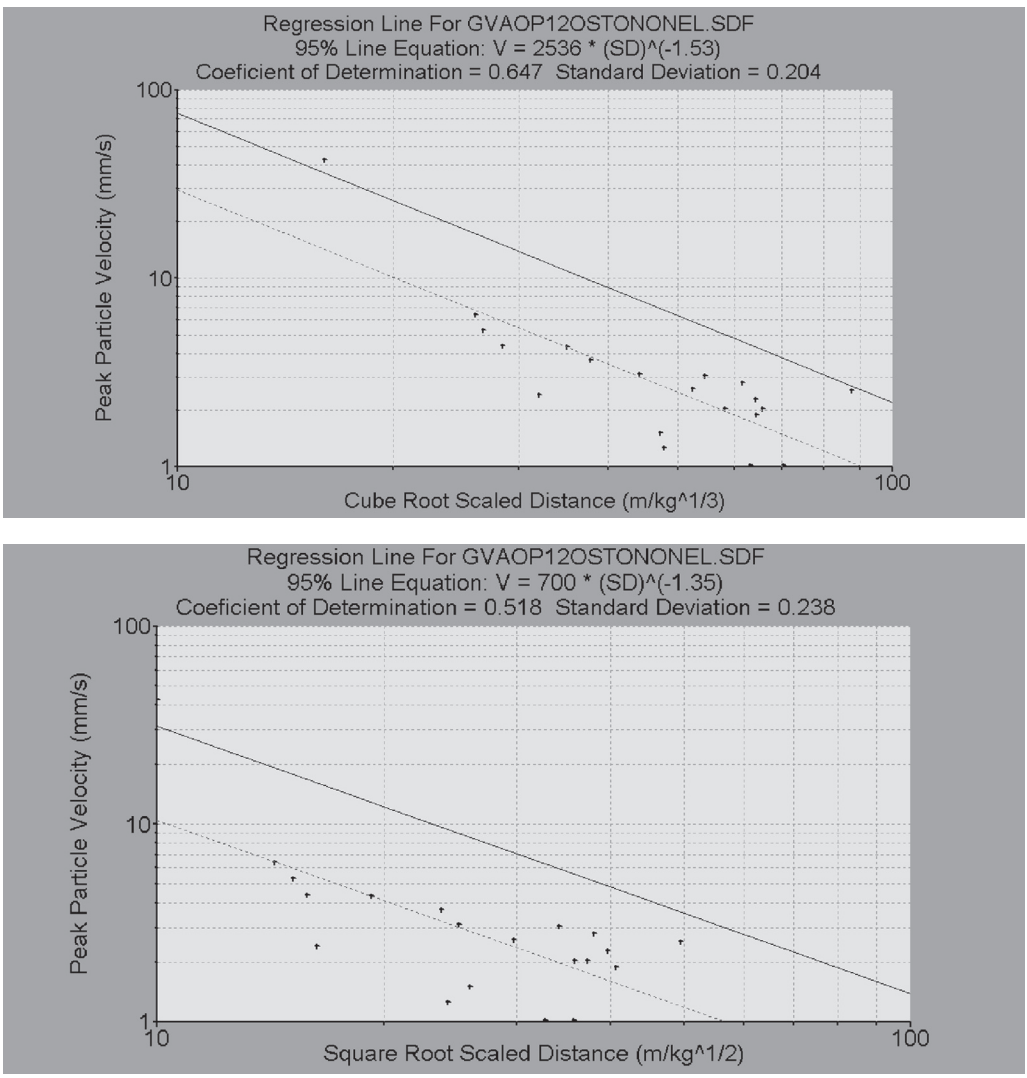


Figure 6. Ground vibration predictor equations using NONEL-initiating system (square and cube root scaling).

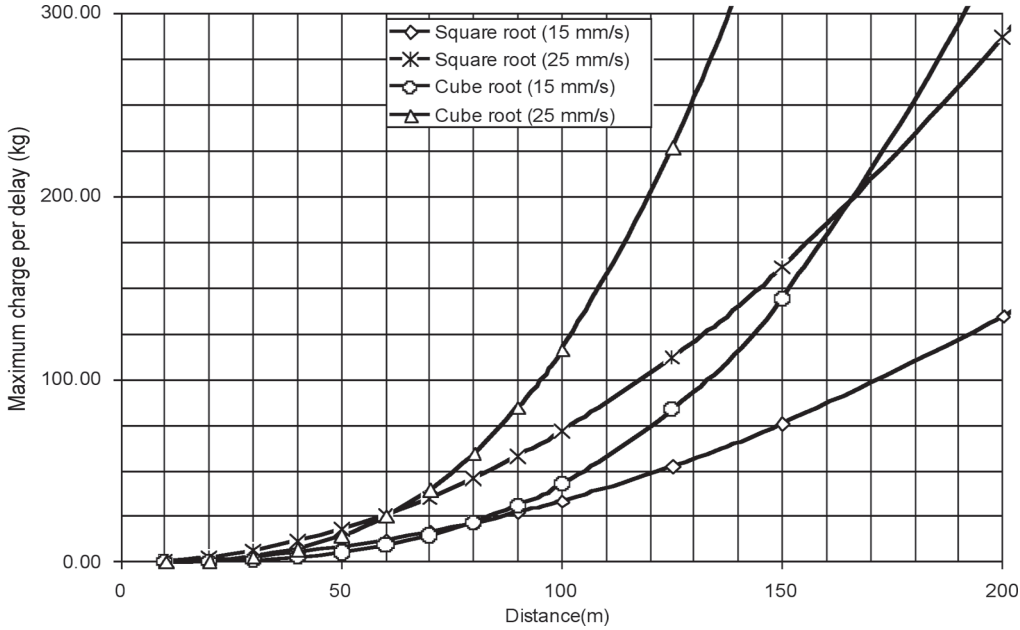


Figure 7. Analysis of square root & cube root scaling on suggested maximum charge per delay with increasing distance.

Table 6. Suggested maximum charges per delay in conventional initiating system (DF + EDD +DR).

Radial Distance (m)	Allowable PPV (mm/s) Dwellings	COB Plant
	15	25
10	0.25	0.48
20	1.00	2.00
30	2.00	4.00
40	4.00	8.00
50	6.00	12.00
60	9.00	17.00
70	12.00	23.00
80	16.00	31.00
90	20.00	39.00
100	25.00	48.00
125	39.00	75.00
150	56.00	108.00
200	100.00	191.00
250	156.00	299.00
300	224.00	430.00
400	399.00	765.00
500	623.00	1195.00

Table 7. Suggested maximum charge per delay in NONEL initiating system.

Radial Distance (m)	Allowable PPV (mm/s) Dwellings	COB Plant
	15	25
10	0.50	1.00
20	1.00	3.00
30	3.00	6.00
40	5.00	11.00
50	8.00	18.00
60	12.00	26.00
70	17.00	35.00
80	22.00	46.00
90	27.00	58.00
100	37.00	72.00
125	53.00	112.00
150	76.00	162.00
200	135.00	287.00
250	211.00	449.00
300	303.00	646.00
400	539.00	1149.00
500	842.00	1795.00

It may be mentioned here that the vibration predictors when established on the basis of initiation systems used (namely, DF, EDD, DR, NONEL) the results were found to be varying owing to the difference in progressive free face generation thus influencing ground vibration values. The scaled distance and peak particle velocity values are plotted on a log-log scale. Statistically the above equations have reasonable index of determination and therefore, can be accepted for prediction of ground vibration and estimation of safe charge per delay for various distances of structures from the blasting sites. The suggested maximum charge per delay using different initiating systems namely conventional and NONEL, for the safety of industrial structure (COB plant) and dwellings nearby (company quarters) has been given in the Table 6 and 7 respectively.

5.2 Air overpressure monitoring

During trial blasts air overpressure (air blast) was also monitored. The values obtained were analyzed for propagation equations and the same have been shown in Figure 8 and Figure 9 for both conventional and NONEL system of blasting respectively. It may be noted that the low correlation coefficient is due to the variations in the initiating systems, wind speed & direction with time of blast and lithological variations etc.

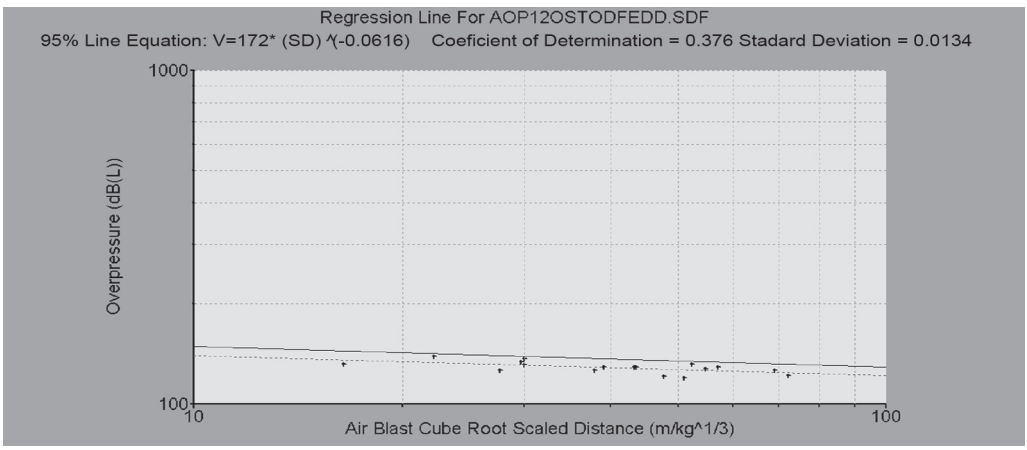


Figure 8. Air Vibration predictor equation using conventional initiating system (DF + EDD+DR)

(DF: detonating fuse, EDD: electrical delay detonator DR: detonating relays).

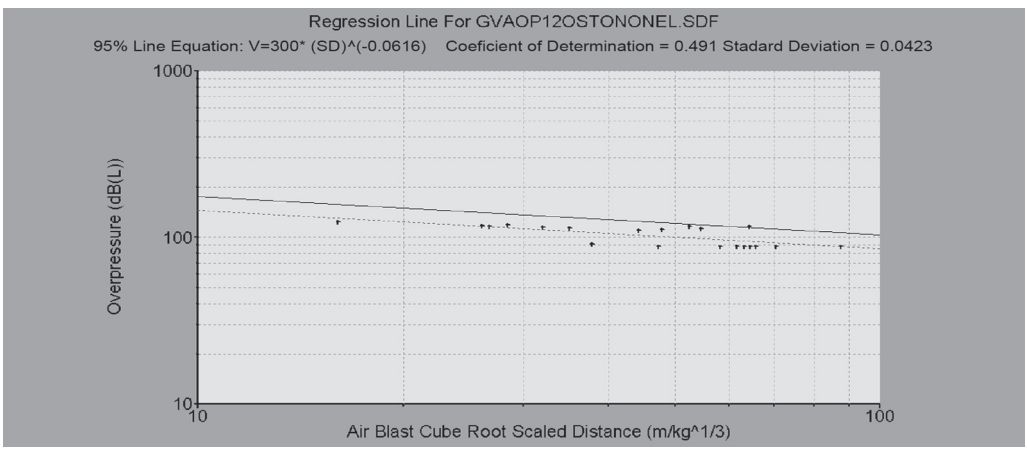


Figure 9. Air vibration predictor equation using NONEL initiating system.

6. CONCLUSION

Considering the charge per hole to be in the range of 15 to 20 kg for effective breaking of rock and the allowable PPV at the plant to be 25 mm/s and 15 mm/s near the dwellings, the safe distance upto which blasting can be carried out using different initiating systems is as under:

6.1 Using conventional systems

- The plant can be retained at the existing place till the quarry reaches upto 60 m distance from the plant. The maximum charge per delay in this case (at 60 m distance) would be 17 kg only
- Considering the dwellings, the safe distance up to which blasting can be carried out is 80 m with a maximum charge per delay of 16 kg (at 80 m)

6.2 Using NONEL systems

- The plant can be retained at the existing place till the quarry reaches upto 50 m distance from the plant with a maximum charge per delay of 18 kg. However when the distances fall below 50 m, air decking with separate in-hole delays of 175 ms and 200 ms for down the holes, 17 ms surface delays between the holes in a row and 25 ms surface delays between the rows should be practiced with the maximum explodable charge per delay strictly followed as per Table 7
- For the dwellings the safe distance is 70 m with a maximum charge per delay of 17 kg (at 70 m distance)
- It is evident from the above analysis that use of NONEL is recommended in general and when the distance between plant and the mine reduces below 50 m in particular

6.3 Suggested blast patterns with conventional & NONEL are shown in the Figure 10 and Figure 11 respectively.

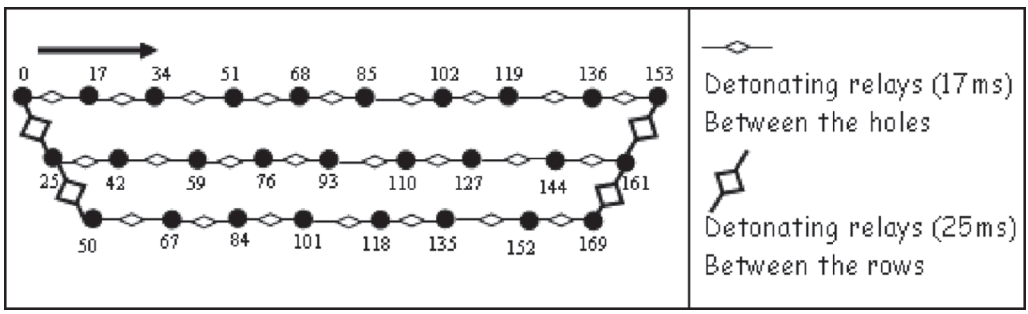


Figure 10. Suggested blast pattern for Conventional initiations systems.

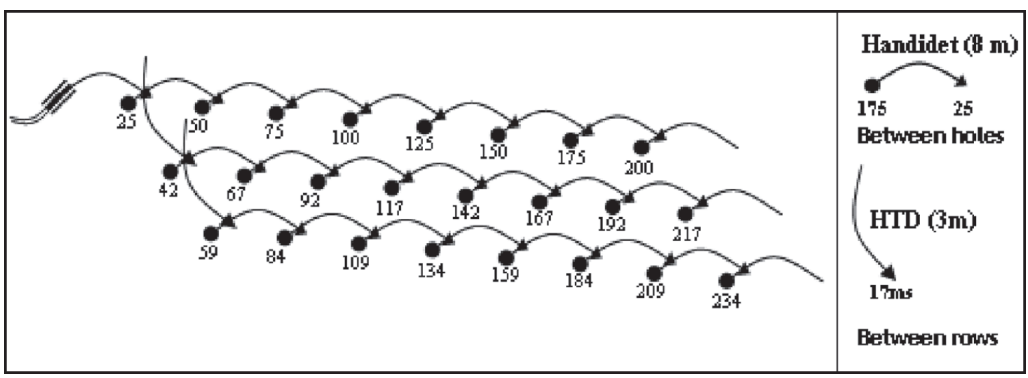


Figure 11. Suggested blast pattern for NONEL initiations systems.

6.4 Suggested blast parameters are shown in Table 8 below.

Table 8. Suggested blast parameters.

Parameters	Value
Hole Dia	110mm, Inclined 15° to 20o
Depth of blast holes	4.0 to 6.5 m (depending on bench height)
True burden	2 to 2.5 m
True spacing	2.5 to 3.0 m
Maximum charge per delay	As per Tables (conventional & NONEL)
The number of blast holes per round	30 to 40
Sub-grade drilling	0.2 times burden
Stemming column	1.2 – 1.5 times burden
Explosives	Indoprime (as primer) and Prilled Ammonium Nitrate (AN) with 6 % Fuel Oil (FO) (as column charge)
Initiation Systems and Patterns	As per the drawings

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Introducing a uniform blast management system for ground vibration control

T. Lewandowski
Enviro Strata Consulting, NSW, Australia

S. Wylie
Muswellbrook Coal Company, NSW, Australia

M. Merchant
Ashton Coal Operations, NSW, Australia

A. Roxburgh
Xstrata Mt Owen Pty Limited, Xstrata Coal (NSW) Pty Limited

ABSTRACT: A system for predicting vibrations, control and damage assessment of underground roadways has been developed into Blast Management Plan (BMP). BMP applied to underground mines located in close proximity to open cut blasting operations (i.e. less than 500 m). The system has a high level of transferability and was successfully implemented across five open cut / underground mining operations in Australia.

The system is designed to deal with the following issues:

– Assessment of the impact of vibration on rock strata conditions
– Detailed ground vibration predictions
– Management of underground personnel during blasting
– Vibration notification procedure

Expertise was developed to deal with the estimation of roof vibration limits, underground personnel management during blasting, assessment of the roof response due to vibration and underground personnel training.

This paper outlines how the system: Blast Management Plan, was implemented in three (of the five) different coal mining operations and how specific problems encountered by each individual mine were to be addressed.

1. INTRODUCTION

The recent mining boom in the coal mining industry has resulted in the development of a high number of underground mines in close proximity to open cut blasting operations (i.e. less than 500 m).

Increased safety requirements for the mining industry in Australia have created a new challenge

for underground mines related to vibration issues in the vicinity of open cut blasting operations. The main concern is related to ground vibrations and their impact on rock strata, and the exposure of underground personnel.

The expertise, to deal with the issue regarding blast management and the notification procedure, has been developed earlier by Lewandowski, Kelly and Weeks (2006). After the procedure was developed and was accepted by one mine, it has since been adopted into a uniform system capable of being implemented at other mining operations. The system is called the Blast Management Plan and has been implemented in several new mine settings.

This paper presents details of three separate case studies which involved the following mines:

- North Wambo Underground / Wambo Open cut
- Ashton Underground / Ashton Open cut
- Mt Owen Complex / Integra Underground

The introduction of a uniform system to deal with ground vibration issues still required the consideration of a number of issues specific to each particular mine. The work conducted in three different sets of coal mines revealed a high level of transferability. The common issues encountered for each mine which needed to be addressed included:

- The management and safety implications of high vibration levels for underground roadways
- The exposure of underground personnel to induced vibrations and management of underground personnel during blasting
- Underground personnel training to familiarise them with open cut blasting issues
- Development of blast notification procedure

The above issues are presented in the context of short case studies from each underground mine. Each case study is based on the introduction of the Blast Management Plan, a uniform system of rock strata assessment, vibration predictions and blast notification procedure.

2. STANDARD APPROACH TO GROUND VIBRATION ISSUES

A standard approach in dealing with ground vibration issues for underground mines was developed. The approach developed consisted of the following steps:

Step 1 – Rock Strata Assessment
Step 2 – Underground Vibration Monitoring
Step 3 – Development of Vibration Predictive Model
Step 4 – Blast Notification Procedure and Personnel Training

2.1 Step 1 – rock strata assessment

The rock strata assessment involves various types of evaluations. The initial assessment includes a site inspection. During the site visit a special emphasis is paid to observations of the roof and rib behavior underground. The aim of the site inspection is to determine the dominant roof and rib conditions. The roof is usually checked for signs of guttering and cracking. Factors such as roof sagging or extra weight on bolts are assessed. The roof behavior is assessed and compared to the roof behavior of other mines. During the inspection notes were taken in relation to these observations.

The rib behaviour is also evaluated. This involves an assessment of direction of coal cleats in relation to the roadway direction. For example if a dominant cleat direction is parallel to the wall there will be a high chance of rib spalling during vibration period. However, a perpendicular cleat direction to the roadway is beneficial as it impedes vibration transfer. Thus, the ribs are assessed in terms of their stability.

The presence of major geological features such as fault lines and dominant jointing in the roof is also of primary concern during the site inspection.

In summary the visual observations allow for the direct comparison of rock strata between various mines and allow for a more accurate assessment of vibration limits.

Prior to ground vibration exposure due to the open cut blasting, it is of paramount importance to assess the behavior of the roof in the context of existing geotechnical conditions. Generally, the roof response in the underground environment is a function of the stress level and local geology. Stress plays a significant part in inducing an impact on underground rock strata conditions. This includes the impact from the vertical, horizontal and / or mining induced stresses.

The other critical factor defining roof response is the dominant geology of the area. The potential impact of any geological weakness in the rock strata conditions should be considered. For example the presence of a weak layer in the immediate roof strata can result in potential roof control difficulties.

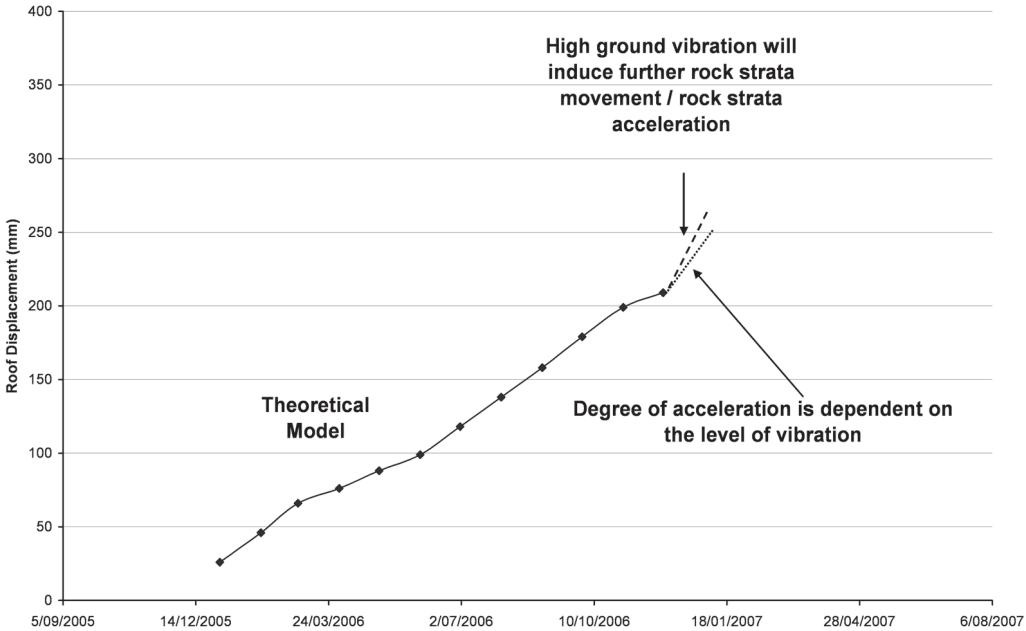

Figure 1. Impact of blasting – theoretical model.

Often visual observations such as an increased load on roof bolts and / or roof guttering and / or breakdown in the immediate roof beam can be observed. These conditions can often be alleviated by an increase in the bolting density.

An example showing the ongoing strata movement and impact of vibrations is shown in Figure 1. In the situation of the ongoing rock strata movement (i.e. an ongoing creep) the impact of vibration is considered as an additional accelerating factor. In essence, if the rock strata keeps creeping, no significant gains will be obtained by reducing vibration levels from say 100 mm/s to 50 mm/s. In both cases the rock strata movement will be accelerated to various degrees, refer to Figure 1. Consequently, prior to exposing the roof to any level of vibration it is important to understand a general roof response to given strata conditions.

In any rock studies the assessment of the geotechnical conditions should be supported by the empirical assessment for rock strata conditions. The information available to an assessor can include the following:

- Borehole log information
- Coal Mines Roof Rating (CMRR)
- Rock strength data
- Sonic velocity data
- Roof response data such as extensometer, borescope and tale-tell data

The inspection of borehole logs, rock strength data, CMRR and other information leads to the classification of the immediate roof. The immediate roof of interest for an underground mine comprises of the first 8 metres of the roof. The vibration impact on the upper rock strata is considered to be insignificant. This is due to the fact that the formation of the roof beam involves only the immediate roof horizon.

Due to the introduction of the roof bolting technology the working beam is usually limited to the first 2 metres of the immediate roof. Often the supporting beam could be as low as 0.6 metres located within the roof bolting horizon. The formation of the working beam is essential and will dictate the prevailing roof conditions. Thus the assessment of the immediate roof strata is very important in terms of the resulting stability and vibration limit estimations. An indicative vibration limit value for various rock strata based on other author's work are presented in Table 1.

Table 1. Summary of indicative vibration limits based on other author's work.

Study	Vibration Level (mm/s)	Findings / Comments	Roof Strata Assessment
Hendron 1977	460	Intermittent zone – tunnel failure	N/A
Rupert and Clark 1977	> 50	Localised thin spalls and collapse of previously fractured coal ribs	N/A
Fadeev et al 1987	240	For repeated blasting (service life up to 3 years)	N/A
	480	For one fold blasting	
Fourie and Green 1987	110	Produced minor damage	N/A
	390	Serious and extensive damage	
Persson 1990	700 - 1000	Proposed limits for hard rock strata	Hard rock
Singh et al 1995	48	No spalling from roof or coal pillars	N/A
Siskind et al 1997	305	Sufficient to induce new cracks or failure for tunnels and old workings	N/A
	254	Estimated limit for shotcrete linings	
Tunstall 1997	46	Minor visible damage	RMR – 49
	175	No damage in good quality rock	RMR – 85
	379	Major damage in poor quality rock	RMR – 49
Lewandowski et al 1999	50	Used as target limit at South Bulga	UCS (24-37 MPa), scattered single vertical joints
	146	No damage observed	
	300	Calculated safe vibration limit	
Singh et al 1999	100	General target limit for Indian mines	RMR – 45
	125.3	Spalling of coal from roof and pillars	RMR – 44.6
	296.7	Major crack developed in roof	RMR – 50.4
Lewandowski, Kelly, and Weeks 2006	10	Established as a permissible level of u/g roadway with personnel	Roof – UCS (25-101 MPa) Coal – UCS (12-21 MPa)
	20	Stone dust starts falling	
	96	No damage observed	
	250	Estimated as allowable vibration limit	

2.2 Step 2 – underground vibration monitoring

The objective of underground vibration monitoring is to capture the actual vibration level delivered to the underground roadway. It is therefore imperative to measure vibrations in the same manner for each blast. Various underground monitoring options were tested. This included recording of vibration measurements at the roof, rib and floor horizons (Lewandowski et al 1999).

The most suitable method identified was to take vibration measurements in the middle of the intersection. For that purpose a flat part of the roof was chosen and a geophone was glued directly to the rock strata. This ensured high monitoring accuracy. Comparative measurements between the centre of the roadway and the centre of the intersection revealed that the vibration levels at the roadway were lower than vibration levels at the intersection. This is mainly due to the difference in the length of the roof span. On average the length of the roof span for underground roadway is approximately 5.6 - 6 metres, while the length of the roof span for intersection is usually at around 9 metres.

The vibration measurements at the rib horizon

were very much dependent on the state of the ribs. This is mainly dictated by the cleat direction in relation to the roadway direction and the effectiveness of cutting. On average, vibration levels measured at the rib horizon were higher than at the roof horizon. The difficulty of collecting reliable vibration measurements at the rib horizon was in finding non-fractured ribs. Based on the results of the comparative tests the vibration measurements at the rib horizon level could not be considered reliable. This is mainly due to the nature of coal and the state of the ribs due to the impact of cutting.

Comparative vibration measurements between roof and floor horizons revealed that the floor stratum is usually exposed to lower levels than the roof horizon. As there is more risk attached to fracturing of the roof than the floor, the vibration measurements at the roof level were considered of primary importance.

2.3 Step 3 – development of vibration predictive model

The development of an accurate vibration predictive model is imperative for the success of the Blast Management Plan. For each presented case in this paper the ground vibration predictive model was based on the actual ground vibration measurements.

Initially, surface vibration monitoring data were used to formulate an interim site law for an area of interest. As soon as access to an underground mine was available the underground vibration measurements were taken. It has to be remembered that there are certain restrictions in the underground coal mining environment in Australia, which prevent taking non-approved apparatus into a hazardous zone area. The hazardous zone area was specified as the area within 100 metres radius of any operating face and all airway return roadways. Thus all underground measurements were taken outside the hazardous zone area.

The vibration measurements from underground mines were used to fine tune the predictive model. In addition, underground vibration measurements were analysed in the context of observed damage. An assessment of roof response to experienced vibration levels was also quantified using extensometers and tell-tale instruments.

An example of the format of the vibration predictive model is presented in Figures 2 and 3. The model consists of two parts. The first part includes the location of the blast, while the second part presents the predicted vibration for each intersection point of the mine and additional points of interests if needed.

In each case study, underground mine management was provided with the predictive model and guidelines on the permitted vibration levels which allowed personnel to stay underground.

In addition, the predictive model proved to be valuable to open cut mines personnel. It allows for the assessment of a blast design in relation to the permitted vibration levels prior to blasting.

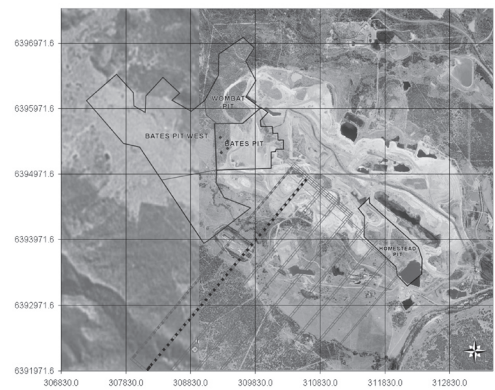

Figure 2. Example of vibration predictive model – part 1.

2.4 Step 4 – introduction of blast notification procedure and personnel training

The development of a blast management protocol required the introduction of a blast notification procedure. It ought to be noted that the blast notification procedure developed is very significant for a number of reasons. Most importantly the notification procedure allows the passage of information from open cut management to underground management and states information about blast time and its location. The supplied information also includes vibration estimations for various points of interest including underground intersections. The information is provided in a graphical as well as quantitative format.

Thus far, the blast notification procedure document can be used in the following manner:

- As an evidence document
- As a management tool for managing underground personnel and a sentry check
- As a management tool for minimising

Ground Vibration Estimation for North Wambo Underground

DATA INPUT - (only highlighted sections can be edited)

Blast I.D.		**B8WA8**				
Date		**21/05/2005**		**Time**	**4:45PM**	
Charge Mass (kg)		**1200**				
		No 1	**No 2**	**No 3**	**No 4**	**Approx Centroid**
Blast Coordinates	**Easting**	**309,405**	**309,400**	**309,309**	**309,292**	**309,352**
	Northing	**6,395,472**	**6,395,361**	**6,395,298**	**6,395,526**	**6,395,414**
	Base RL	**78**	**71**	**68**	**85**	**75.5**

OUTPUT - (do not alter this section)

Office	**Distance**	**Gr. Vibr.**
Intersection point	**1552**	**2.7**

TG1			**MG1**			**Main Headings**		
C/T	**Distance**	**Gr. Vibr.**	**C/T**	**Distance**	**Gr. Vibr.**	**C/T**	**Distance**	**Gr. Vibr.**
1	1037.5	**3.2**	1	1298.2	**2.2**	AP	1060.1	**3.1**
2	1007.2	**3.4**	2	1274.5	**2.3**	A1	1088.8	**3.0**
3	979.6	**3.5**	3	1252.6	**2.4**	A2		
4	961.6	**3.6**	4	1238.5	**2.4**	A3	1144.7	**2.7**
5	953.9	**3.7**	5	1232.6	**2.4**	A4	1180.9	**2.6**
6	956.7	**3.6**	6	1234.6	**2.4**	A5	1259.7	**2.3**
7	969.8	**3.6**	7	1244.9	**2.4**	A6	1311.8	**2.2**
8	992.3	**3.4**	8	1262.5	**2.3**	A7	1344.6	**2.1**
9	1018.4	**3.3**	9	1288.3	**2.3**	A8	1381.4	**2.0**
10	1049.0	**3.1**	10	1313.8	**2.2**	A9	1440.7	**1.9**
11	1088.2	**3.0**	11	1345.4	**2.1**	A10	1478.1	**1.8**
12	1135.1	**2.8**	12	1383.1	**2.0**			
13	1188.1	**2.6**	13	1427.4	**1.9**			
14	1247.1	**2.4**	14	1476.6	**1.8**	BP	1039.3	**3.2**
15	1311.3	**2.2**	15	1530.9	**1.7**	B1	1072.7	**3.0**
16	1411.5	**2.0**	16	1590.3	**1.6**	B2	1094.1	**2.9**
17	1484.6	**1.8**	17	1652.7	**1.5**	B3	1131.3	**2.8**
18	1526.5	**1.7**	18	1719.1	**1.4**	B4	1168.5	**2.6**
19	1604.3	**1.6**	19	1788.7	**1.3**	B5	1248.3	**2.4**
20	1684.9	**1.5**	20	1860.8	**1.3**	B6	1300.5	**2.2**
21	1766.8	**1.4**	21	1935.9	**1.2**	B7	1335.0	**2.1**
22	1850.5	**1.3**	22	2012.5	**1.1**	B8	1371.2	**2.0**
23	1936.2	**1.2**	23	2091.5	**1.0**	B9	1430.7	**1.9**
24	2023.1	**1.1**	24	2172.0	**1.0**	B10	1468.1	**1.8**
25	2111.2	**1.0**	25	2254.3	**0.9**			

Figure 3. Example of vibration predictive model – part 2.

interruption to underground activities

The timely distribution of the blast notification document was an important part of the process. In each study case email transmissions were selected as the most efficient method. The notification protocol was usually requested at least 24 hours prior to blasting. This time frame allowed for the information to be channeled through to underground management. For each document sent from the open cut to the underground mine management a confirmation response had to be received with permission for a blast at the specified time. Then, the underground management was obliged to take all necessary steps to manage underground personnel during blasting. This could involve limiting access through road blocking for a specific section of the

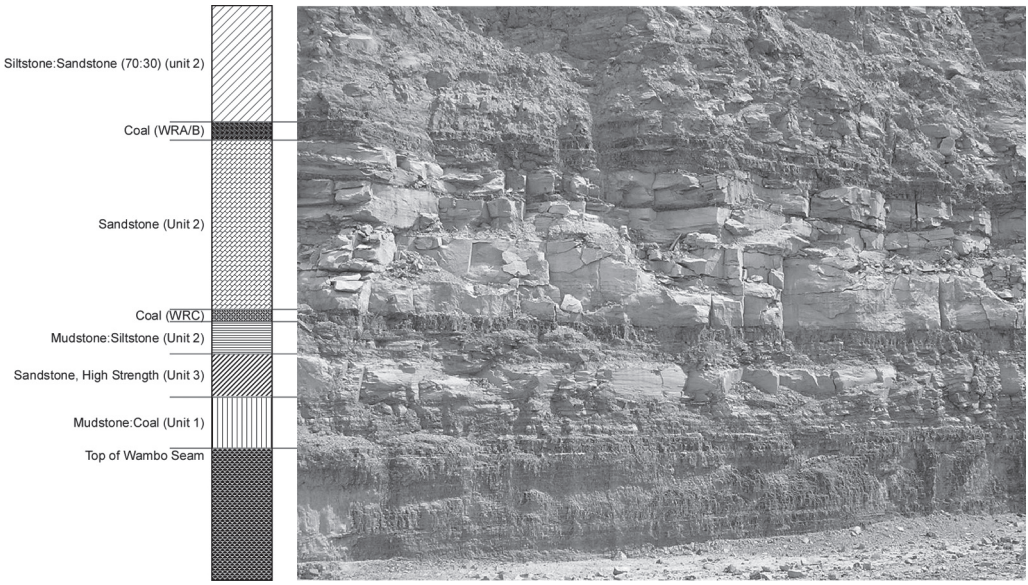

Figure 4. Rock strata cross-section – North Wambo underground mine.

mine or immediate inspection of the areas exposed to high vibration levels, and limit loss time for underground production activities due to surface blasting activity. In effect, the document is a two-way communication system between two different operators, that is, open cut and underground mine managements.

Training for underground personnel was essential to the success of the whole process. It is important for both underground and open cut management to be educated about the various levels of ground vibration and their impact on underground personnel and rock strata behavior. The two limits pertaining to underground personnel safety and rock strata stability are substantially different. Note that the recommended and actual ground vibration limit for personnel to remain underground is 10 mm/s. This is based on a detailed human response study (Lewandowski, Kelly and Weeks 2006).

Due to the increasing safety standards in Australian mines, relevant training modules for underground personnel had to be developed. These training modules included the following information:

- An assessment of impact of charge mass and distance on the level of vibration
- An assessment of impact of various vibration levels on the rock strata behavior, and
- An assessment of impact of vibration on underground personnel

The safety measures introduced, such as the Blast Management Plan, were fully explained to underground personnel. In summary the training modules allowed for better understanding of the safety implications for underground personnel and so resulted in better co-operation between open cut and underground mines. The trainings provided information and also removed misinformation related to blasting and associated risks related to it.

3. CASE STUDIES

3.1 Case 1 – North Wambo underground / Wambo open cut

This project was initiated by North Wambo underground mine management. North Wambo underground was originally designed to function as a longwall operation. The entry to the mine was from the highwall adjacent to the active open cut blasting operations.

The mine management sought expertise in the area of vibration management. The main issue which required to be addressed was the impact of ground vibration from the adjacent open cut

operation on the newly developed highwall mine.

The blasting in the open cut operation was performed as close as 350 metres to the underground workings, with blasting frequency of two to four blasts per week. The main concerns included:

- The impact of vibration on roof stability
- The impact of vibration on underground personnel and the management of underground personnel during blasting

The project consisted of the following three stages:

- Stage 1: The establishment of an allowable vibration limits in relation to the existing rock strata conditions and presence of underground personnel
- Stage 2: Underground vibration measurements and the assessment of the vibration transfer mechanism
- Stage 3: Development of a vibration predictive model including implementation and underground personnel training

The initial stage of the project involved the establishment of the allowable vibration limits for North Wambo underground. The task included the evaluation of the available roof lithology, rock strength data and geotechnical information. An example of the existing roof strata conditions is presented in Figure 4.

To approximate allowable vibration limits, information from various areas was collected and analysed. The source of information included: several borehole logs, UCS information, tale-tell roof displacement data, roof classification and roof response data based on site visit.

The estimated maximum allowable vibration limit was in the range of 300 - 350 mm/s. Values exceeding the above range theoretically will induce vibration damage on the dominant roof strata for North Wambo underground.

Next a series of underground vibration measurements, resulting from blasting of the adjacent rock strata in the Wambo open cut, were collected. These indicated that low vibration levels were generated. The vibration transfer mechanism identified was classified as indirect and could be described as very ineffective, see Figure 5. It was deduced that there were two factors which had a major impact on the effectiveness of the vibration transfer mechanism. These were: the discontinuity in the ground surface caused by the excavated ground in front of the highwall and the actual height of the highwall face.

As a consequence, the level of vibration measured for the underground mine was significantly reduced in comparison to other mines with similar geology but different topography.

Subsequently, a ground vibration predictive model and notification procedure were introduced. In addition a training program explaining the related vibration issues to the new workforce at North Wambo underground was introduced.

The study also revealed that the vibration level should not exceed 20 mm/s for the next four longwall blocks. It was concluded that for the estimated vibration level the roof should respond in an elastic manner and no damage to the strata is expected.

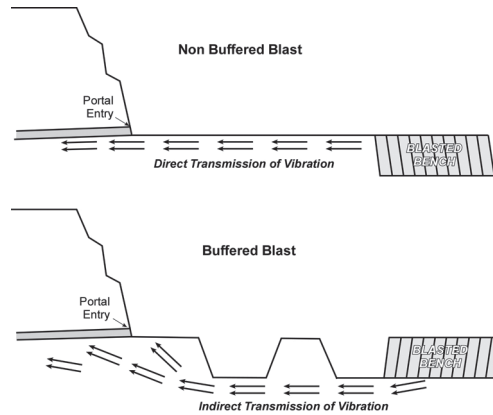

Figure 5. Schematic of vibration transfer mechanism.

3.2 Case 2 – Ashton open cut / Ashton underground

Similar to the other projects this project was initiated by concerns from the underground mine management in relation to the vibrations and their possible impact on the ventilation shaft. The main concerns were related to the possibility of the ventilation shaft being switched off after certain level of vibration was exceeded. Further questions were asked in relation to the impact of ground vibrations on the stability of underground rock strata and the implications for underground personnel. To address these issues a multifaceted investigation was undertaken including:

- An assessment of the impact of blasting on rock strata stability based on a geotechnical assessment

Table 2. Summary of UCS values for immediate roof – based on sonic velocity data.

Hole No.	Sample Description/Depth	Inferred UCS (MPa) (based on sonic velocity)
WML 009	Sandstone / Siltstone / Mudstone (seam at 143 m)	30 – 90 MPa – sandstone 10 – 20 MPa – mudstone
WML 013	Sandstone / Siltstone (seam at 127.5m)	40 – 87 MPa – sandstone
WML 004	Sandstone / Siltstone / Mudstone (seam at 84.2m)	40 – 70 MPa – sandstone 10 – 40 MPA – mudstone
WML 82	Sandstone / Siltstone (seam at 64m)	60 – 90 MPa – sandstone

- An assessment of levels of ground vibration generated. This included the development of the vibration predictive model
- An assessment of the impact of vibration on underground personnel

During the initial stage of the study the impact of vibration on the rock strata conditions was evaluated. A review of geological and geotechnical data was conducted utilizing the following information: several borehole logs, rock strength data, a standard classification of Coal Mines Roof Rating (CMRR), and a site visit. An example of the estimated Uniaxial Compressive Strength (UCS) values for the immediate roof is presented in Table 2. The UCS values were derived from sonic velocity data.

An assessment of the geology of the area was based on a number of borehole logs. A typical borehole log for Ashton underground is presented in Figure 6. The evaluation of the immediate roof did not reveal any significant weak bands with the exception of the mudstone band adjacent to the coal seam horizon. On average the UCS values were in the range of 30 - 90 MPa for the sandstone / siltstone band and 10 - 40 MPa for the mudstone band, see Figure 7. Based on these estimates the immediate roof was classified as moderately strong.

The site visit confirmed the above findings. There was no load on bolts or rib spalling observed. The inspection revealed only minor skin deterioration. The moderate roof strength conditions were further indicated by low bolting density i.e. minimum of 6 bolts per 1.5 metre as stipulated in the strata management plan.

A different type of rock strata assessment using the Coal Mines Roof Rating (CMRR) system was performed by an independent consultant. The analysis revealed that the immediate roof is also classified as 'moderately strong'.

Based on these results it was concluded that the immediate roof conditions are sufficient to sustain substantial vibration levels. It was estimated that the structural vibration limit for Ashton conditions is approximately 250 mm/s for the mudstone band and more than 350 mm/s for the sandstone / siltstone band.

The development of the predictive model was based on the ground vibration data collected from a number of monitoring stations. The model indicated that at that stage of the project (during 2006) the impact of blasting was relatively minor, see Figure 8. However, as time progresses, the blasting will advance towards the underground workings (to the West) and a more stringent protocol will be required. This will include personnel management during blasting. The management decisions will be based on the distance to the underground workings and vibration criteria. The recommended vibration limit for the underground personnel remaining underground during blasting is 10 mm/s. This will limit access to the areas with a predicted level of vibration higher than 10 mm/s.

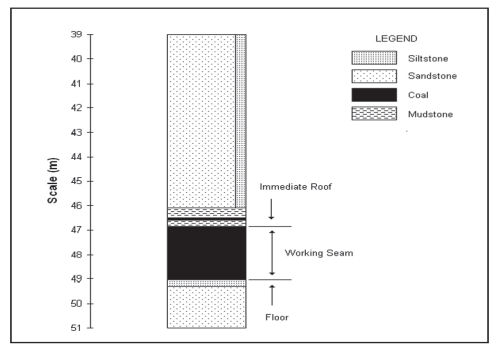

Figure 6. Typical borehole log from Ashton underground mine.

Figure 7. General view of highwall showing rock strata cross-section – Ashton underground.

Figure 8. Layout of Ashton open cut and Ashton underground.

3.3 Case 3 – Mt Owen complex / Integra underground

A detailed blast impact assessment study was requested by the management of Xstrata Mt Owen Pty Limited. The study was instigated due to a proposal for a new open cut pit development. The proposed open cut development was in the proximity of the existing underground mine, see Figure 9. The timing of the open cut development with regards to underground progression was critical in minimising the impact of vibration on the underground mine. In the least favorable scenario the proposed open cut mine was to be active and located directly above the underground workings. This would constitute a vertical distance between the mines of approximately 400 metres.

This study was designed to address a number of issues. These included the following:

- An assessment of the potential impact of blasting on the adjacent underground mine
- Development of underground vibration predictive model and notification procedure regarding upcoming blasts and their predicted vibrations
- Preparation of training and explanatory materials for management and underground personnel

One of the main challenges of this project was interacting between the management of the two different companies involved. These two companies represented two entirely different mining environments, that is, an open cut blasting operation and an underground longwall operation. The complexity of the interaction was due to the fact that both operators had limited knowledge about each other operational constraints.

Throughout the project the surface and underground mine management was to be fully informed of the results of the investigation. The knowledge also had to be passed to the underground workforce – who will be ultimately exposed to various levels of ground vibration.

The project included the management of blast impact on underground workings. The project was subdivided into three stages. These were as follows:

Stage 1:

- A detailed study regarding the blast impact assessment on underground workings

Stage 2:

- A full risk assessment conducted in the presence of representatives from the two mines, a consultant and an independent facilitator
- A number of information sessions explaining the effects of blasting on underground rock strata and underground personnel. These meetings involved representatives from senior management from both mines, a consultant and a representative from mines inspectorate
- An underground visit during the time of blasting at an underground mine with a similar blasting issue
- The development of training material for underground personnel explaining details of the study and effects of blasting on underground personnel

Stage 3:

- An underground vibration monitoring study involving the participation of underground personnel
- The implementation of the blast predictive model and the notification procedure regarding upcoming blasts and predicted vibrations

The project comprised a full technical study including assessment of the impact of blasting on the rock strata and assessment of human exposure to ground vibration. In this particular case the study presented a major challenge in terms of liaising with personnel of both mines and passing on the necessary information. The information transfer and distribution of knowledge regarding vibration issues turned out to be the main objective of this study.

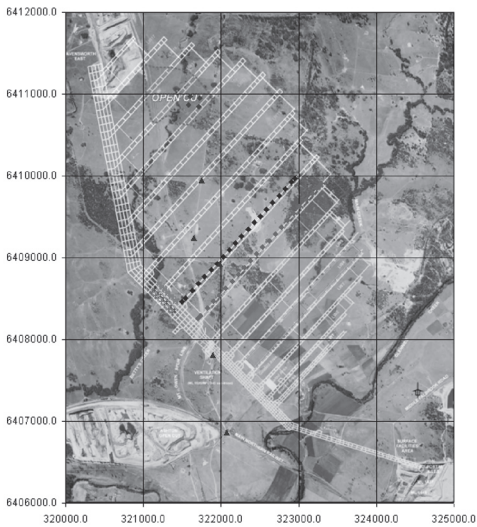

Figure 9. Location of Xstrata Mt Owen Pty Limited open cut pit and Integra underground mine.

4. CONCLUSIONS

This paper presents details of the Blast Management Plan developed for underground mines located in the proximity of open cut mines. The Blast Management Plan addresses the issue of ground vibration generated by open cut mines and their impact on nearby underground workings. The Blast Management Plan developed consists of the following steps:

- Rock Strata Assessment
- Underground Vibration Monitoring
- Development of Vibration Predictive Model
- Blast Notification Procedure and Personnel Training

The system developed provides a standardised program regarding how to deal with vibration issues. The approach developed appears to be quite successful today and has already been implemented across five different open cut / underground mining operations in New South Wales, Australia.

The paper describes experiences and challenges involved in the implementation of the Blast Management Plan in three different mine settings, namely:

- North Wambo underground / Wambo open cut mines
- Ashton underground /Ashton open cut mines
- Mt Owen complex / Integra underground mines

As described in this paper, each case had a different set of requirements, which had to be dealt with appropriately. The Blast Management Plan proved to have great transferability between mines and it contained the required degree of flexibility to allow for individual differences between mines to be adequately considered.

5. ACKNOWLEDGEMENTS

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Integra Underground - Sandy Tickell, Steve Kovac, Geoff MacKenzie

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Ashton Coal – Brian Wesley, Brian Chilcott, Paul

Gresham
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Measurement of blasting seismic effects during the excavation of the Frukov Krč Tunnel

Z. Deković
Motorway Zagreb-Macelj Ltd, Velika Ves bb, Lepajci, Croatia

M. Dobrilović & Z. Ester
Faculty of Mining, Geology and Petroleum Engineering, University of Zagreb, Pierottijeva 6, Zagreb, Croatia

ABSTRACT: The paper describes the method of measurement of blasting seismic effects during the excavation of the Frukov Krč Tunnel on the already existing Veliki Stog Tunnel. The immediate proximity of the existing state road D-1 and existing Veliki Stog Tunnel to the motorway under construction, and especially the construction of the new Frukov Krč Tunnel, set forth the method and dynamics of the tunnel excavation through blasting. Ground oscillation velocities were measured during the blasting in the left and right tunnel tube of the Frukov Krč Tunnel at four measurement points in the existing Veliki Stog Tunnel and at two measurement points in the nearby village. Through measurement of blasting seismic effects during the excavation of the Frukov Krč Tunnel it was possible to define such parameters of drilling and blasting in order to carry on with unobstructed and safe tunnel excavation.

1. INTRODUCTION

Zagreb – Macelj motorway is a part of the Pyhrn road route. The motorway is marked as A2 in Croatia, while it is classified in European road routes as E-59 connecting northern and central parts of Europe with its southern-eastern part. The Frukov Krč Tunnel is located in the last section of the Krapina-Macelj (Slovenian border) motorway. In this area the existing state road D-1 is situated very close to the new motorway. Figure 1 shows just how close the proposed and existing tunnel on the state road are. The excavation of the Frukov Krč Tunnel was carried out through drilling and blasting in accordance with NATM (New Austrian Tunnelling Method) starting from the southern tunnel portal towards the northern one. The Frukov Krč Tunnel has two parallel tunnel tubes with 25 m axis distance between the parallel tubes. The left tunnel tube is 341 m long, while the right tunnel tube is 354 m long. The surface of the tunnel's cross section during excavation amounts to app 78 m^2. The Veliki Stog Tunnel, located on the existing state road, has one 300m long tunnel tube which was constructed in the 1960s. The shortest distance between the existing (old) Veliki Stog Tunnel and the (new) Frukov Krč Tunnel is 16.5 m. The shortest mutual distance between the two tunnels is at the 30th metre of the excavation of the left tunnel tube, that is, at the 20th metre of the excavation of the right tunnel tube. Rock mass in which the excavation of the Frukov Krč Tunnel was carried out consists of the Macelj sandstone of the Lower Miocene which also

Figure 1. Location of the existing Veliki Stog Tunnel and the new Frukov Krč Tunnel.

Table 1. Detonation velocity of explosives and initiation devices.

Explosive	Detonation velocity (m/s)
Lambrex 1	5600
Lambrex 2 contour	4200
Austrogel G1	6000
Electric detonators Schaffler: moment TED-BRWP 0; millisecond MSD-MIZP 80	

represents the geological characteristic of that area. In tectonic sense this is an area of complex structure, that it, numerous faults are present, which has been proven during the excavation of the tunnel when six fault areas have been identified which represent at the same time intensively fragmented and weakened zones. The categorisation of the rock mass has been carried out in accordance with RMR classification and categories III, IV and V of the rock mass had been predicted.

2. DRILLING AND BLASTING

The excavation of the Frukov Krč Tunnel has been carried out in accordance with NATM (New Austrian Tunnelling Method). The basic features of drilling and blasting on the Frukov Krč Tunnel are the following: blasting is carried out by cartridged explosives and electric initiation system; drilling is limited with maximum of 3.0m depth, while the diameter of the blast hole is 45mm. Tunnel drill 'Atlas Copco Rocket Boomer L2' with two drill hands and one hand with platform was used for drilling of blast holes. Double wedge-cut was used in all categories of the excavation, while explosives 'Austrogel G1 Ø 30 mm' and 'Lambrex 1 Ø 35 mm' were used for charging cut (main) and auxiliary blast holes. Emulsion explosive 'Lambrex 2 contour Ø 25 mm' was used for contour blast holes. The characteristics of the used explosives and initiation devices are shown in Table 1.

The excavation of the tunnel in categories III and IV of the rock mass was carried out in two phases: by drilling and blasting. The surface of the cross section of the first phase of the excavation amounted to 55 m^2, while in the second phase it amounted to 23 m^2. The length of blast holes in category III of the rock mass amounted to 3.0 m, while the progress step amounted to 2.3 m, while the length of blast holes in category IV of the rock mass amounted to 2.3 m, and the progress step amounted to 1.8 m. Mechanical method of excavation was applied in category V of the rock mass. Figure 2 gives the blasting scheme of the first phase of the excavation in category III of the rock mass, while Figure 3 gives the blasting scheme of the first phase of the excavation in category IV of the rock mass. Figure 4 gives the blasting scheme of the second phase of the excavation.

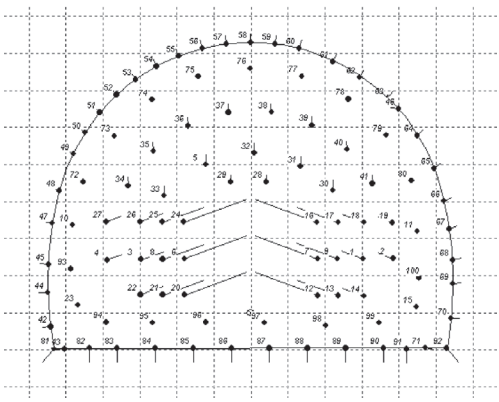

Figure 2. Blasting scheme-category III-first phase of excavation.

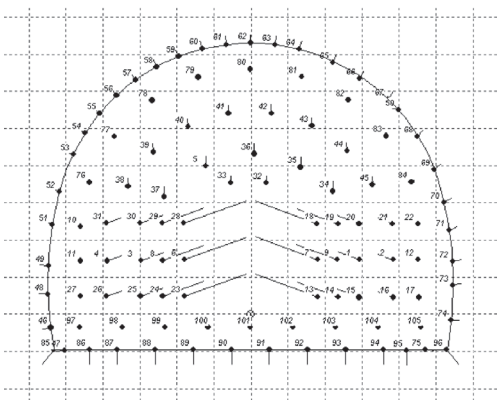

Figure 3. Blasting scheme-category IV-first phase of excavation.

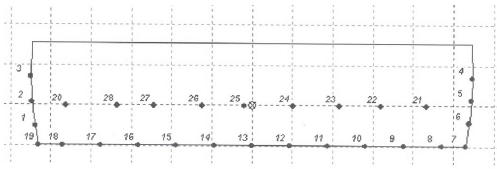

Figure 4. Blasting scheme-second phase of excavation.

3. MEASUREMENTS OF BLASTING SEISMIC EFFECTS

3.1 Oscillation caused by blasting

Energy liberated by initiation of the explosive charge in a blast hole manifests itself in two forms, as a shock pulse (dynamic part) and as energy of the expansion of explosion products (pseudo static part). The percentage of the specific form in the liberated energy also depends on explosive type, initiation mode, explosive charge (cartridged or pumped explosives) and quality of the blast hole plug. The shock pulse energy expands like an elastic wave radial in all directions starting from the blast hole and it can cause unwanted effects on the rock mass and tunnel support system. Oscillation velocity is proportional to rock deformation, and it is reliable indicator for determination of possible damages caused by blasting, which is used as marginal value in standards in order to define marginal values of allowed oscillation velocities for different types of constructions. The value of maximum allowed velocity for surface industrial objects, according to majority of the world standards, is 50 mm/s for ground oscillation frequencies greater than 50 Hz. There are no standards for underground facilities and tunnels. The quantities of explosive charge that does not cause damage of the rock mass and support system is defined according to the oscillation intensity as the function of the quantity of explosive charge per level of initiation, distance between the blast field and the support system, attenuation of seismic waves as the consequence of geological characteristics of a rock mass and maximum allowed value of ground oscillation.

3.2 Criteria for critical velocity of detonation

Amount of explosive charge that does not cause damage of the rock and supporting system is

estimated as follows:

- predicted oscillation intensity as a function of the amount of explosive charge per level of ignition
- distance between blasting field and supporting system
- attenuation of seismic waves as the consequence of the geological characteristics of a rock mass
- maximum allowed level of ground oscillation

Connection between maximum oscillation velocity at a measuring point, amount of explosive charge per level of initiation and a distance from a blasting point to a measuring point is given by empiric formula after M. A. Sadovski (Krsnik 1989):

$$v = k\left(\frac{\sqrt[3]{Q}}{R}\right)^{n} \text{(cm/s)} \qquad (1)$$

Where is: ν ground oscillation velocity (cm/s), Q amount of explosive charge used for detonation (kg), R distance between measuring point and blasting field (m), k and n constants depend on the geological characteristics of a rock mass based on the results of measuring of the oscillation velocities and consequential damages of rock mass in surroundings of the blast hole, a ground oscillation velocity was estimated for Swedish gneiss and granite which causes increase in rock deformations (Holmberg 1979). Damage was noticed up to 32 m away from the blasting point by velocity ν = 700 mm/s. Langefors and Kihlström suggested the level of ν = 300 mm/s for unsupported tunnels constructed in rocks like granite. I. D. Isaac suggested level of ν = 450 mm/s based on measurements taken by construction of underground engine room of the power plant Dinorwic (Isaac et al. 1981). T. R. Yu and S. Vongpaisal suggested the criterion for estimation of allowed ground oscillation velocity for underground constructions (Yu et al. 1998) given by blast damage index:

$$\text{BDI} = \frac{vdC}{K_t T} \text{(cm/s)} \qquad (2)$$

Where is: BDI (Blast Damage Index), ν ground oscillation velocity (m/s), d rock density (g/cm^3), C compressive pulse velocity in a rock (km/s), K_1 constant of rock quality, T dynamic compressive consistency of rock (MPa). The value of BDI index varies between 0.125, with no damage occurring, and 2 when great damage occurs related to ground oscillation velocity ν = 85 mm/s (no damage) and ν = 1750 mm/s (rock collapse). A. K. Chakraborty had applied the suggested criterion by construction of underground chambers in basalt rock and found out that the level of possible damages ν = 85 mm/s is far too low (Chakraborty et al. 1998). For the orientation about possible support structure damage, standard DIN 4150 is applied. Table 2 (subsection 1, under DIN 4150-3-1999-02) gives values of oscillation velocity for support structures, like armour-concrete columns, massive foundations and other.

Table 2. DIN 4150-3 1999-02, part 3.

	Maximum oscillation velocity V_{max} (mm/s)			
Frequency (Hz)	1-10	10-50	50-100	over 100
Support structures	40	40-80	80-100	min. 100

3.3 Hopkinson's effect theory

Based on the observation data we conclude that the ground oscillation velocities reached, at least at one detonation level, values above allowed velocities according to standards for industrial objects. Possible deformations increase in the rock mass on which measuring was done, can be defined upon analysis of cores before and after blasting or by seismic tomography. Nearly each blast triggered exfoliation of rock along the lower left side of the right tunnel pipe. This phenomenon, known as Hopkinson's effect, was noticed by Johansson et al. (1970) as an effect of cylindrical explosive charge on metal plate 15 cm thick. Later on, influence of Hopkinson's effect on rock samples was tested in a laboratory (McCarter et al.1993, Katsuyama et al 1993). Figure 5 explains the simplified influence of Hopkinson's effect which appears as a consequence of interference of pressure shock pulse H1 F1 (triggered by detonation) and a reflected compressive strain pulse of free surface H1" F1". The amount of strain is a sum of strains (Pa-Pb) of a compressive pulse and a reflected compressive pulse, and δ is a thickness of blasted rock. In case of sufficient kinetic energy exfoliation of rock oc-

Table 3: Characteristics of the blast fields of MP-1 and MP-2.

	MP-1	MP-2
Number of blast holes	105	100
Cut	V	V
Progress (m)	2,3	1,8
Blast Hole Diameter (mm)	45	45
Surface of tunnel cross section during excavation (m^2)	55	55
Initiation	electric	electric
Number of initiation levels	18	18
Retardation between ignition level (ms)	80	80
Maximum quantity of explosive charge per ignition level (kg)	26,60	33,60
Category of the rock mass	III	IV
chainage (tunnel meter (TM))	30,00	20,00
Total blasting time (ms)	1440	1440
Total explosive quantity (kg)	200,00	162,00

curs on a free surface. It is assumed that velocity of free surface movement above 8.0 cm/s causes exfoliation of rock on the tunnel side.

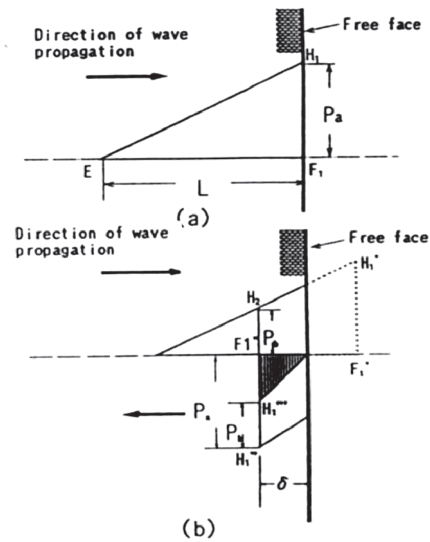

Figure 5. Theory of Hopkinson's effect.

3.4 Measurement of ground oscillation velocities

Measurement of blasting seismic effects was carried out during detonation of blast fields MP-1 in the left tunnel tube and MP-2 in the right tunnel tube. Blast field MP-1 is located at chainage of 30.00 TM (tunnel meter) in the left tunnel tube, and the distance between the tunnel axis of the left tube of the Frukov Krč Tunnel and the tunnel axis of the Veliki Stog Tunnel is 24.70 m. Blast field MP-2 is located at chainage 20.00 TM, and the distance between the tunnel axis of the right tube of the Frukov Krč Tunnel and the tunnel axis of the Veliki Stog Tunnel is 50.10 m. Table 3 shows technical characteristics of blast fields MP-1 and MP-2.

Ground oscillations occurred in course of blasting and they were measured by following types of transportable seismographs: White Digital Seismograph Mini-Seis II 2D2G, INSTANTEL Blast Mate II DS-447 INSTANTEL Blast Mate Series III and INSTANTEL Minimate, which are in accordance with standard ISO 9001. The instruments give automatic reading of the ground oscillation frequencies for each of the measured values and for all of the three components of the ground oscillation velocities by producing a diagram of standard DIN 4150 and of US standard USBM RI 8507-1980. Six measurement points were set for both occurrences of blasting, four in the concrete tunnel lining of the existing Veliki Stog Tunnel and two in the adjacent village Fruki. Instrument Instantel BM Series II was placed at

the first measurement point (MO-1) at a distance of 1.5 m from the carriageway in the eastern end of the tunnel which is closer to the blast field. Instrument Instantel Blastmate III was placed at the second measurement point (MO-2) at a distance of 4.5 m from the carriageway in the eastern end of the tunnel which is closer to the blast field. Instrument Instantel Minimate was placed at the third measurement point (MO-3) at the vault crown of the existing tunnel. Instrument Instantel Minimate was placed at the fourth measurement point (MO-4) at a distance of 1.5 m from the carriageway in the western end of the tunnel, while Instrument White Digital Seismograph Mini-Seis II 2D2G was placed at the fifth measurement point (MO-5) in the village of Fruki. Instrument White Digital Seismograph Mini-Seis II 2D2G was also placed at the sixth measurement point (MO-6) in the adjacent village of Fruki. Measurement points' distances from blast fields MP-1 and MP-2 are shown in Table 4.

Table 4: Measurement points' distances-blast fields MP-1 and MP-2.

Blast field	MP-1	MP-2
Distance of the blast filed from measurement profile [m]	26	53
Distance of blast field from MO-5 [m]	175	175
Distance of blast field from MO-6 [m]	200	203

Measurement profile in the existing (old) Veliki Stog Tunnel and measurement points (MO-5 and MO-6) in the adjacent village Fruki are shown in Figure 6. The position of measurement points in the existing (old) Veliki Stog Tunnel (MO-1, MO-2, MO-3 and MO-4) is shown in Figure 7.

3.5 Measurement results

Seismic intensity caused by blasting was defined on the basis of measured ground oscillation velocities at the measurement points. The analysis of seismic intensity for individual measurement point was carried out on the basis of the measured oscillation velocities in relation to the oscillation frequencies in accordance with standards USBM RI 8507-1980 and DIN 4150. The resultant ground oscillation velocities have been calculated on the basis of the vector sum of component oscillation velocities (each instrument measures maximum measurement resultant), that is:

$$v_r = \sqrt{v_t^2 + v_v^2 + v_l^2} \text{ (cm/s)} \quad (3)$$

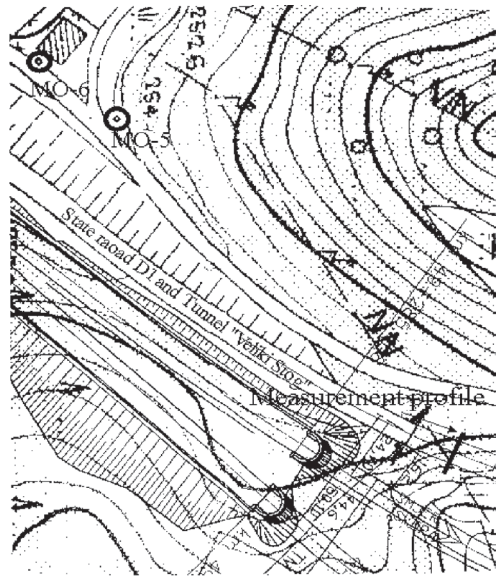

Figure 6. Measurement points-adjacent village and measurement profile-existing tunnel.

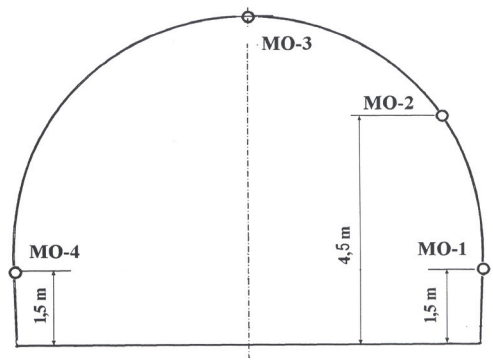

Figure 7. Position of measurement points-existing Veliki Stog Tunnel.

3.5.1 Overview of measurement results for blast field MP-1

The values of the ground oscillation velocities measured in course of blasting of blast field MP-1 are shown in Table 5, while

Table 5. Values of component and resultant oscillation velocities of MP-1.

Measurement point	Component ground oscillation velocity (cm/s) v_t	Component ground oscillation velocity (cm/s) v_v	Component ground oscillation velocity (cm/s) v_l	Resultant oscillation velocity (cm/s) v_r
MO-1	2.88	4.93	3.38	5.57
MO-2	1.38	1.04	2.96	3.08
MO-3	0.0826	0.00635	0.00635	0.0826
MO-4	0.406	1.07	0.311	1.12
MO-5	0.0508	0.127	0.0508	1.368
MO-6	<0.05	<0.05	<0.05	<0.05

Table 6. Values of component and resultant oscillation velocities of MP-2.

Measurement point	Component ground oscillation velocity (cm/s) v_t	Component gr ound oscillation velocity (cm/s) v_v	Component ground oscillation velocity (cm/s) v_l	Resultant oscillation velocity (cm/s) v_r
MO-1	0.597	0.940	0.521	1.02
MO-2	0.305	0.483	0.229	0.524
MO-4	0.0572	0.203	0.0699	0.210
MO-5	0.0508	0.0762	0.0762	0.1077
MO-6	<0.05	<0.05	<0.05	<0.05

trajectories of component velocities for measurement point MO-1 are shown in Figure 8.

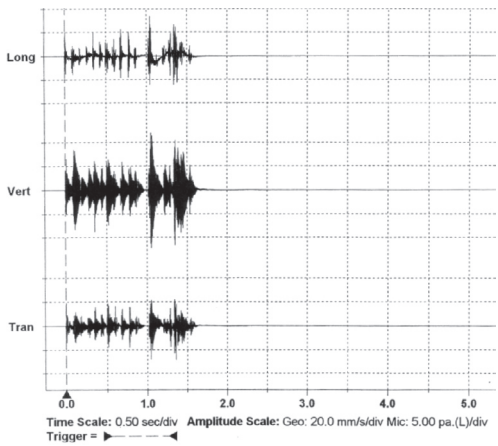

Figure 8. Blast field MP-1-measurement point MO-1.

It can be seen from Table 5 that in course of detonation of blast field MP-1 values of ground oscillation velocities measured at measurement points MO-1, MO-2, MO-3, MO-4 and MO-5 are below permitted oscillation values in accordance with standards DIN 4150 and USBM RI 8507. The measured values of the ground oscillation velocities at measurement point MO-6 (measurement point in the adjacent village Fruki) are below sensitivity of the instrument (<0.5 mm/s).

3.5.2 Overview of measurement results for blast field MP-2

The values of the ground oscillation velocities measured in course of blasting of blast field MP-2 are shown in Table 6, while trajectories of component velocities for measurement point MO-1 are shown in Figure 9.

It can be seen from Table 6 that in course of detonation of blast field MP-2 values of ground oscillation velocities measured at measurement points MO-1, MO-2, MO-3, MO-4 and MO-5 are below permitted oscillation values in accordance with standards DIN 4150 and USBM RI 8507. The

measured values of the ground oscillation velocities at measurement point MO-6 (measurement point in the adjacent village Fruki) are below sensitivity of the instrument (<0.5 mm/s).

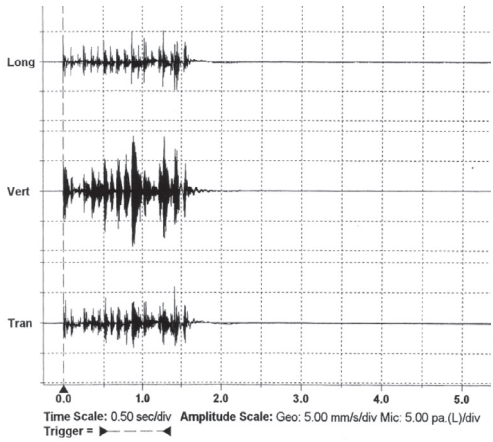

Figure 9. Blast field MP-2-measurement point MO-1.

4. CONCLUSION

The detrimental effect of blasting on adjacent facilities is assessed on the basis of permitted oscillation velocities, that is, on the basis of seismic intensities which are permitted for different types and states of facilities in accordance with standards USBM RI 8507-1980 and DIN 4150. All measured ground oscillation velocities (at all six measurement points) in course of detonation of blast fields MP-1 and MP-2 were within the range of permitted velocities in accordance with the above-mentioned standards. The measured values of resultant ground oscillation velocities and seismic intensity were below the permitted limits in accordance with standards USBM RI 8507-1980 and DIN 4150, therefore, detrimental effect of blasting during excavation of the Frukov Krč Tunnel on the facilities in the adjacent village and on the existing Veliki Stog Tunnel on the state road was excluded. Measurement results of the seismic effect of blasting during excavation of the Frukov Krč Tunnel have demonstrated that the blasting parameters' selection was adequate. Through systematic and correct approach to blasting parameters' planning during tunnel excavation and through continuous monitoring of seismic effects of blasting it was possible to avoid occurrence of any damage on the already constructed tunnel. Therefore, implementation of the supervision of blasting works and especially the implementation of monitoring and measurement of seismic effects of blasting is necessary because of the works' safety and because of the prevention of possible damages to adjacent facilities (in this case this referred to the already constructed tunnel) caused by blasting.

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