1. Blasting covering experience from projects

Budapest Conference Proceedings 2009, P. Moser et al
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Experiences of blasting demolition of structures

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ABSTRACT: Demolition of structures using blasting techniques began in many countries after the Second World War. For two decades this work was done by soldiers using their experience. After that it was also done by miners, who were skilled in blasting rocks. Now both soldiers and miners work in this area of engineering. There are only a few huge buildings requiring such demolition in Hungary. But blasting techniques are still used for demolition although to a lesser extent than earlier. A range of demolition projects carried out in the past decade will be demonstrated in the paper, with explanations of what took place.

1. CYLINDRICAL SILO

The cylindrical silo containing alumina in the former Aluminium Works in Tatabánya was located between two high buildings. Our duty was to drop the silo in question between these buildings. As some of the neighbouring buildings were as close as 1.5 m to the silo, the most important criteria was the exact direction of fall.

The silo stood on four wide pillars. The two front ones were drilled and blasted. Symmetrical drilling and charging was a most important requirement. Reinforcement of the back pillars was weakened by cutting approximately 0.2 m long pieces of the steel fibres from the internal surface of the pillars. This ensured that the back

Figure 1. The blast of the cylindrical silo.

pillars could bear the silo and it could move to the direction of the two front pillars. The steel fibres of the back pillars were torn when the structure fell. The destruction of the building was quite successful as the axis of the laying silo was just 0.3 m from its planned position.

Figure 2. The successful blast.

2. CONCRETE STACK

The date of the destruction of the 80 m high reinforced concrete stack of the Kispest power plant was delayed due to the complicated removal of an internal insulating layer made of asbestos. By the time this was complete a pipeline, located in the vicinity of the stack, was in use supplying hot water to the neighbouring housing estate.

Taking requirements and specialities into consideration the following procedure was selected:

An opening was made on the stack. The opening was as high as the charged part of the bottom of the stack. This made for easier removal of the asbestos and decreased the volume of concrete to be blasted.

All blast holes were drilled.

A structure protecting against flyrock was set up.

Blast holes were charged.

An 8 m long section was removed from the pipeline. Its ends were capped by steel plates.

The stack was blasted.

The removed section of the pipeline was replaced.

The heating was stopped only for two hours, there were no complaints.

Figure 3. Charging of blast holes.

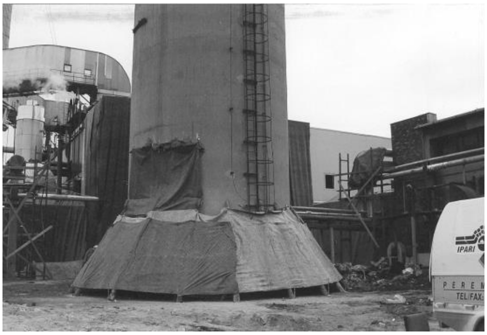

Figure 4. Protecting against Flyrock.

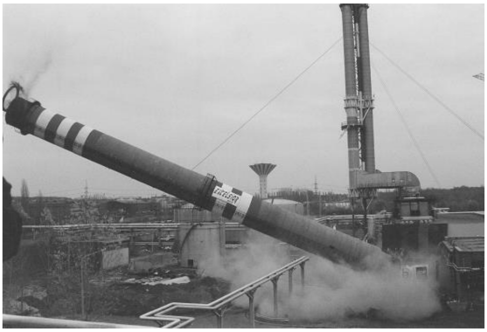

Figure 5. Steel ring at the top of the stack as it was pushed up by the shockwave.

3. POWER PLANT BUILDING WALL

One of our projects was the destruction of a main wall at the end of the central building as a part of reconstruction of the Inota Power Plant. Reconstruction of the structure and its associated machinery commenced on completion of the wall's removal.

The thin, reinforced concrete wall was built on two pillars which were wide at the bottom and narrowed as they got higher. The wall was opened manually to an appropriate height to enable a reduction in the number of blast holes required. Covers were put on the pillars to protect against flyrock. Blast holes were drilled only in the outer side of the columns.

A wedge-shaped slot was formed by the blast, so the wall could fall outwards. Fortunately not even one windows was damaged by (Figures 6. and 7.).

Figure 6.

Figure 7.

4. RADIO ANTENNAS

Listening to western radio broadcasts was forbidden in the former socialist countries, including Hungary. A number of jamming transmitters were built to block signals from western broadcasters. One of these operated in the central part of the country near Csévharaszt. The system consisted of approximately 120 antennas, each of them was 25-50 m high, and 4 towers of 114 m and 2 of 136 m high, connected with cables.

The lower steel towers was demolished using appropriate manual techniques. But blasting was the method chosen to remove the six high antennas to avoid any risk of unexpected electric shock. Non load-bearing bars on the bottom of the antenna were removed with oxygen cutting equipment. The other rods were cut by blasting, each of them at two points. Linear cumulative charges made of SEMTEX 1A explosive were put on the rods (steel pipes). As expected, each antenna fell in the planned direction.

5. HIGHWAY BRIDGE OVER RAIL LINE

A highway bridge over the international railway from FelsSzsolca to Hidasnémeti was rebuilt near Onga as the road from Miskolc to Szerencs was reconstructed and widened at the same time. Blasting techniques were used to remove the old bridge. This was a very sensitive operation as the rail traffic below the bridge was not allowed to stop during the demolition.

These were the phases of the operation:

A slot was made perpendicular to the axis of the road in the reinforced concrete plate of the bridge. The reinforcing steel fibres were cut with oxygen except the 5-5 fastened ones on both outer sides of the plate.

35 mm diameter PVC tubes were put in the slot which was then filled with concrete to make the structure solid. (The pipes created a space to be charged when the concrete solidified).

Earth was removed from both outer sides of the pillars.

Consoles (a projection from a wall to support a shelf) showing outwards were made of rails and timbers on the top of the pillars.

The consoles were loaded with earth which was removed from the outer side of the pillars.

Special cutting charges, made of hexotol were produced for an experimental blast.

Charges were placed to cut the fastened steel fibres and the concrete which was filled in the slot.

Then the surface of the bridge was covered and flyrock prevention was in place.

The blast cut the concrete plate at the slot and both pillars fell outside.

The basic concept of this method is filling concrete into the shot mode in the centre of the bridge to avoid any part of the structure falling on the railway. The fastened steel rods can bear the increasing force as both consoles made on the pillars are being loaded. Then first the reinforcing steel rods are cut by special cutting charges and subsequently the filled-in concrete itself with a 50 ms delay. Three pieces 0.4 m long cumulative charges of 700-800 g each were put on both sides of the bridge to cut the 5-5 cylindrical fastening rods. 45 spaces were formed in the filled-in concrete by putting 35 m diameter pipes made of PVC perpendicular to the axis of the bridge for the explosive. 150 g of Permon 10 was filled in each tube under the road surface and 200 g at the sidewalks. The initializing network was formed using electric detonators of type DEM. Charges cutting the steel rods were blown up first and were followed by the others with a 50 ms delay.

According to our calculations it was expected, that both pillars, loaded with earth on their consoles will move outwards as the concrete plate will be cut as a result of the explosion.

Figure 8. The southern side of the bridge to be destroyed.

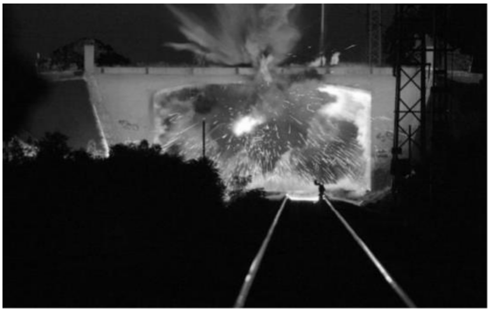

Figure 9. The moment of the blast.

Figure 10. First train after the blast.

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Rock mass damage assessment at granitic formation subjected to repeated blast vibrations at a hydroelectric construction project

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ABSTRACT: Rock blasting induced ground vibrations produce deformations in the vicinity of the blasting site. The effect of blast loading on structures is a growing concern of safety and stability. Extensive data is available on the behaviour of surface structures subjected to blast vibrations. However, only limited information is available on the effect of blast induced dynamic forces on the underground openings like tunnels and caverns. The reported findings state that blast induced dynamic stress, one or several cycles of repeated strains may cause deterioration in the rock mass or create damage to the dam foundation. This paper deals with the research work carried out at Jurala Hydroelectric Power Project (JHPP) on the effect of repeated blast vibrations on powerhouse foundation in a jointed rock mass. The damage caused by blast induced vibrations can be categorized into two types: i) near-field damage due to high frequency vibrations when the blast is occurring in the close proximity and ii) far-field damage due to low frequency vibrations when the blast is occurring at relatively further distances. The near-field damage was assessed by monitoring blast vibrations and borehole camera inspection survey. The far-field damage was assessed by continuous monitoring of vibrations, borehole camera inspection survey and P-wave velocity measurement by ultrasonic testing machine. A borehole camera was used to examine the crack extension and damage inspection of rock mass. The study reveals that repeated dynamic loading imparted on the jointed rock mass from subsequent blasts, in the vicinity, resulted in damage even at 22% of critical peak particle velocity (V_{max}). The far-field damage due to the repeated blast loading of 50-75 rounds was more than 89% of the near-field damage. The results of the study indicate that vibration levels, even at less than critical V_{max}, can cause safety and stability problems to the structures in/on jointed rock mass, when exposed to the repeated blast loading. The paper stresses the need for consideration of the effect of repeated blast loading in fixing the threshold limits of V_{max} to avoid both near-field and far-field damage.

(Keywords: Tunnelling, Repeated blasting, Rock mass damage, Peak particle velocity).

1. INTRODUCTION

The rock mass damage problem will be manifold if the blast loading is applied for a repeated number of times, in contrast to the conventional single episode blast loading. Repeated blast loading causes progressive accumulation of damage in joints which may lead to achievement of residual strength state in joints, with resultant large displacement at the joint surface (Brady 1990). Studies on blast induced damage on underground openings are well documented by many researchers globally (Langefors and Kihlstrom 1963; Hendron 1977; Holmberg 1993; Singh 1993; Paventi *et al.* 1996; Yu and Vongpaisal 1996; Chakraborty *et al.* 1998). In a series of experiments the Swedish Detonic Foundation has investigated the extent of cracking emanating from blastholes in controlled conditions (Olsson and Bergqvist 1996; Ouchterlony 1993; Ouchterlony 1997). In this paper, it was aimed at prediction and assessment of blast induced damage and deterioration due to repeated dynamic loading produced by opencut blasting on the nearby underground openings of an hydroelectric project. The effect of repeated blast loading on jointed rock mass was qualitatively studied by many researchers globally (Atchison and Pugliese 1964; Oriard 1989; Law *et al.* 2001). By stressing the need for the study on the repeated dynamic loading, Brady (1990) states that, substantial progress has not been attained in the study of repeated exposures of dynamic loading on jointed rock mass in comparison to conventional blasting with single episode of loading. It was reported that rock mass subjected to repeated blast loading resulted in relatively excessive damage than a single fold blasting (Otuonye 1997; Villaescusa 2004). Brown and Hudson (1974) state that rock mass damage by blast loading is predominantly due to joint motion, which is consistent with the experimental observation that joints decrease in shear strength under cyclic shear loading. This effect indicates pronounced modification of the peak-residual strength characteristic of a jointed rock specimen. Model studies of excavations in jointed rock under cyclic loading by Barton and Hansteen (1979) confirmed that excavation failure occurred by accumulation of shear displacements at joints. On the basis of these findings, St. John and Zahrah (1987) stated that, under dynamic loading, it is the number of excursions of joint motion into the plastic range that determines damage to an excavation. Wagner (1984) provided an indication of the general inadequacy of dynamic design based on V_{max} of single blast round. A possible conclusion with regards to dynamic behaviour under a range of V_{max} is that repeated dynamic loading may amplify problems of dynamic instability in jointed rock mass in the underground openings like tunnels and caverns. The major concern of this paper is to assess the effect of repeated blast vibrations on the damage intensity of jointed rock mass at the hydroelectric power project.

2. ASSESSMENT OF ROCK MASS DAMAGE DUE TO BLASTING

The damage caused by blast induced vibrations can be categorized into two types: i) near-field damage due to high frequency vibrations when the blast is occurring in close proximity and ii) far-field damage due to low frequency vibrations when the blast is occurring at relatively further distances. In case of opencut blasting, if the seismic source is within a distance of 25m from the monitoring point, that vibration is called a near-field vibration (Rustan 1998). Although far-field damage is not a severe problem in single tunnel excavations, it was observed by the authors as an acute problem when the rock mass is subjected to repeated vibrations due to multiple excavations in the vicinity. The study was carried out at Jurala Hydroelectric Power Project, India (JHPP), which is located approximately 200 km South of Hyderabad in Andhra Pradesh, India. Rock excavation near the JHPP dam was carried out for construction of an hydroelectric power project with 6x39 MW units. The JHPP project construction required to excavate about 75,000m3 of rock for making water conducting channels, called Vents (5 to 60m distance from the dam foundation) and about 0.1 million m3 of rock for making Tail Race Channel (TRC). A rock portion of 10m wide, 60m long and x 25m height left unexcavated in between the vents for separating different units and water conducting systems in front of the dam. These unexcavated rock structures were called rock ledges. The ledges are formed in front of the dam across its axis, as shown in Figure 1. Two utility tunnels passed through the ledges in perpendicular direction to the ledge axis, as shown in Figure 2. The utility tunnels driven in Ledge-I and Ledge-II are known as Tunnel-I and Tunnel-II, where the instrumentation for assessment of damage was installed. Bench blasting in different slices was planned for the excavation of Vents. There were 14-20 blasts conducted for each slice and there were 6-7 slices in each vent excavation. Therefore there would be about 200-250 blast rounds conducted in each vent excavation and there were 6 vents to be excavated. Thus, the rock mass at any part of Ledge is going to undergo blast loading from both the neighbouring vents and

TRC blasts for several times, which is going to induce dynamic loading on the utility tunnels. The vibration intensity of these blasts would range from 50 to 1500 mm/s at a distance range of 2-50m. The experimentation for determination of repeated vibrations on jointed rock mass was conducted at the inside the utility tunnels, which are situated at the foundation rock of the ledges.

3. GEOTECHNICAL DETAILS OF THE EXPERIMENTAL SITE

The rock mass at the ledges was moderately weathered to fresh and hard grey granite with thin pegmatite and dolerite dyke intrusions at the top 10-15m. The foundation rock was pink granite with fractures and weak joints (Figure 1.). The RQD of the rock mass just beside the foundation rock was determined as 45-55. Schmidt hammer rebound values were used to calculate in-situ properties of the rock mass. The rock formation was biotitic granite at first two ledges and pink granite at remaining ledges. The biotitic granite is relatively more sound rock than pink granite. The average RQD of the rock just beside the foundation was calculated as 50%. The damage assessment study was conducted at the Tunnel-I and II. The geo-technical properties of the intact rock observed at JHPP are given in Table 1.

Figure 1. View of dam foundation and blasting location at PJHPP.

4. DETAILS OF THE BLASTING AND INSTRUMENTATION

The production bench blast details for TRC and Vent excavations are given in Table 2. Instrumentation carried out in this study include, triaxial geophones for vibration monitoring, borehole cameras for observing crack extension loosening of joints and P-wave velocity measurements of rock mass.

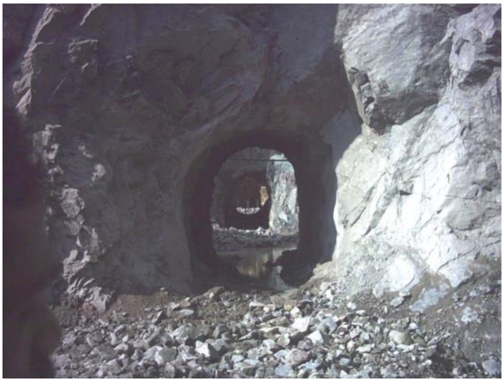


Figure 2. View of instrumentation tunnels driven across the ledges.

It has become common practice, recently, to use Vmax as an indicator of the potential for rock mass damage, as the Vmax is directly proportional to the dynamic strain (Jaeger & Cook 1979). Numerous authors used Vmax as criteria for blast damage in rock mass (Langefors and Kihlstrom 1963; Kutter and Fairhurst 1971; Holmberg and Persson 1978; Bauer and Calder 1978; Oriard 1982; Singh 1993; Andrieux *et al.* 1994; LeBlanc 1995; Yu and Vongpaisal 1996; Villaescusa *et al.* 2004). Application of borehole cameras for blast damage inspections has been reported by many authors globally (Beyer and Jacobs 1986; Stacey *et al*. 1990; Rocque *et al.* 1992; Singh 1993; Andrieux *et al.* 1994; Doucet *et al*. 1996; Liu *et al*. 1998).

Ultrasonic methods have been used to detect flaws in metals and concrete. Such tests were used in this study to estimate crack depths in rocks. Due to the inhomogeneous nature of the rock, only a few rock types such as rock salt and basalt are suited to flaw detection. This is because of the fact that acoustic velocity in a single material may vary over a large range due to the grain size and density variation (Koltonski and Malecki 1958).

The in situ values of the velocity also vary because of the pressure effect. An ultra sonic device called a ‘Telesonic’ instrument (Roop Telesonics-India) was used for the measurement of P waves of core samples. The instrument consists of a double probe (separate transmitter and

Table 1. Geotechnical properties of the intact rock at PJHPP.

S.No.	Rock type	Location	UCS, MPa	Young's Modulus, GPa	Tensile strength, MPa	P-wave Velocity, m/s
1	Granite (biotite)	Ledge-1	85.6	134	10.75	6800
2	Granite (biotite)	Ledge-2	68.2	117	7.25	6400
3	Granite (pink)	Ledge-3	58	88	9.5	5520
4	Granite (pink)	Ledge-4	47	79	6.7	5410

Table 2. Brief details of blast design for the production rounds of vents & TRC excavations.

Sl No.	Blast Parameter	Vent blasting with 32mm hole diameter	TRC blasting with 80mm hole diameter
1	Burden	2.0m	3.0m
2	Spacing	2.5m	4.5m
3	Hole depth	2.5m	5.0m
4	Charge per hole	4.17 kg	33.75 kg
5	Charge per delay	4.17 kg	67.5 kg
6	Specific charge	0.3 kg/m^3	0.5 kg/m^3

receiver combination) which can be moved separately on the surface. The frequency range of the transducer is between 1 and 1000 kHz. The technique uses the indirect method of recording travel time of ultrasonic waves across a crack. The principle behind the technique is that a sound wave travels from a transducer to a receiver along the shortest path. Presence of fractures or crack extensions makes the P-waves to either slow down or traversing a longer path. P-wave velocity (Vp) of rock mass was determined by fixing the ultrasonic transducer at the monitoring station and receiver in a borehole, specially made inside the ledge wall as shown in Figure 3. The axis of the borehole made for ultrasonic receiver is parallel to the axis of the tunnel. The Vp levels gradually fall down as the damage extends from the surface of the tunnel wall. A room of 0.5 m3 inside the ledge rock mass was made for installation of geophones to capture the blast vibrations from the surrounding excavations. A typical damage monitoring set-up with respect to blast site is shown in Figure 2., where the geophone room and borehole camera holes are shown. It was required to measure the blast vibrations in the near-field as well as far-field zones from blast sites to assess the rock mass damage. Therefore a room of 0.5 m3 (1m x 1m x0.5m) was excavated inside the ledge wall for installation of geophones at an approximate height of 1m from the bottom, as shown in Figure 3. The geophone room excavations were carried out by controlled blasting by using mild explosive charges to avoid disturbance to surrounding rock mass. The blasting zones ranged from 2 – 75m from the instrumentation location. The vibration monitoring was carried continuously, while the blasting face was proceeding towards the geophones from a distance of 75m. The geophone sensors of higher frequency and recording equipment with faster sampling rates were used for near-field measurements, and ordinary geophone sensors were used for far-field vibration monitoring. Borehole camera observation holes of 36mm diameter and 4m depth were drilled vertically downwards in the ledge rock mass, approximately 1m away from the geophone location (Figure 3.). The camera observation holes were placed close to the geophone holes, so that the possible rock mass damage levels can be correlated with the measured Vmax. The monitoring program consisted of surveying the observation hole before and after each blasting event. The borehole camera used in this study was a robust unit with semi-rigid fibreglass signal cable. This camera contains a standard video output and can be connected with any TV or video-recording system with VCR input. Borehole surveys were made using a front view lens attachment, which could capture images from all sides of the hole. This gave a clearer picture of the borehole wall, before and after blasting. All surveys were recorded in a computer attached to the camera and analyzed on the surface to determine the frequency of cracks and crack extensions before and after a blast.

5. EXPERIMENTATION ON THE EFFECT OF REPEATED BLAST LOADING

5.1 Near-field blast damage assessment

The near-field damage to the rock mass at the experimental station occurred due to the production blast rounds conducted at the Vents. The near-field damage was assessed by the Holmberg-Persson model (1978) as well as a borehole camera survey.

The principle of Holmberg-Persson Equation, is to add the contribution of every small portion of the explosives column along the full charge length to derive the Vmax at a fixed sensor location. More details about the Holmberg-Persson model can be known from the sited references (Andrieux *et al.* 1994; LeBlanc 1995; McKenzie *et al.* 1995; Meyer and Dunn 1996; Liu and Proulx 1996).

The Holmberg-Persson Equation can be simplified to,

$$V_{max} = K[a] \tag{1}$$

where, a is here defined as the Holmberg–Persson term and K and Y are the rock mass and explosive specific attenuation constants.

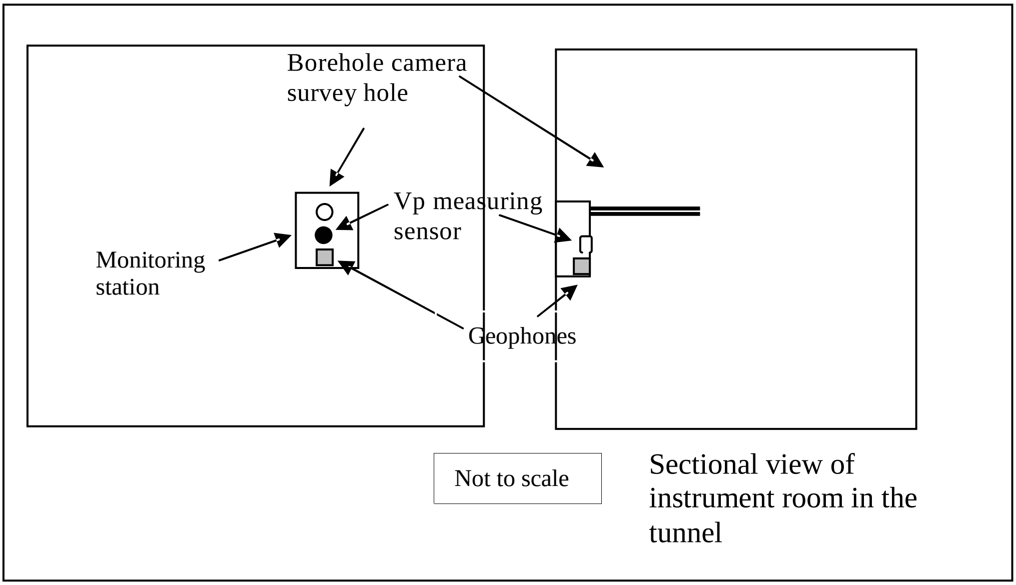


Figure 3. Schematic of installation locations of geophones and borehole camera survey holes and seismic survey holes.

K and Y can be obtained by linear regression from experimental data vibration and distance. The mean values of K and Y show the general trend of vibration attenuation in the rock mass.

In this study, The Holmberg–Persson approach was applied to determine the site specific constants K and Y to model peak particle velocity attenuation across rock mass. The same type of explosive and design parameters were used for all the experiments and analysis of the results are presented in the following paragraphs.

Near-field peak particle velocity (Vmax) measurements were grouped and analysed separately for two rock ledges. Typical log–log plot of the measured Vmax values obtained in this study against the Holmberg–Persson term. The Vmax amplitudes were being experienced at similar distances and for the same design parameters. This plot is used to determine the K and Y constants by fitting the linear relationship of the form, Log(Vmax)= Y Log(a)+Log(K). Results of the analysis are summarized in Table 3. The calculated K and Y constants were slightly different for the Tunnel-I and II. After determining the site specific attenuation constants, preliminary predictions of the extent of blast damage into wall-rock were made by applying the Holmberg–Persson model and by considering a site specific critical Vmax or damage threshold (Vcr) given by the following relationship (Persson *et al.*1994),

$$V_{cr} = \frac{{}_T V_p}{E} \qquad (2)$$

where,

V_{cr} = Critical peak particle velocity before tensile failure (mm/s);

T = Uniaxial tensile strength of rock (Pa);

V_p = Compressional wave velocity in rock mass (mm/s);

E = Young's Modulus of rock (Pa).

From the properties described in Section 4 (Table 1.) and by adopting the above relationship, the value of damage threshold, Vcr, for the rock mass at Tunnel-I was 590mm/s and at Tunnel-II was 460mm/s. These threshold values were used to compare the extent of damage caused by the near-field blast rounds, which obviously generated maximum peak particle velocity. Figure 4. shows the results of this analysis. The above analysis indicates that on average the extent of blast induced damage was 1.35m and 1.58m at the Tunnel-I and II, respectively.

The damage levels assessed by Holmberg-Persson model were cross checked by borehole camera survey before and after each blast round. An initial pre-blasting survey was performed in each hole to take pictures of pre-existing structural features for comparison with post blast surveys. This survey served as the reference survey in which pictures were taken at several centimetre intervals for comparison with the same pictures taken after each subsequent blast. A total of 2 borehole camera surveys were taken, one each at two ledges. All the pictures were sorted out first by editing and matching the same images before and after each blast. The images of observation holes captured by borehole cameras clearly indicated that the near-field damage due to production blasts extended up to 1.51m and 1.85m at Tunnel-I and II, respectively. Near-field damage observations of both the methods are given in Table 3.

5.2 Far-field blast damage due to repeated vibrations

The multiple rounds of blasting activity were carried out for the Vents and TRC of the proposed power house (Figure 1.). Far-field rock mass damage observations were carried out by using a borehole camera and by ultrasonic velocity measurements of pre-blast and post-blast rock mass. The Vmax levels were also recorded for every blast round until the vibration intensity attenuated to about 50mm/s. The observation holes were under continuous monitoring with the borehole camera for about 65 rounds of blasts at the surrounding Vents (I and II), Tunnel and TRC blasts. As the blast site is moving away from the monitoring point the vibration intensity, obviously, reduced gradually. The effect of these reduced vibration levels with the repeated number of exposures on the extent of further damage was studied by the borehole camera. Most of the blast rounds executed for the above mentioned excavations were monitored continuously by seismographs and borehole camera. The vibration range of 50-1500 mm/s was monitored at both the Tunnel-I and II. The near-field vibrations with high intensity were monitored by high frequency geophones, and low intensity vibrations were monitored by low frequency geophones. The blast damage assessments at Tunnel-I and II are discussed in the following sections separately.

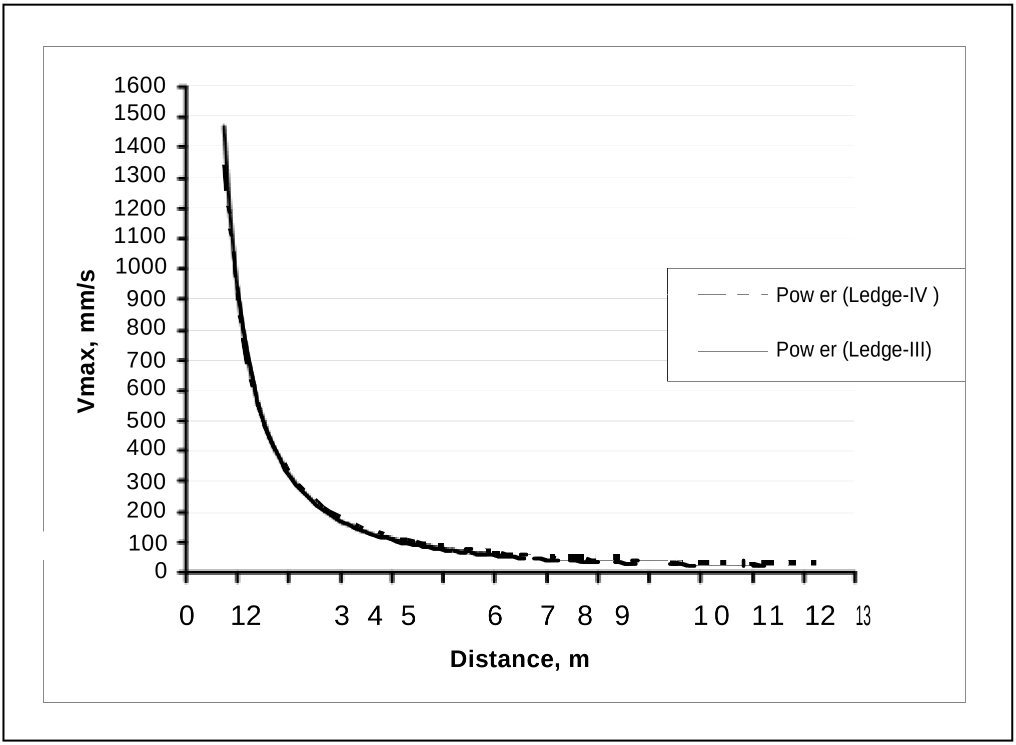


Figure 4. Damage predictions using the Holmberg & Persson model.

Table 3. Extent of predicted rock mass damage into the tunnel wall.

SNo.	Rock type	K	γ	V_{cr} from H-P model	Extent of damage from H-P model	Extent of damage from Borehole camera survey
1	Tunnel-I	940	0.55	590 mm/s	1.35 m	1.51 m
2	Tunnel-II	887	0.61	460 mm/s	1.58 m	1.85 m

6. ROCK MASS DAMAGE

6.1 Rock mass damage at Tunnel-I

The vibration attenuation curve plotted for peak particle velocities versus scaled distance at rock mass of the Tunnel-I, is shown in Figure 5. The Vmax recorded at Tunnel-I were exceeding critical peak particle velocities (Vcr) in the initial 7 blast rounds, which were near-field vibrations. The Vp and Vmax values were measured before and after each blast round. The plot of Vp values versus Vmax is shown in Figure 6. The following observations are made from the plot of Vp versus Vmax:

- There was no considerable reduction in Vp below critical peak particle velocity.
- Substantial reduction in Vp was noticed after 22 initial blast rounds.
- The fall in Vp was continued until the Vmax level reached 130 mm/s, which is about 22% of the Vcr.
- There was no considerable reduction in Vp below the 22% of Vcr irrespective of the number of cycles of blast loading.

The borehole camera survey was conducted to determine the extent of damage due to repeated loading. The borehole camera captured pictures indicating that the damage was observed only for the near-field blast vibrations and the crack extensions drastically come down and stopped after 15 initial blast rounds. The corresponding vibration level and the depth, where the damage stopped completely was 595 mm/s and 1.56m respectively. Again the damage was noticed by the borehole camera at a Vmax of 315 mm/s after the occurrences of 35 vibration events. The damage and crack extension continued up to Vmax levels of 130 mm/s and to a depth of 2.86m. There were no crack extensions noticed by borehole camera surveys below the Vmax level of 130 mm/s for about 38 cycles of loading. The pre-blast and post-blast images captured by the borehole camera at the Tunnel-I at a depth of 2.86m are shown in Figure 7.

6.2 Rock mass damage at Tunnel-II

Similar methodology was followed for assessment of rock mass damage due to repeated loading at Tunnel-II. The vibration attenuation curve plotted for peak particle velocities versus scaled distance at Tunnel-II rock mass is shown in Figure 5. The Vmax recorded at Tunnel-II were exceeding critical peak particle velocities (Vcr) in the initial 8 blast rounds, which were near-field vibrations. The Vp and Vmax values were measured before and after each blast round. The plot of Vp values versus Vmax is shown in Figure 6. The following observations are made from the plot of Vp versus Vmax:

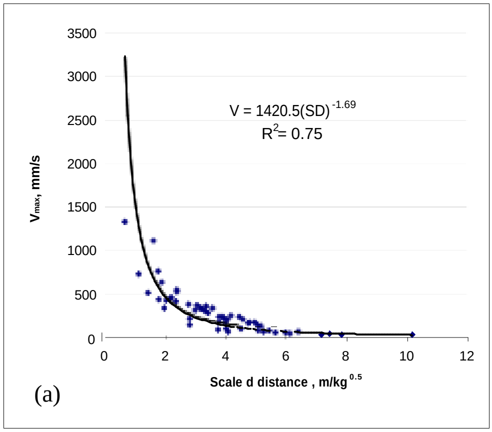
There was no considerable reduction in Vp below critical peak particle velocity

Substantial reduction in Vp was noticed after 21 initial blast rounds.

The fall in Vp was continued until the Vmax level reached 116mm/s, which is about 25% of the Vcr

There was no considerable reduction in Vp below the 25% of Vcr irrespective of the number of cycles of blast loading

The borehole camera survey was conducted to determine the extent of damage due to repeated loading. The borehole camera captured pictures indicating that the damage was observed only for the near-field blast vibrations, and the crack extensions drastically came down and stopped after 20 initial blast rounds. The corresponding vibration level and the depth, where the near-field damage stopped completely was 460mm/s and 1.85m respectively. Again the damage was noticed by the borehole camera at a Vmax level of 278mm/s after the occurrences of 32 vibration events. The damage and crack extension continued up to Vmax levels of 116 mm/s and to a depth of 3.64m. There were no crack extensions noticed by borehole camera surveys below the Vmax level of 116 mm/s for about 20 cycles of loading. The observations of borehole camera survey are almost in line with the results of ultrasonic testing of rock mass.



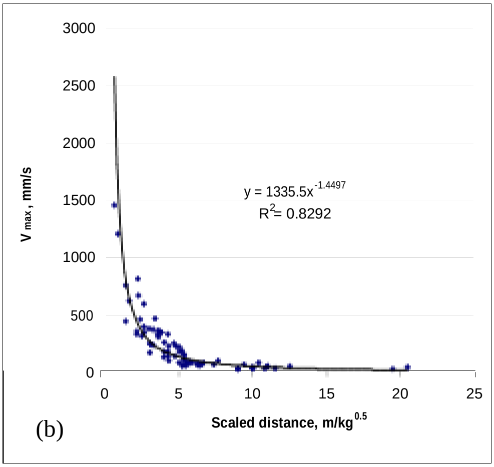
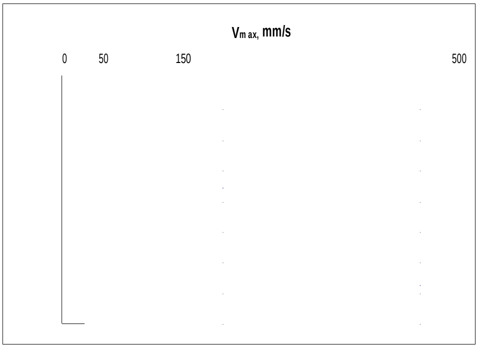
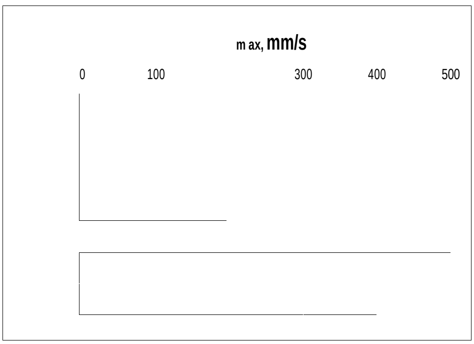


Figure 5. Vibration attenuation curve for (a) Ledge-III and (b) Ledge-IV.



(a)



(b)

Figure 6. Plot of V_p versus V_{max} for (a) Tunnel-I and (b) Tunnel-II.

7. RESULTS AND CONCLUSIONS

The near field blast loading due to main production blasts of Tail Race Channel (TRC) excavations could generate displacements in the rock mass, only when the peak particle velocity exceeded the critical vibration levels (Vcr). After repeated exposures of vibrations from the TRC blast rounds could produce displacements in the rock mass even at vibrations lower than the Vcr. The extra damage due to repeated blast loading at Tunnel-I and II were 1.35m and 1.79m respectively.

After the exposures of 65 blast rounds with the Vmax levels ranging from 25 to 1500 mm/s, considerable damage was observed by borehole cameras even at the Vmax level of 130mm/s, which is approximately 22% of Vcr at Tunnel-I. Similarly, the rock mass damage at Tunnel –II was observed even at the Vmax level of 116.0 mm/s i.e. at approximately 25% of Vcr after 60 exposures of blast loading. The threshold vibration limits with number of cycles of repeated loading for two different rock mass are given in Table 4.

The observations also indicate that the repeated dynamic loading resulted in the damage at the vibration levels even at about 22% of Vcr. This observation was almost similar to the finding of Adamson and Scherpenisse (1998), which says that threshold vibration levels fall down to 25% of Vcr in repeated loading conditions.

(i) (ii)

Figure 7. Images of borehole camera (i) before and (ii) after repeated dynamic loading at Tunnel-I.

Table 4. Vcr with number of cycles of repeated loading for Tunnel-I and II.

SNo.	Rock type	No of cycles of blast loading	Threshold vibration limit, mm/s
Tunnel-I	Pink granite (moderately joints)	65	150
Tunnel-II	Pink granite (heavily jointed)	60	106

Almost similar results were achieved by Rajaram (1978). The present study emphasizes that the reduction of strength of jointed rock mass is more than 90% when there is repeated loading. The repeated dynamic loading also results in reducing the threshold peak particle velocity to 22% and 25% of critical peak particle velocity for initiation of damage at Tunnel-I and Tunnel –II, respectively. The study indicates that the effect of repeated vibrations on structural foundations must be considered while multiple excavations are being conducted by blasting method.

8. ACKNOWLEDGMENTS

The authors express their sincere thanks to the Director, CMRI for permitting to publish this paper. The authors express their sense of gratitude to the executives of JHPP, APGENCO for their cooperation during the field studies.

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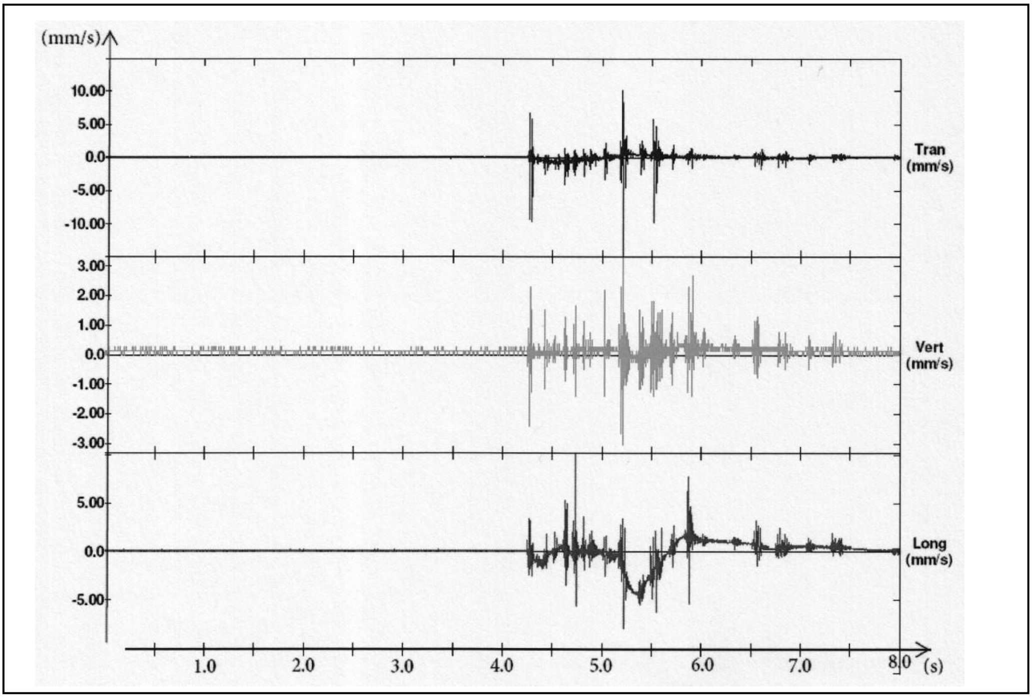

Figure 5. Trajectories of the ground vibration velocities.

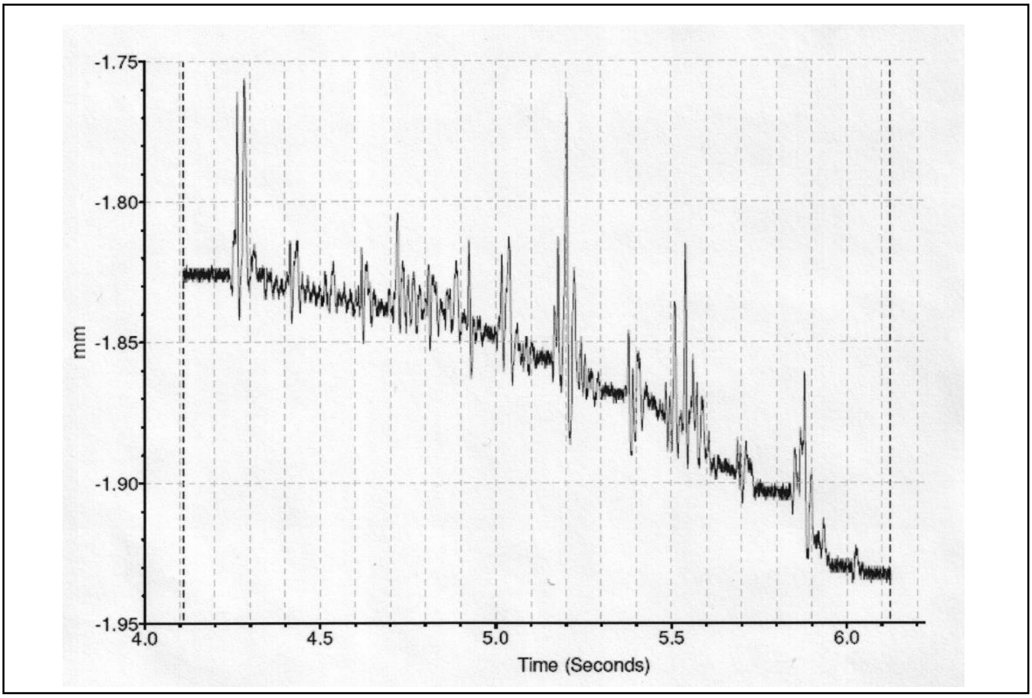

Figure 6. Trajectory of the relative displacement of the boulder.

Table 4. Values of permitted ground vibration velocities with regards to frequencies for historic monuments according to DIN 4150.

	1. PERMITTED GROUND VIBRATION VELOCITY Vpermitted (mm/s)			
Frequency (Hz)	1-10	30	50	100
Historic monuments and similar	3	5	7	9

b) Zone 18 mm/s

Zone 18 mm/s was established based on the registered relative displacement of the boulder at the measuring point MO 2 during blasting of the field MP-1. This is also the critical zone since the displacement of boulders is expected to occur. All boulders in the area must be treated in order to prevent their displacement. Some of the protection measures include:

Anchoring of boulders

Securing of boulders with protective net

Mechanical removal of boulders

c) Zone 10 mm/s

A zone 10 mm/s is the zone, which requires obligatory measurement of ground vibration velocities caused by blasting. Since the first measurement could not include all the boulders, it is impossible to establish with certainty that there will be no displacement of boulders in this zone. According to the measurement results, displacement of boulders is initiated at vibration velocity of 17.82 mm/s. Since the boulders differ in shape, mass, attachment to the base rock and position on the slope, a boulder of different characteristics can be displaced at smaller values of vibration velocity. Therefore the zone 10 mm/s requires constant measurement of vibration velocities and possible relative displacement of boulders.

Calculated distances corresponding to the above mentioned zones are entered on the tunnel cross sections. The points in which the radii of the zones intersect the surface of the terrain are marked on the layout plan and connected. Figure 7. shows the boundaries of the safety zones determined in such a manner.

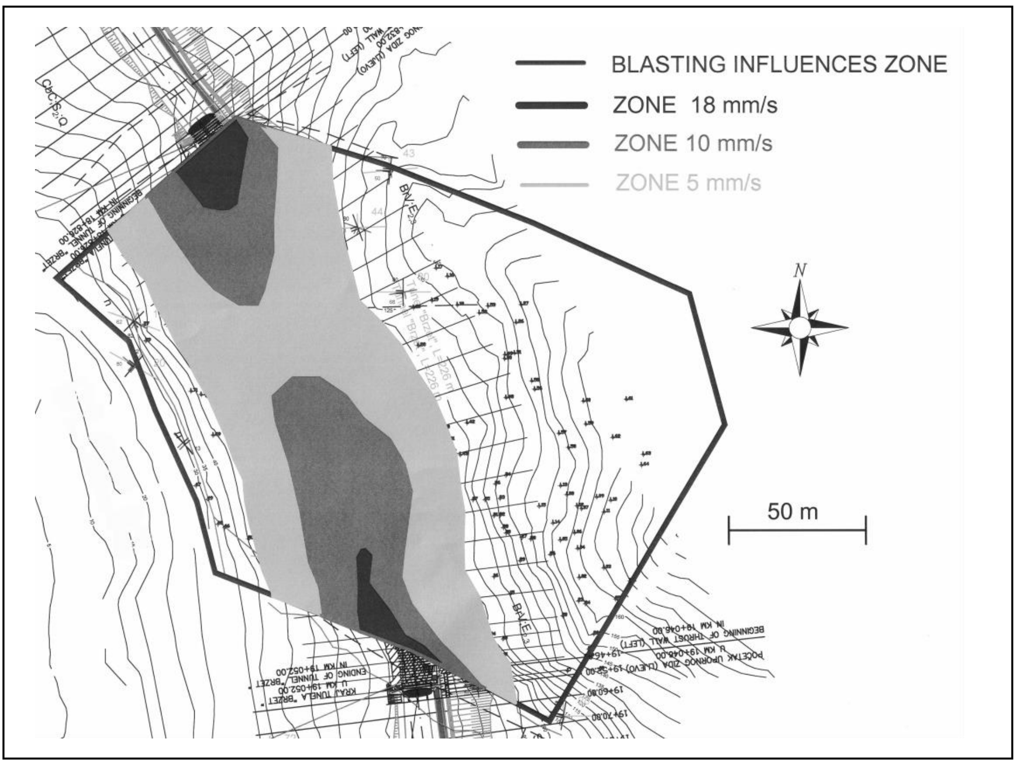

Figure 7. Safety zones.

4. CONCLUSION

The Brzet tunnel represents a classic example of a tunnel excavation by blasting in special conditions. Conditionally unstable boulders in the zone of tunnel excavation pose a potential danger to the Adriatic Highway and the surrounding residential area. Guidelines for further tunnel excavation by blasting are as follows:

- Treatment of all unstable boulders in the safety zone with the limit of 18 mm/s
- Decrease of the explosive charge mass per initiation interval to 5 kg (charge mass that is established according to M.A.Sadovski based computation referred to the trial blast fields measurements results including safety coefficient)
- Measurement of ground vibration velocities and relative displacement of characteristic boulders

Boulders must be treated in the safety zone with the limit of 18 mm/s. Boulders may also be treated under this limit depending on the results of subsequent measurements of seismic effect of the blasting.

Decreasing of explosive quantity per delay will decrease the particle vibration velocity and therefore eliminate the possibility of displacement and movement of boulders during blasting. In order to achieve this, it is required to decrease the length of boring, maximum instantaneous charge and use all degrees of millisecond delay available in the non-electric initiation system. It is necessary to continually measure vibration velocities and to monitor possible displacement of unstable boulders for various reasons. By changing the blasting method, that is, boring and blasting patterns, there is also a change in the effect on conditionally unstable boulders. By continually monitoring the effects of blasting during the advance of excavation through rock mass, with the present change of geological characteristics of the surroundings, we have the possibility for timely correction of boring and blasting patterns, which will enable us to prevent initial displacement of the boulder from its existing unstable position.

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Underwater blasting of a rock cofferdam bench (vibration and flyrock limitations, terminal building only 15 metres away) at Risavika harbour, Stavanger, Norway

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ABSTRACT: Risavika harbour is undergoing a period of considerable expansion, strengthening its position as the most important port on the coast of Norway. Risavika is also in the process of becoming a hub in the European logistics network. Risavika, which also includes Sola harbour and the Norsea base Tananger, is the Norwegian petroleum industry's main base today. It is also a point of entry for goods being imported into Norway, and for fresh fish that has to be transported quickly to markets in Europe.

There are plans to construct a modern container port in Risavika harbour, together with a new international terminal and quays specially designed for bulk transport. Once the work has been completed the port will have 1800 metres of quays, a harbour area measuring 75 hectares, and large areas which may be used for future expansion.

The quay area, measuring 4.5 hectares, has been excavated as an open basin down to a depth of 15 metres. The local contractor T. Stangeland AS has been responsible for the removal of the rock. Approximately 2 million cubic metres of rock had to be blasted away in order to create the harbour basin which has a volume of 700,000 cubic metres.

In April 2008 the blasting operations in the harbour basin were finished, and a 200 metre long, 15-20 metre wide and 15 metre high rock cofferdam bench was left for blasting and dredging in order to open the harbour to large ships. The contractor T. Stangeland AS engaged Multiconsult to measure general blast-induced vibrations in the area, especially at the nearby the International Terminal building. The technical support department of Orica Mining Services was involved in the planning and implementation of the blasting operation.

This paper describes the blasting techniques employed in order to achieve the best possible fragmentation of the rock cofferdam bench, and how fly rock and destructive vibrations on the terminal building were avoided. The new basin was filled with water up to sea level before blasting. Using 65 tons of emulsion explosives (the Titan SSE surface system) and a redundant Nonel Unidet initiation system, the blast went off successfully without any damage to the surroundings. However, the effect of the wave surge created was larger than expected.

1. INTRODUCTION

1.1 Project description

Risavika harbour is undergoing a period of considerable expansion, strengthening its position as the most important port on the coast of Norway, by means of expanding its harbour water area by about 40 hectares. The port will become of central significance to the European logistic network, through being the main base of the Norwegian petroleum industry. It is also a point of entry for goods being imported to Norway, and for fresh fish that has to be transported quickly to markets in Europe. The Norsea base and the port of Sola provide all necessary harbour services. Stavanger is not only the oil capital of Europe, but is also the home of a strong maritime tradition and a rapidly growing aquaculture industry.

There are plans to construct a modern container port, a new international terminal with 1800 metres of quays and a 75-hectare harbour district in Risavika. In addition there will be large areas, which may be used for future expansion. Risavika will also become an international centre for research and development, and for bringing to market environmentally friendly and future-proof energy, technology and harbour services.

The quay area, known as Vågen, is 4.5 hectares in area and has been excavated as an open basin, with a depth of 15 metres and a volume of 700,000 cubic metres (Figure 1). When all the finishing work at Vågen was complete, seawater was admitted to the new basin from Risavika harbour. Demolition of the cofferdam of the new basin entailed special requirements and the planning of this work had to be conducted with particular care. The rock will be subsequently removed with waterborne excavating equipment and the facility will be ready to receive ships before the end of 2008.

The construction contractor decided to build the Risavika International Terminal well in advance of the cofferdam being ready for demolition. The building had reached its full height of five floors and the glass façade was being mounted during the period scheduled for blasting. The shortest distance from the International Terminal to the western end of the cofferdam was only 15 metres. This circumstance was of great significance during the planning of the cofferdam demolition, of course.

Stangeland Maskin AS are conducting all excavation of rock and all blasting in the Risavika project, including the demolition of the cofferdam. The total amount of rock involved was about four million cubic metres of solid rock and one million cubic metres of loose material. These volumes were to be moved within a period of 19 months. The total budget for the expansion of the harbour is NOK 295 million and the work was due to begin in May 2006 and be delivered in September 2008.

Explosives were supplied as required from a local slurry depot set up by Orica Mining Services. The depot supplies SSE bulk emulsion directly to the site. Multiconsult was involved in the measurement blast induced vibrations and Orica

Figure 1. Risavika Harbour and Vågen, viewed from the cofferdam near the International Terminal.

Mining Services for the planning and execution of drilling and initiation plans.

2. COFFERDAM DEMOLITION

2.1 General

The majority of experience of regarding blasting of cofferdams comes from the sector of hydroelectric schemes where cofferdams at new intakes are regularly blasted away, or from the demolition of cofferdams at new dock installations. Other types of underwater blasting include port extensions, ferry quay deepening, excavations for bridge piers and general demolition. Within the offshore industry, the explosive demolition of the cofferdams of drill platforms in dock is the nearest parallel to the cofferdam demolition in Risavika harbour.

It is relevant that the amount of rock, which has to be blasted, is usually completely or partly underwater. When blasting takes place under water, greater consideration and experience is required than for everyday blasting as the water will cover large parts of the explosion. Each case of underwater explosive demolition brings about some of the biggest challenges conceivable.

Examples are:

The water itself

- is often set in motion
- is an additional burden
- the explosive is seriously put to test
- visible oversight of the blasting area is often prevented

Resistance to pressure and time

- sleep time for charged explosives may be extended

Misfires must be avoided

- an unsuccessful shot can have extensive consequences

Must pull grade on first shot

- sufficient explosives energy distribution
- frequent drilling problems, uneven bench front

Environmental circumstances can be more serious

- blasting in the vicinity of fish farms
- pressure propagation and flooding wave formation

Some of the special consideration for each blast design include:

- Geology
- Explosives and initiation systems
- Water and mud as overburden
- Specific charge
- Propagation and vibration

3. THE COFFERDAM IN RISAVIKA HARBOUR

3.1 Planning

Once Vågen, the basin for Risavika harbour, was almost completely excavated, there remained a cofferdam against Tananger Fjord, with a volume of 100,000 cubic metres. This cofferdam was about 200 metres long. Its width varied from typically 15 metres at the top to 20-30 metres at the bottom. Its height was 15-17 metres at depth levels 0 to +2, and down to –15. The principal rock type was granitic to granodioritic gneiss, mainly consisting of quartz and feldspar. In the northeastern part of the peninsula the rock was significantly stratified.

In addition to the granitic gneiss strata, amphibiolite, quartzite, mica schist, mica gneiss and marble could be observed. The strata lay mainly northwest/southeast and dropped down 30° to the southwest. The separate layers were usually about 1-2 metres thick. The rock was relatively eroded and broken up near the surface. See Figure 2.

Initially it was thought that the cofferdam should be blasted in one large shot. When planning began, it was decided to carry out a calibration blast inside the Vågen in order to reduce the size of the cofferdam bench. See Figure 3.

This blast would allow data to be gathered on the vibrations that the International Terminal on the one side and the existing underground oil stores on the other, would experience. The blast covered 31,000 cubic metres, was 130 metres long and extended from profile 0 to profile 130. Measuring instruments were set up in four places with designated distances from the shot. The calibration blast was set off on 13 February and

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Special blasting conditions and measurement program during the excavation of the Brzet tunnel

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ABSTRACT: The paper describes the technology of blasting works during the excavation of the Brzet tunnel, with a special review of a secure seismic blasting regime, taking into account the delicate geological situation of the surface over the tunnel. Overlaying rock consists of limestone boulder. Single unstable rock boulder can be found on the surface above the entire tunnel route. There is a fault at the tunnel portal, which presents additional difficulty in the execution of works. Taking into account the vicinity of the highway and residential area beneath the tunnel, it was necessary to establish a secure blasting regime. Maximum instantaneous charge in the tunnel-blasting field must not result in such ground vibrations, which would cause falling of the boulders. Therefore, measuring of ground vibration velocities in the rock mass were conducted, as well as measurement of relative displacement of boulders by using LVDT displacement sensors. In accordance with the safety requirements, the permitted ground vibration velocity level is set at 15 mm/s. Based on the results of measurement of ground vibration velocities and the maximum instantaneous charge, individual ground vibration velocity zones were determined with regards to the thickness of rock between the side of the tunnel and the slope of the hill. At each blasting, the permitted ground vibration velocities will be monitored. In case the measured velocities reach the limit permitted value, blasting patterns will be corrected and advance per single blasting will be decreased. Additional safety measures include: controlled removal of fully detached boulders and execution of protective embankments under the tunnel route.

1 INTRODUCTION

Designed length of the Brzet tunnel, located on the state road D-8, section Dugi Rat-Omiš, is 226 meters. Besides tunnel Brzet, this section encompasses the tunnel Omiš, with the length of 1.471 km and three viaducts of total length 489.69 m. The contractor is a company Strabag d.o.o. Design solution anticipates blasting of the tunnel in two stages for all tunnel categories of excavation. In the first stage the blasting of the calotte is planned and in the second stage the blasting of the bench is planned.

Geological characteristics of the terrain represent a special problem during the blasting of the tunnel. Numerous boulders are situated along the designed route of the tunnel. Faculty of Mining, Geology and Petroleum Engineering, Zagreb conducted the measurements and completed the Study on blasting effects on neighbouring residential buildings, taking into account residential buildings and possible nearest to the blasting field with access possibility for sensors setup. Displacement sensors measured relative movement of boulders caused by blasting vibrations. Ten measuring points were established (MO 1; MO 10). Figure 2. shows the installation method of displacement sensors and geophones, and Figure 3. shows the arrangement of the measuring points.

Figure 2. Installation method of displacement sensors and geophones.

There is a fault by the south portal area. During a trial blasting, measuring points MO 1- MO 4 were located at the hanging wall of the fault while the blasting field was in the footwall. Due to the variable geological condition along the tunnel, the trial blasting will not be representative for the entire tunnel route. The effect of following blastings shall be adequately estimated during the future blastings in the tunnel.

Measurement and evaluation of the blasting effects on unstable boulders has to be repeated when the blasting is performed in the hanging wall of the fault. Figure 4. shows the south tunnel portal and the fault.

3.2 Measurement results

Table 2. presents the results of measurement conducted during blasting of the first blasting field (MP-1) and Table 3. presents the results of measurement conducted during blasting of the second blasting field (MP-2).

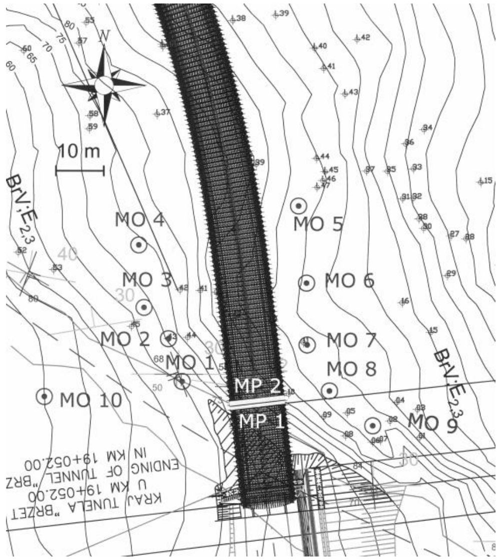

Figure 3. Arrangement of measuring points.

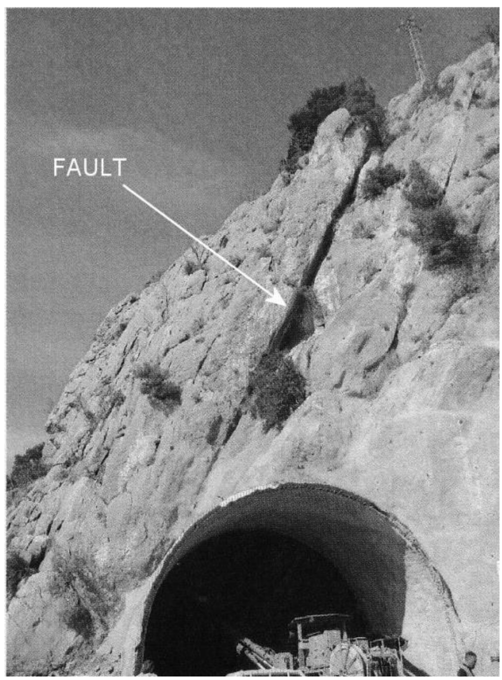

Figure 4. South tunnel portal and the fault.

A relative displacement of the unstable boulder number 43 was registered during blasting of the produced an excellent result. The measuring point at the terminal showed 22 mm/s at a distance of about 115 metres from the shot, while measurements showed 252 mm/s at 18 metres and 101 mm/s at about 44 metres; measurements to the south showed 97 mm/s at a distance of 42 metres. The maximum charges per interval were respectively 300 and 600 kg, so that the unit charge was determined to be respectively approximately 200 kg and 400 kg. Recordings from the calibration shot thus provided a good basis for the evaluation of the blast technical data for the underwater blasting of the cofferdam. Technical parameters as shown in Figure 4.

Table 1. Blast induced vibrations in the International Terminal, theoretical and measured.

Measuring Point, MP	MP 5	MP 6	MP 7
Distance from cofferdam to MP in the terminal	40 m	25 m	16 m
Theoretical value, PPV, *)			168 mm/s
Measured value PPV,	48 mm/s	98 mm/s	-
Rock constant	156	200	220*)
Max. cooperating Charge	150 kg	150 kg	150 kg

*) Stipulated from the calibration shot

3.2 Distance to the International Terminal - Risks of damage

The shortest distance from the cofferdam bench to the International Terminal was only 15 metres. With an assumed unit charge of 150 kg per interval this would give vibrations (PPV), after adjustment calculations, of about 168 mm/s (vertical PPV). In accordance with NS 8141, it can be seen that an office building built on a gravel bed over rock requires a limiting value of 50 mm/s. This means that the risk of damage to the International Terminal was highly likely.

3.3 Plan to limit blast induced vibrations and back break damage

In order to prevent back or side break damage from the blast, and also to reduce the overall vibrations, it was planned to drill a seam of 3.5-inch borehole diameter with a c/c distance of 250 mm. To cut the risks which could arise below the bottom charge level, the seam was drilled down to 3 metres under the sub-drilling, i.e. to depth level –20. The seam was extended in this manner to that part of the cofferdam rock lay closer than 40 metres from the terminal, which was cut off by the seam. This part of the seam was drilled from level –15 inside Vågen.

Figure 2. Part of the cofferdam in Risavika (Vågen) Harbour, 200 metres long, 15 metres high. Volume 100,000 cubic metres.

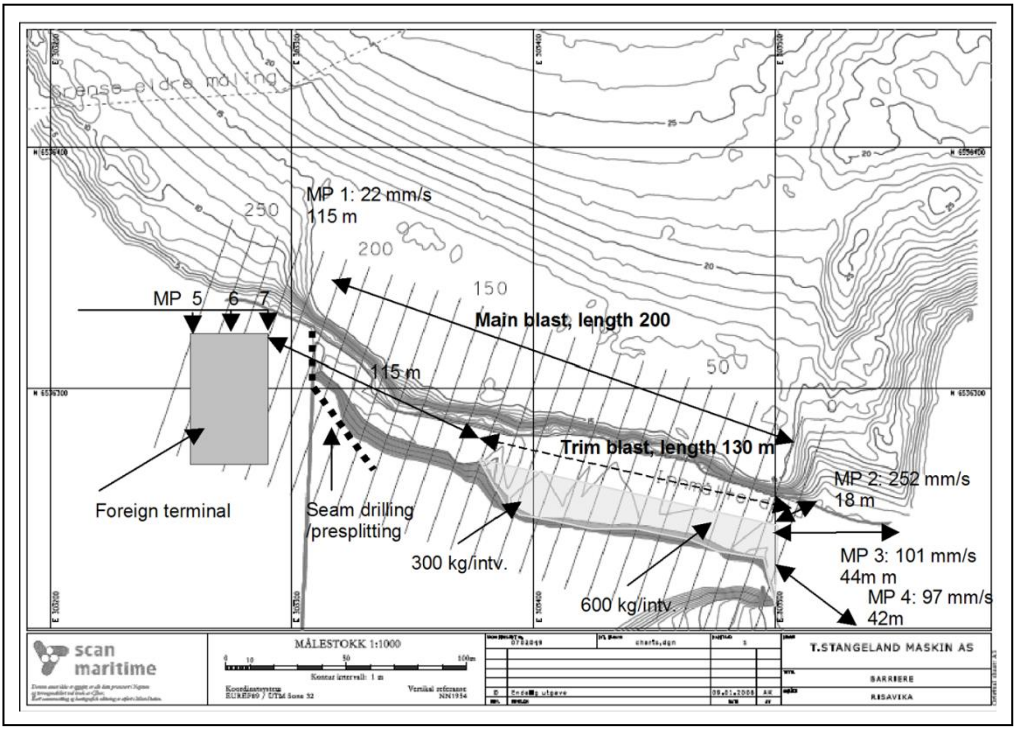

Figure 3. Vibration measurements from the calibration shot inside Vågen, 13 February 2008.

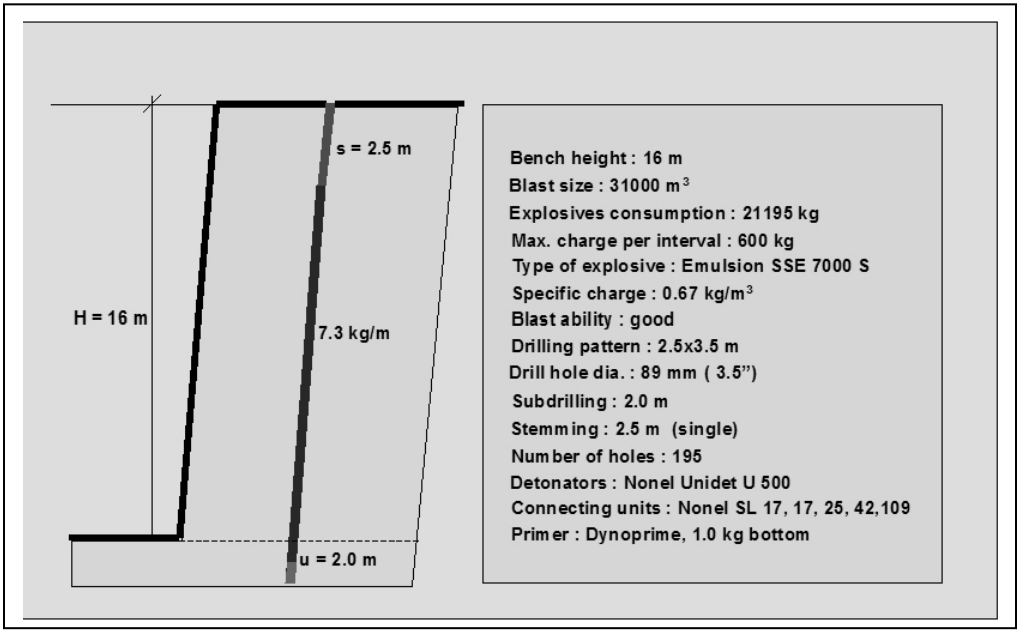

Figure 4. Technical parameters for the calibration shot covering about 31,000 cubic metres.

blasting field MP-1 (measuring point MO 2). Figure 5. shows the trajectories of ground vibration velocities, while the trajectory of the relative displacement of the boulder is shown on Figure 6.

Table 2. Measurement results for the blasting field MP-1.

Blasting of MP-1					
	V_t (mm/s)	V_v (mm/s)	V_l (mm/s)	V_r (mm/s)	LVDT (mm)
MO1	31.5	18.8	25.9	33.1	-
MO2	14.37	3.3	10	17.82	0.105
MO3	6.73	1.4	1.52	6.79	-
MO4	1.65	0.762	0.127	1.69	-
MO5	3.17	2.92	2.67	4.5	-
MO6	3.05	3.56	0.381	4.28	-
MO7	< 5.0	< 5.0	< 5.0	<5.0	0
MO8	6.1	4.45	5.46	6.76	-
MO9	7.112	5.08	11.684	13.208	-
MO10	1.778	1.143	3.048	3.175	-

Table 3. Measurement results for the blasting field MP-2.

Blasting of MP-2					
	V_t (mm/s)	V_v (mm/s)	V_l (mm/s)	V_r (mm/s)	LVDT (mm)
MO1	30.5	62	49.8	63.7	-
MO3	9.27	1.14	1.78	9.27	-
MO4	2.41	1.02	0.127	2.43	-
MO5	4.57	4.06	4.32	5.86	-
MO6	5.46	6.1	2.92	7.19	-
MO7	6.98	1.52	2.29	7.50	0
MO8	8.51	7.49	9.14	9.92	-
MO9	9.144	6.64	13.208	17.018	-

3.3 *Analysis of measurement results*

Movement of the boulder 43, represented with a displacement from domain rock, caused by measured oscillation velocities was origin for determination of safety velocities zones. Attenuation of particle vibration velocities for each direction was calculated based on the measurement results, distance of the measuring instruments from the blasting field and the maximum instantaneous charge. Calculation was made by using the computer program Mine (Faculty of Mining, Geology and Petroleum Engineering), which is based on the formula by M.A.Sadovski:

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

v – particle vibration velocity (cm/s),
k – coefficient of blasting technique,
Q – quantity of explosive per delay (kg),
R – distance from measuring point to blasting field (m),
n – coefficient of attenuation of particle vibration velocities

Values Q and R are known, and the measuring data provide the resulting ground vibration velocities at measuring points. Coefficients k and n are calculated by a system of equations where vibration velocities are measured at two measuring points during one blasting. Measuring points shall be at different distances and in the same path with regards to the blasting field.

Three safety zones were established in accordance with the results of calculation, measurement results from displacement sensors and requirements from the standard DIN 4150:

a) Zone 5 mm/s
b) Zone 18 mm/s
c) Zone 10 mm/s

a) Zone 5 mm/s

Zone 5 mm/s was established according to the standard DIN 4150 since the Republic of Croatia has no national standard for permitted ground vibration velocities during performance of blasting in the vicinity of structures. The standard determines three types of structures and permitted ground vibration velocities for each type:

Industrial structures,

Building structures,

Sensitive structures (historic monuments and similar)

Table 4. presents the values of permitted ground vibration velocities for historical monuments (including dry stone wall located in the vicinity of the Brzet tunnel) according to standard 4150-3-1999-02.

The frequencies of vibration velocities measured on all measuring points were higher than 30 Hz, so according to the standard the established criterion was $v_{permitted}$ = 5 mm/s. This is a safety zone, where no displacement of boulders is expected.

displacement of boulders from their original position. The study determined safety blasting zones, permitted advance and maximum quantity of explosive per delay.

2. GEOLOGICAL CHARACTERISTICS OF THE TERRAIN

Croatian Geological Survey (Institute) prepared the Report on geological survey and marking of unstable boulders in the zone of the tunnel Brzet. Engineering geological survey was performed and unstable boulders were marked on the SW slope of the hill. The surveyed area amounted to 3.13 ha, with the average slope inclination of 43 degrees.

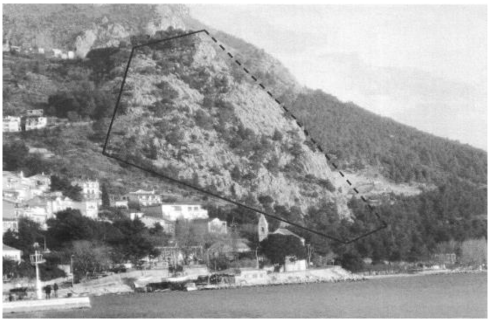

Figure 1. Surveyed zone around the Brzet tunnel.

The area consists of limestone breccia, i.e. of bio-accumulated calcarenite. Breccia is small to coarse-grained, firm, gray and gray-brown in colour. There is a poor to medium rock stratification, so the overall rock mass appearance is boulder-like. Rock mass is considerably fractured along fissures of various directions. Relief is steep with frequent vertical cuts, especially in the border areas. Most of the tectonic fissures are gaping and karstic, and the filling material consists mainly of clay and rock debris, with roots from the surrounding plants.

A total of 149 boulders were surveyed. Boulders were divided into 5 groups or types, according to their location, origin and instability. All boulders were marked, located using a GPS device and entered into the layout plan.

3. MEASURING OF GROUND VIBRATION VELOCITIES

3.1 Blasting fields and measuring instruments

During the two initial blastings with incomplete excavation profile measurements were conducted at the south tunnel portal. Explosive charge consisted of explosives Lambrex and Austrogel. Table 1 presents quantity of explosive per blast hole type for the first and the second blasting field.

Maximum explosive charge mass per delay for MP-1 was 12.20 kg, and 18.88 kg for MP-2.

Ground vibration velocities that occurred during blasting were measured by portable seismographs type Instantel Blast Mate and White Digital. Measurements were conducted by three-component geophones, which measure the peak particle velocity and by LVDT displacement sensors. The task was to determine peak particle velocity, which causes displacement of unstable boulders. For that purpose, two boulders were chosen and equipped with geophone and displacement sensors. Each boulder was the representative one for areas, left and right, respective to the tunnel axe. Those boulders were

Table 1. Explosive charge quantity for blasting fields MP-1 and MP-2.

	First blasting field MP-1			Second blasting field MP-2		
	Charge (kg)	Total (kg)	Number of blast holes (pcs)	Charge (kg)	Total (kg)	Number of blast holes (pcs)
Cut	1.18	31.86	27	1.53	45.81	30
Floor	1.18	12.98	11	1.53	21.38	14
Auxiliary	0.76	12.98	17	1.18	28.32	24
Contour	0.92	13.75	15	0.85	9.32	11

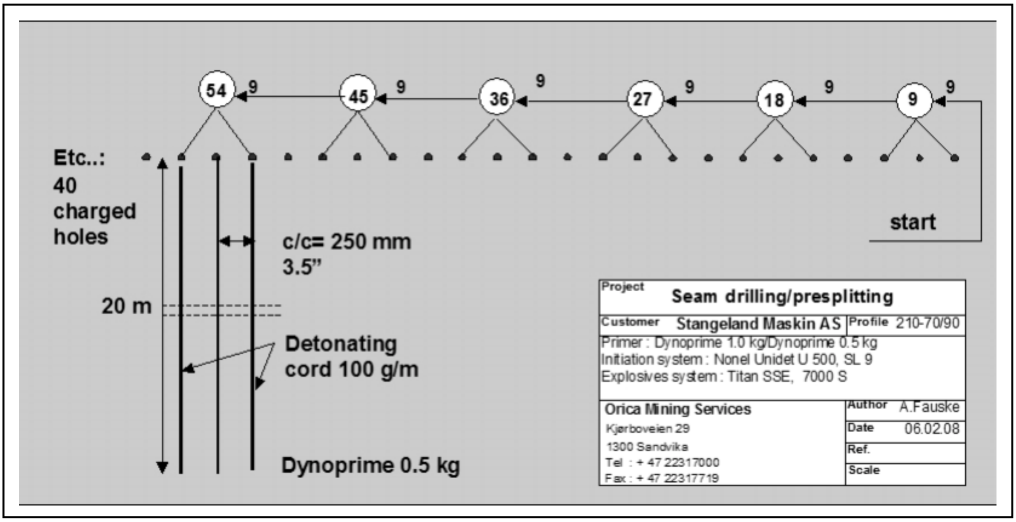

Figure 5. Drilling and charging plan for presplitting of seam holes 15 metres from the International Terminal.

Figure 6. Detonation of seam holes 15 metres from the International Terminal. The sand embankment with the shot mats functioned according to plan.

The seam holes were shot separately before the main cofferdam blast. Alternate seam holes were charged with Nonel Unidet U 500 and Dynoprime 0.5 kg + detonation cord 100g/m, totalling 2.5 kg charge per hole.

The presplit seam was detonated by 2 holes per detonation time interval with a progressive increase in delay time of 9 milliseconds by means of E-clip 9MS. Theoretically this gave a charge of 3.3 kg. The number of drilled holes in the full length of 20 metres of the presplit line was 80, and so the number of charged holes was 40.

Seam holes usually have a significant vibration-reducing effect. Additionally with the presplit seam holes, an extended slot is formed in between. This gives even further reduction effects. This is conditional on the seam not being situated underwater. In this case, this could not be avoided, and it is uncertain what effect the presplitted seam actually had. However, it is considered that it had an advantageous effect.

Generally, normal or instantaneous presplitting gives a significant air shock, increased vibrations and the danger of flyrock. The method chosen above can be regarded as a shear blasting which gives significant damping of both vibrations and flyrock; this is on account of the limited amount of energy which is released per interval; it was however just big enough to create problems with regard to the glass façade of the International Terminal.

3.4 Plan to limit flyrock

A quantity of stones and gravel can be thrown up on both sides of the seam holes. The seam holes were covered with heavy shot mats with suitable spacing so that the separation between the seam holes and the shot mats was a minimum of 1 metre. The shot mats nearest the terminal were anchored and held by a 3 metre sand embankment. The other side of the mats lay freely over the seam-drilled line.

The reasoning for this was for the shot mats to function as a pressure valve and direct any flyrock away from the terminal. The sand embankment for its part was to assist in directing the shock wave away from the building. On 9 April 2008, everything was ready for detonating the presplit seam. The detonation went according to plan without any form of damage to the terminal. Vibrations were measured at a distance of 16 metres and registered 12mm/s.

3.5 Drilling challenges

Profiles of the cofferdam every 10 metres were drawn up. Profile 170 for example, shows the most challenging part for drilling of the cofferdam out towards the sea. (Figures 3 and 7)

The front of the cofferdam bench is mostly loose slack in this area. This required drilling at a slant with relatively large inaccuracies, up to 30°. As the cofferdam had a greater width at the bottom than at the top, additional drilling was required so as not to have too great a load at the bottom. Furthermore the rock was greatly disintegrated, and contained a lot of loose material in the profile region 160 – 190. Drilling on the edge of the cofferdam could also be highly risky so remote-controlled drilling was required in this region.

3.6 Blasting plan for the Risavika rock cofferdam bench

For technical drilling reasons, it is preferred to use a 4-inch borehole diameter, but 3.5-inch is also used. The drilling pattern is generally set at 2x3 metres for 4-inch, 1.6x2.7 for 3.5-inch. This gives a specific charge of 1.55-kg/cubic metre for emulsion explosive.

In the case of underwater blasting in general and cofferdam demolition in particular, the starting point is to use a specific explosive charge which is twice that which would be used for corresponding blasting on land. This however requires a rock type with good blasting characteristics. A rock type like Fyllitt which is dominant in the Stavanger area, will require a significantly larger explosive charge. As a result of the calibration shot on the cofferdam where a specific charge of 0.67-kg/cubic metres was used, we got a good indication of the requisite specific charge. Increasing the explosive charge to 1.55 kg/cubic metre which is a little more than twice as much, was however vital to compensate for possible failures, or larger drilling deviation in this type of rock, so that we could actually get satisfactory fragmentation. The consequences of an unsatisfactory blast result underwater are great and have to be avoided as far as possible.

Furthermore, with underwater blasting, it is usual to employ an underdrilling of a similar size to the burden. This is because the subsequent underwater handling of boulders is distinctly more demanding than corresponding work on land. It is essential that the equipment available can cope with the removal or dredging of the cofferdam without too many difficulties. Technical parameters for the cofferdam are shown in Figure 8.

As regards future operations in the project, it was decided to demolish the cofferdam in two shots, where the first dealt with about 70% of the cofferdam. The final drilling and initiation plan for this part is shown in Figure 9. A total of 450 boreholes were drilled, divided between 4-inch and 3.5-inch bore holes in three sections with a group of 3.5-inch bore holes in the middle, as

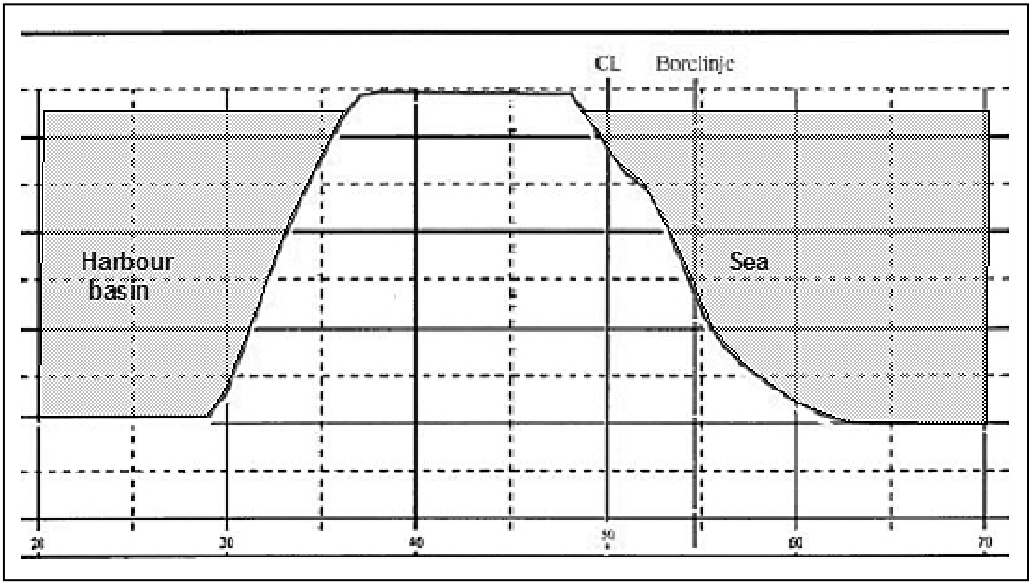

Figure 7. Profile 170 of the cofferdam in Risavika. Slanted and complex drilling in the bench facing the sea is necessary. Inaccuracies up 30 °. A big challenge for the drill operator.

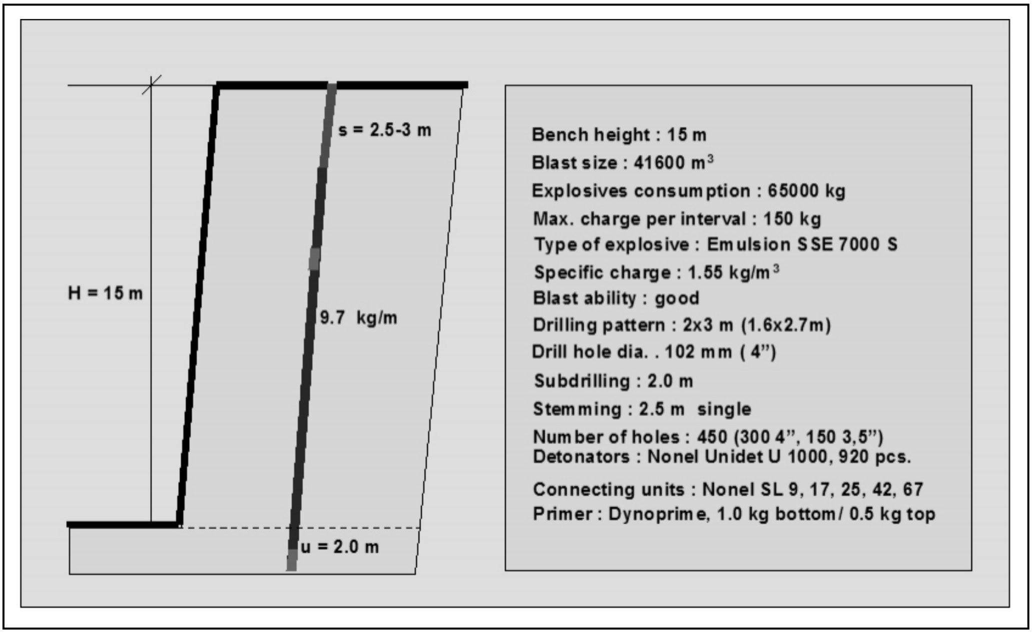

Figure 8. Blast technical parameters for the cofferdam of 41,600 cubic metres. Total explosive - 65 tonnes.

shown in the plan. Totalling 42,000 metres of drilling. The first cofferdam blast covered about 41,600 cubic metres. The cofferdam was cut off on the right-hand side by a presplit line drilled from profile 70 to profile 90. The shot was about 150 metres broad at the front and 120 metres towards the basin. The presplit was charged with 300 kg Dynopre, 22 mm. Then all the holes were charged with 2 primers, Dynoprime 1.0 as bottom primer and Dynoprime 0.5 as top primer, totalling

750 kg primers altogether.

Two Nonel Unidet U 1000 detonators were used per hole, totalling 920 detonators. These detonators were specially produced for the lowest possible spread of delay time in order to avoid too high a likelihood of overlap. Nonel U 1000 have 1000 millisecond delays, which in this context means that all the holes were activated before the first hole on the explosive charge was initiated.

Following an evaluation of technical explosive conditions, and taking into consideration the extent and form of the cofferdam bench, it was decided to initiate the shot in the right-hand side as shown in Figure 9. The detonator system was arranged in the same direction as the charge being gradually directed away from the International Terminal. Furthermore the system was designed to give hole-by-hole initiation, the closer the run came to the terminal. This was to keep the blast-induced vibrations down to the lowest possible levels. The time intervals in the initiation phase were 25 milliseconds for the first 2 rows and then 42 milliseconds for the next 2-5 rows. Closer to the centre, there was one row of 67 milliseconds, and then it became 42 milliseconds again. The explosion was completed partly with 67 and partly with 42 milliseconds intervals.

The bottom and top detonators were to be connected with identical leads to the detonators in order to achieve the greatest possible initiation simultaneity. However this connection method proved rather impractical and also time consuming. But we probably ensured that the proportion of top blowout was somewhat reduced and thereby reduced flyrock.

3.6.1 Redundant initiation system

In a demolition of the size of the Risavika harbour cofferdam, the consequences can be considerable if the detonator system does not function, as it should. The run of initiations in the Nonel system is unidirectional which means that every single connection unit must function optimally for the whole charge to be detonated. The manufacturer guarantees a functional certainty of 99.99%.

However the greatest likelihood of fault arises in usage, usually in connector faults or damage to the product after quality checks.

There are several possibilities for eliminating the risks of the detonator system failing. The simplest solution in order to achieve a redundant solution is parallel replication of the detonator system. This requires more detonators, but is more secure in actual use. We have examples over 15 years usage where parallel replication of the Nonel system in quarrying, or open pit mining has not led to a single misfire.

3.7 Titan SSE bulk emulsion system

The choice of explosive for the Risavika cofferdam blasting was simple. During the whole blasting out of the harbour basin, the principal explosive was the SSE system and the Titan 7000

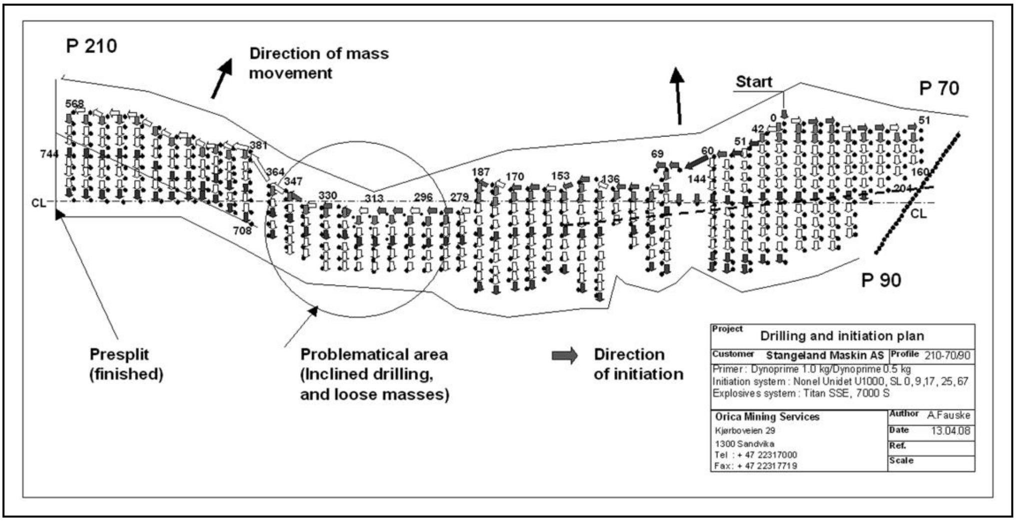

Figure 9. Drilling and initiation plan for the Risavika cofferdam.

S product supplied by a locally established slurry depot. The depot stored the raw materials, which in the main consist of only an emulsion matrix and a gas sensitisation material. A depot SSE explosive truck was loaded with these raw materials as required.

The emulsion matrix has a density of 1.4 kg/litre and is classified as Class 1 oxidising material according to the UN classification system. During the charging process, the sensitising material is added which creates small gas bubbles in the matrix. The density is reduced and the volume of the matrix increases. In a borehole of 17 metres the average final density will be about 1.1 kg/litre, and the explosive Titan 7000 S is ready for initiation. In extreme conditions such as underwater blasting, it is vital to have a good primer as described above.

The internal chemical structure of the Titan 7000 S explosive makes it most efficient as an explosive and gives it very good water resistance (See Table 2). Another of its characteristics is that it develops fewer poisonous gases than other conventional explosives.

At Risavika an SSE explosive truck was used, which has a load capacity of about 12 tonnes of emulsion matrix and a maximum loading capability of 100 kg/minute. While the charges were being inserted into the cofferdam, the basin was already filled with water and the most boreholes were filled with water at the same time. In such situations, the charging tube is placed at the bottom of the hole and when charging starts the emulsion pushes the water up out of the borehole. The uncharged part was set at about 3 metres, nearest to the International Terminal and 2-2.5 metres in the area further away. The charge height was then almost the same height as the water level, or about 2 metres below the top surface of the cofferdam.

Table 2. Technical data, Titan 7000 S.

Titan 7000 S	Properties
Density (15 m column ; 1.1 kg/m^3)	0.85-1.20 kg/m^3
Energy	3.2 MJ/kg
Weight strength *)	94 %
Volume strength *)	110 %
Detonation velocity	5000 m/s
Gas volume	950 l/kg
Water resistance	Excellent
Water pressure resistance	3 bar
Charging capacity	100 kg/min

*) ANFO as reference

In total, 65 tonnes of explosive were used the 150 metres long cofferdam shot and this covered

Figure 10. Shot firer Arne Petter Olsen connecting the redundant Nonel Unidet system.

70% of the whole barrier. Although somewhat reduced in extent, the Risavika harbour cofferdam was one of the largest, which has been shot in this country. Charging and connecting up was completed on Monday 14 April and the blasting took place at 6 p.m.

3.8 The water in the harbour basin.

Two significant milestones in Risavika harbour were reached in April 2008. On the 9 April the water was let in, after the blasting out of a carefully designed seam drill, and on 14 April the cofferdam was explosively demolished. The harbour basin was filled after only 5 hours during the night 9/10 April and about 650 million litres of water flowed in. When the basin was filled, there was not much left to see of what had become known as the "Big Hole". This water will now forever cover the enormous efforts, which the contractor and everyone involved, made.

3.9 Wave surge.

With regard to the question of the wave surge from blasting the Risavika cofferdam, experts were consulted in order to produce, if possible, an analysis with firm calculations using as background data on cofferdams in nearby regions. However it was not possible to give firm answers in the time available.

But we had a number of observations from earlier cofferdam demolitions, and from this experience were able to give some advice on established practice in this type of task.

We have experience of demolitions of cofferdams in front of hydroelectric dams. On these occasions the distance from the cofferdam to the dam, or parts of the dam, was extremely short, say 20-50 metres.

We have never experienced wave surge as a problem in this context, and so have not made any observations of waves. At Solbergsfoss, a cofferdam was blasted away about 30 metres from the dam, where the freeboard was about 4 metres, and no comments on wave surge were made. With this information we managed to establish grounds for assuming that damping towards the Tananger side 800-900 metres from the cofferdam, would be of such magnitude for it to be just a question of insignificant waves. Within the dock there would be larger waves, perhaps up to 1-1.5 metres.

Figure 11. The Risavika cofferdam demolition explosion underway. Huge quantities of water are set in motion over Vågen basin and out into Tananger fjord.

3.10 The results of the Risavika cofferdam blasting

We were witnesses to a spectacular underwater blasting, which reached at its nearest point just 15 metres from the International Terminal under construction. Sixty-five tonnes of explosive released 62,000 cubic metres of explosive gases, and fragments of 41,000 cubic metres of rock, which towered up with enormous quantities of water and stones. However, in the glass façade facing the cofferdam, only one window was damaged. Parts of the International Terminal were subjected to larger vibrations than those, which are generally accepted with reference to NS 8141. After a thorough investigation, it was later confirmed that the building had not been subject to any significant damage on account of blast induced vibrations.

A successful presplitted wall was revealed at the International Terminal, but the presplit at the cofferdam's other end did not achieve the expected result. Here the rock was already too broken up, as a result of several previous shots.

The wave surge over the harbour basin was somewhat higher than expected, with a maximum height of about 2 metres, but this did not cause any real problems. Nor did the wave height on the Tananger side, but the effect of the quantity of water in the wave surge caused some greater problems. Individual boats alongside quays in the offshore harbour got minor damages as a result of the wave surge. The remaining part of the Risavika cofferdam, consisting of 30% of the harbour, was shot at a later stage, and the same blasting technical parameters were used. The dredging of parts of the demolished cofferdam was conducted later without further need to blast boulders etc, as more powerful equipment was employed on the dredging task.

4. CONCLUSION

The underwater blasting of Norway's largest cofferdam was successful, and thereby set the seal on a well-conducted blasting operation undertaking, completely contained within the 700,000 cubic metre harbour basin. All who took part in this challenging project has gained important professional experience.

Budapest Conference Proceedings 2009, P. Moser et al

Rock dredging using underwater blasting: case study of a major port in India

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ABSTRACT: Rock dredging in ports is a critical activity. Fragmentation of hard rock at sea bed is a vital component of rock dredging, which will have a significant bearing on the economics of the dredging project. Many ports in India are going for expansion of facilities and underwater blasting operations are, therefore, increasing. Major problems encountered with underwater blasting are fragmentation and environmental effects. Other major component of rock dredging is the judicious selection of equipment, which should be able to facilitate effective and timely completion of project. Deepening of draught was carried out in front of Berth No. 8 in Tuticorin Port, Southern India, involving under water blasting in about 129,930 sq.m (155,266 sq.yards) area, for a depth of 3-4m (3.27 - 4.36 yards). This paper describes the methodology adopted and equipment used and various steps taken in the safe execution of the rock dredging project.

2 INTRODUCTION

Fragmentation of hard bed rock at the sea bed by underwater (UW) blasting is one of the most important operations in port construction/expansion/rock dredging projects. There are 12 major ports in India and most of these ports are going for expansion of existing facilities, including rock dredging, for increasing the draught. UW blasting at sea bed, for facilitating subsequent excavation and transport of fragmented material is essential when hard rock bottom conditions prevail. It is always associated with problems like obtaining required fragmentation, which is related to the type of equipment (dredgers), and minimizing environmental issues like ground vibrations and water borne shock waves. Unless these detrimental effects are controlled, they may pose problems to the surroundings, may be structures or human beings, other aquatic life and vessels. In some specific cases the working schedule may be restricted to 6-8 hours a day, due to port traffic, narrow channel or other practical conditions prevailing. In some cases, it is required to carry out UW blasting operations even at a distance of 2.5 m (2.7 yards) from the existing berths/jetties in ports. The rock dredging project, therefore, has to take all these variables into consideration.

Tuticorin Port, one of the 12 major ports in India, located in Tamilnadu Province of Southern India, is owned by the Government of India. The rock dredging project handled at Tuticorin Port is a major milestone in the history of Rock Dredging in the country. The subject of dredging the Port and related areas has been in long association with

Tuticorin Port. The earlier trend was that, whenever the sea bed encountered is hard rock having higher compressive strengths, the process of pre treating the sea bed was subjected to breaking process into fragments before resorting to grabbing. Shaped charges were used in earlier days for accomplishing the job of pre-treating the hard sea bed. Hostile marine conditions like intense wave actions and the area of dredging closer to existing marine structures like breakwaters, wharf and jetty structures complicate the dredging activity. The sea bed at Tuticorin presents a peculiar problem in tackling the capital dredging and the Port had to wait almost for more than a decade to arrive at the right type of methodology and equipment to dredge the channel and berth areas, with reasonable cost component. Underwater blasting using confined charges was planned to fragment the hard sea bed.

In order to increase the capacity of Tuticorin Port, deepening of sea bed (draught) was taken up for a depth of 3.7 m (about 4 yards) in front of multi purpose berth No.8. Draught available in front of the berth was about 7m. In order to match the draught of other berths in the Port, it was decided to dredge this berth to -11.9 m to cater to 10.7 m draught vessels. Deepening of hard rock bed was done in the dock basin in front of berth No. 8, from the existing draught of -7 m (8 yards) to -12.2 m (13.4 yards). The total area of rock dredging, which required UW blasting was about 129,930sq.m (155,266 sq.yards) as shown in Fig. 1.

2. UNDER WATER BLASTING

Geotechnical information about the hard sea bed was obtained from 25 boreholes. In addition to this, 75 boreholes were drilled at different locations, to get detailed information about the rock formation, as the borehole data was insufficient to arrive at definite conclusions about the strata. Additional rock samples were collected for assessing the geological condition of the rock bed to be blasted and for acquiring information about relevant physico-mechanical properties. The unconfined compressive strength of the rock samples, based on laboratory tests, was found to vary between 28 to 59 MPa (4×10^3 to 8.5×10^3 lb/sq.inch). Depth of rock to be dredged in different zones was determined and planning of the blasts was done subsequently, based on the condition of rock mass. Total area to be dredged was divided into different blocks, based on the compressive strength, in order to devise the underwater blast design.

Initially, a pre-blast survey was conducted involving 50 different structures, 33 on the southern brake waterside and 17 on northern brake waterside, including berth No.8. The aim of this exercise was to assess type and condition of

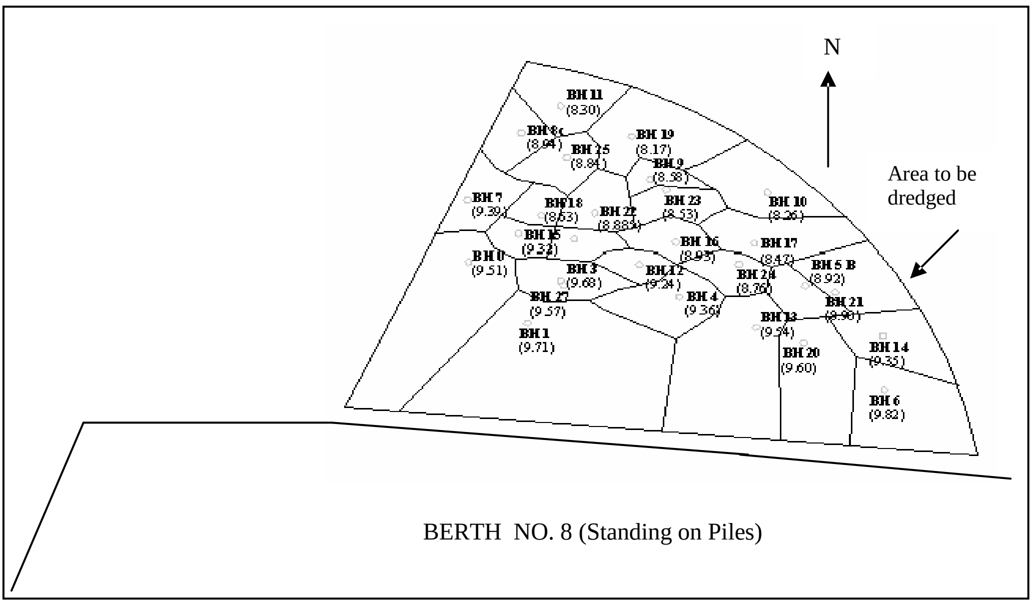


Figure 1. Area to be dredged with respect to berth no. 8

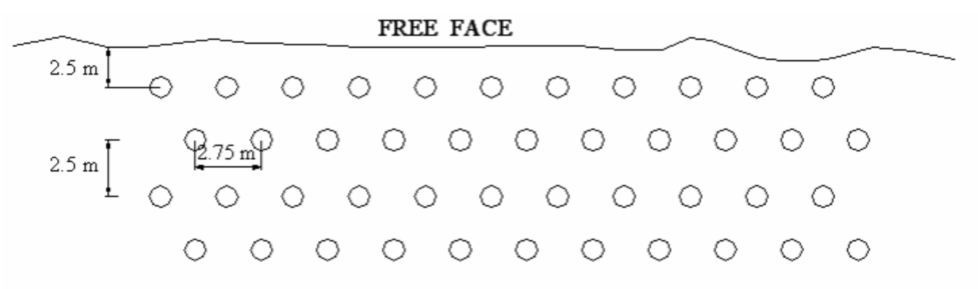
structure and to assign permissible Peak Particle Velocity (PPV) values to these structures based on their condition. Condition of structure was assessed based on different types of cracks present (major, minor and hairline), age, etc. This formed a major input for the subsequent blast design for protecting the structures from blast vibrations resulting from UW blasting operations.

3. UNDER WATER DRILLING & BLASTING

The method of drilling and blasting was adopted in this project owing to the depth to be dredged and large area to be blasted. The methodology of confined charges was adopted, involving placing of concentrated explosive charges in blastholes drilled for required depth. Normally, drilling and blasting operations are carried from a floating pontoon with drill machines mounted at one end of pontoon/vessel over a central well. The number of drill machines deployed over drill barge depends on the area to be dredged. In the UW blasting projects carried out earlier in New Mangalore Port and also in Tuticorin Port, floating drilling pontoons with central wells having four drill machines were used (Sastry 1996 & 1997). However, in another dredging project carried out in front of jetty No. 6 in New Mangalore Port, a floating barge was deployed with two drill machines mounted to one side of the barge (Sastry 1998). In the present rock dredging project, 11 DTH drill machines were mounted to one side of vessel, with each drill supported by a separate air compressor (Fig. 2).

Figure 2. Drills mounted over a vessel.

This is probably for the first time in the world that 11 drill machines were mounted on a single vessel for carrying out underwater drilling operations. Blasthole diameter was 110 mm (4.29 inch). Drill machines were mounted with a spacing of 2.75 m (3 yards), ensuring spacing between holes in a row. Burden distance varied from location to location, depending upon the distance between blast site and the nearest structure, and also yield required from blast round. The drill barge/vessel was positioned at desired locations using two units of Differential Global Positioning Systems mounted at both ends. Each blast round was made with 3-4 rows of blastholes, resulting in 33 to 44 holes (Fig. 3). Slurry explosives available



a. Spacing between blast holes : 2.75m (3 yards)
b. Burden : 2.5 m (2.27 yards)
c. No of rows : 4 (maximum)
d. Blastholes should be drilled in staggered pattern
e. Positioning of holes should be accurate, any deviation in the alignment of rows will lead to improper results (Poor fragmentation, no breakage etc.)

Figure 3. Layout of a typical blast.

Figure 4. Continuous bucket chain rock dredger.

in couplable tubes were used as explosive charges. Charge per hole varied from 1 kg to 10 kg (2.2 to 22 lb), resulting in a total explosive charge/round up to 440 kg (968 lb). Sequential blasting was adopted with appropriate delays in firing the blast rounds. In total 1900 blasts were conducted in the entire project. A total explosive quantity of 95 t was consumed in this rock dredging project.

4. ROCK DREDGING – OPTIONS

Fragmented material from the sea bed was excavated by a Bucket Chain Rock Dredger (Fig. 4). This is a continuous excavating machine having 30 buckets mounted on a ladder, each with 0.77 cu.m capacity. The average output/day is 16,000 cum. This dredger collects fragmented material from blasted area and unloads into floating barges on both sides, for carrying the material for dumping at specified area.

The major advantage with bucket rock dredger is the positive digging force, better scooping efficiency of the material and above all a continuous operation of the dredger compared to cyclic operation with the other dredgers like back hoe dredger or grab dredger. Another specific advantage of bucket chain rock dredger is the depth it can directly negotiate. Another specific feature of this dredger is that a large variety of formations it can operate with. It can negotiate not only soft formations, but also very hard and compacted formations like limonite, containing 70-80 % Fe. Positive digging force and continuous application of the buckets make it excavate even improperly fragmented strata to some extent, which is not possible in the case of back rock dredger or grab dredger. This clearly indicates that the three most important parameters for the successful implementation of hard rock dredging project are:

Suitability of the dredger,

Methodical under water drilling & blasting, and

Appropriate fixation of project duration, based on the equipment chosen.

5. ENVIRONMENTAL EFFECTS

Economy of rock dredging project depends on the fragmentation results achieved from primary blasts. As fragmentation requirement becomes finer, the cost of drilling and blasting operations increases. However, the cost of loading and transporting operations decreases, and the overall project cost becomes optimum. It is extremely difficult to implement secondary breakage in a UW blasting project. Primary blasts should, therefore, be conducted with higher charge factors. As the area of blasting is close to marine structures like berth No.8, necessary care was taken to minimize the environmental effects resulting from UW blasts. Major problems are due to ground vibrations and water borne shock waves. Environmental restrictions and obligations and therefore the environmental care formed a part of project implementation.

6. GROUND VIBRATIONS

Field studies conducted by the author in many mining and civil engineering projects revealed that the USBM equation with Square Root Scaled Distance gives better correlation coefficient compared to many other commonly followed equations. Therefore, the maximum explosive charge per delay and the total quantity of explosive per round were determined based on USBM ground vibrations propagation equation. As there are no specific standards available for marine structures like jetties/berths (piled structures standing in water) the USBM standards specifying different PPVs with respect to wide frequency spectrum were adopted for protecting the structures in Tuticorin Port from UW blasting operations.

7. INVESTIGATIONS

Initially, 22 trial blasts were conducted to establish the vibrations propagation equation to determine the safe explosive charges for different distances. Natural frequency of berth No. 8, a piled structure, was determined, by creating vibrations in the structure and analyzing with STAAD/PRO software, as 26Hz in particular and first 5 wave forms showed it as varying up to 76 Hz. This formed the basis for assigning permissible vibration levels for the pad of berth No.8, the nearest structure. USBM permissible levels were taken as a guideline for designing blasts and assessing damage potential for the structure concerned. Later, all the UW blasts were monitored on a continuous basis for the total duration of project. Blast vibrations generated were monitored using two units of Minimate DS-077 and Blastmate DS-477 of Instantel, Canada. One instrument was kept at the edge of berth and the second instrument 10–15 m (11-16 yards) away on the berth. Figure 5 shows the PPVs recorded on berth No. 8 from about 550 UW blasts. Frequency from all the vibration events (PPVs) recorded on berth no.8 was observed be >8Hz. In many cases, the predominant frequency of vibration was found to vary between 25-35 Hz.

8. PRE-SPLITTING

Blasting was carried out in sequence, covering the designated area in stages. In general, normal sequential blasting was carried out in a major portion of the area to be blasted. However, when the blasting operations were to be conducted within 30m (33 yards) from berth No.8, a pre-split plane was created to increase the safety of structure against ground vibrations. Pre-split plane is a very effective method of controlling the ground vibrations. It was created from a distance of 2m (2.2 yards) from the edge of berth No. 8. Intensity of vibrations on berth No.8 was restricted to 25 mm/s (0.98 inch/s) as per the norms, while conducting the pre-split blasts also. The depth of pre-split plane was 4 m (4.36 yards) from the hard rock bed (Figure 6).

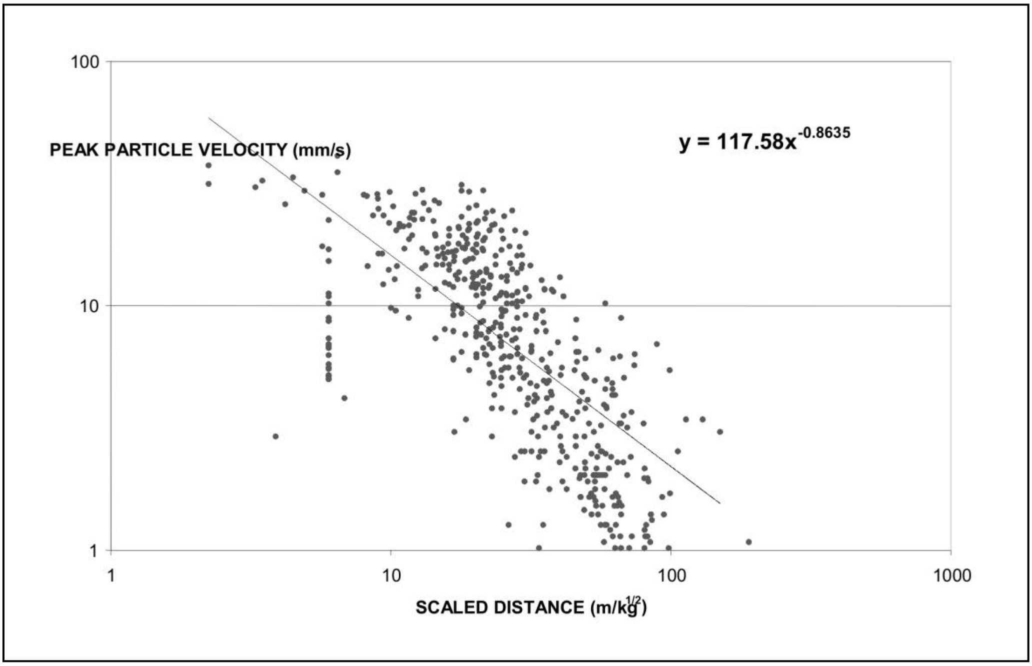


Figure 5. Peak particle velocity vs Scaled distance.

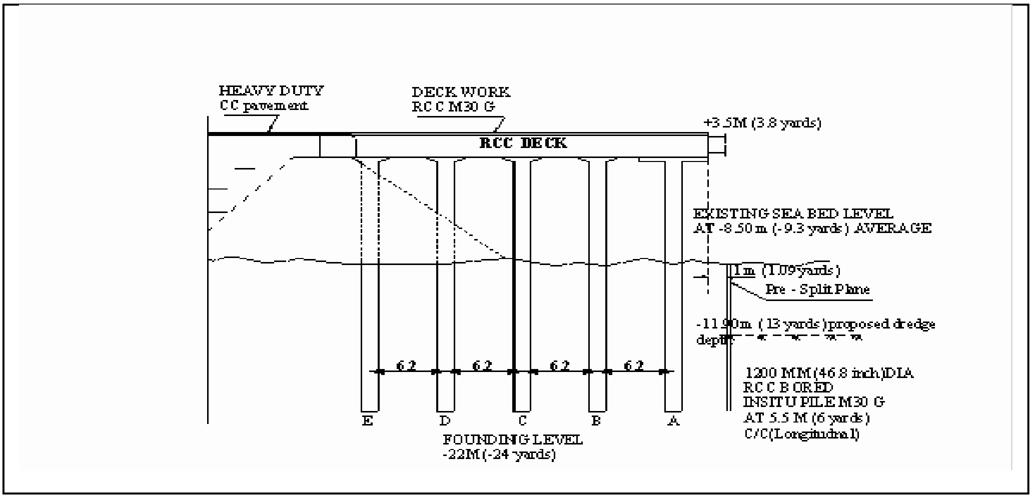


Figure 6. Cross section of pilled wharf at cargo berth – 8.

9. CONCLUSIONS

- Blasting is an essential component of hard rock dredging project in ports.
- Design of underwater blasts requires sound knowledge of geology, physico-mechanical properties of rock mass and blast design aspects.
- Cost of drilling and blasting operations should not be assessed separately, as the results from blasting operations have significant bearing on the production and productivity of subsequent dredging operation, the capital investment of which is very high.

Pre-blast and post-blast surveys and post-blast analysis help in achieving such goals.

Continuous monitoring of underwater blasts will be of immense help in minimizing the environmental effects of UW blasting operations and generating necessary data bank about the effect of blasts on structures. This will also be useful for legal protection of the contractor.

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Impact of blast design parameters on rock fragmentation and muck profile – a case study

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ABSTRACT: The mining process, when considered in its full context, consists of many separate stages or sub-processes such as drilling, blasting, loading, hauling, crushing, grinding, flotation, etc. These sub-processes make up the production chain, and each one of the stages of a chain is needed for the creation of the final product. When the blasting of a rock is performed, interactions occur between rock mass properties, explosives properties, blasting geometry and the detonation timings. A more optimised use of resources requires a better understanding of these interactions. Extensive field studies have shown that blast fragmentation influences the performance of downstream processes in a mine, and as a consequence, the profit of the whole operation can be greatly improved through optimised fragmentation. Among the above mentioned parameters, determination of optimal blast design parameters has been the main priority job for the blasting engineer on the mine. Blasting parameters have to be adjusted according to the desired fragmentation level especially and muckpile profile on the mines to ensure optimal use of loading equipments. In view of the above a study was conducted at Injepalli Limestone Mine, in India. Twelve blasts were conducted with varying blast design and charging pattern. In-the-hole VOD of explosives was determined and placement of boosters was standardized. The signature blasts were conducted to optimize the delay interval between the holes in a row and between the rows. The optimized burden was 4.5 to 5.5 m and spacing was 7.0 to 9.5 m. The hole depth was 10 to 11.5 m and were drilled with 150 mm drill diameter machine. Rock fragmentation analyses were carried out for each blast using photo-analysis system. The blasts resulted with excellent uniform fragmentation. The efficiency of loading equipments was enhanced significantly.

3 INTRODUCTION

Blasting performance is determined by the interaction of the detonation products of an explosives and confining rock mass. Rock mass properties dominate this process. The blasting engineer is, therefore, faced with the challenge of determining which rock mass properties most influence blasting performance in each situation and thereof deciding how blast designs should be changed to suit different geological conditions for better fragmentation and muck pile with low level of vibration. Fragmentation is a key factor to control and minimize the loading, hauling, crushing, classification and processing costs in aggregate and industrial minerals operations (Jose *et al*. 2006).Therefore attempts should be made to achieve the final product by blasting directly, so that the other costs can be minimized.

The establishment of the theory that expresses the relationship between a size distribution of the blasted rock and a blast design parameters is vital in order to achieve this objectives. Many researchers have given their outcome of the studies, although clear and universal theory has not established yet and each solution has been investigated at every site by a trial and error technique. The Kuz-Ram model (combination of Kuznetsov and Rosin-Rammler equations) has been widely applied to predict blast induced fragmentation since its introduction (Cunningham 1983). It allows a blast designer to quickly estimate the fragment size distribution based on a given set of rock parameters, drill pattern, and explosives loading factors. Because of the availability of quality data from production blasts, the Kuz-Ram fragmentation prediction often needs verification or calibration by other means such as an image based fragmentation analysis. Furthermore it is not clear whether such a simple model can represent in reality and the effect of blast design parameters accurately.

In most of the mining operations, drilling and blasting precede loading, hauling, and crushing operations. For this reason, it is always necessary that drill and blast operations be carried out in a manner that results in a suitable muck pile that can be handled efficiently. A good blast should always result in increase loading rate and productivity, reduced maintenance, reduced secondary breakage, and increased overall crusher performance (Singh *et al.* 2005). Thus, efficiency of any mining operation depends on the fragmentation of the muck pile produced.

The paper deals with a field study conducted at Injepalli Limestone Mine, in India in order to optimise the blast design parameters to get optimum fragmentation and subsequently improvement in productivity of the loading equipment.

2. GEOLOGY OF THE MINE

The Injepalli Limestone mine is located in the Gulbarga district of state of Karnataka in India. The location of the mine is detailed in the part of survey of India Toposheet No. 56 G/8. The area lies between East Longitudinal $77^0 8'$-$77^0 20'$ and North Latitude $17^0 03'$-$17^0 15'$. The mine area is gently undulating with average mean sea level of 420 m.

The mine geology pertains to the sedimentary rock formation of Bhima series. This fine-grained bedded deposit has no major structural disturbances. The entire area is absolutely flat terrain with a capping of black cotton soil while at some places the outcrops of siliceous limestone are seen. The general strike of the deposit is $N38^0E$ to $S38^0W$. The total remaining mineable reserve of the mine is 605.98 Mt. There are four operative benches and the blasting yields very smooth floors which indicate that the deposit is not having structural disturbances but well jointed formations (Photograph 1).

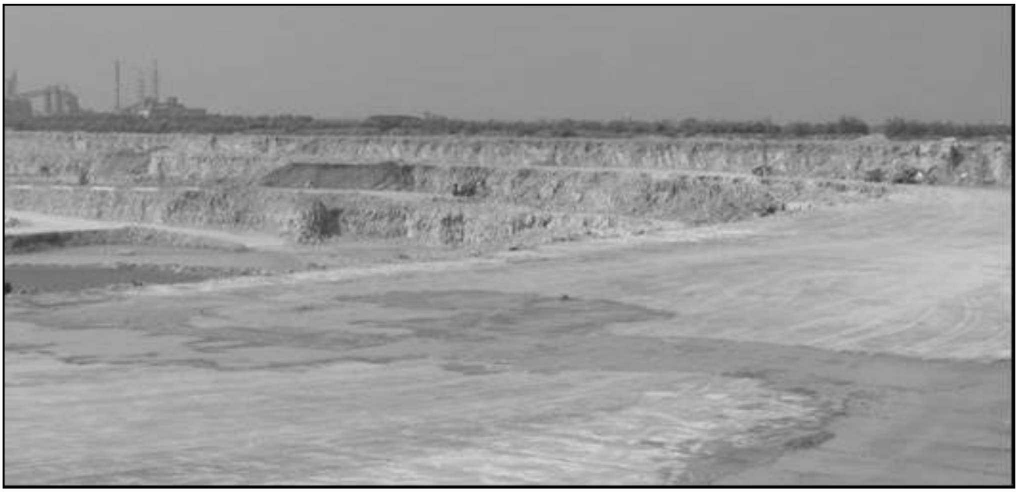
3. INSTRUMENTATIONS

Blast face profiling was performed with Burden Ace before blasting. The Burden Ace is capable of profiling the blast face in two dimensions. The rock fragmentation analyses were carried out with the help of WipFrag software of M/s WipWare Inc., Canada. The in-the-hole VOD was determined by VODMate of M/s Instantel Inc., Canada to document the quality of explosives in the hole at the time of detonation. Video camera was deployed to document the ejection of flyrocks and blast detonation sequences.Vibrations induced by production blasts were monitored by seismographs namely MiniMate Plus, MiniMate Blaster and MiniMate DS-077 (Made in Canada by M/s Instantel Inc.).

Signature blast was also conducted to document the wave transmission characteristics of the particular rock strata and transmitting media between the blast face and the concerned structures. The monitoring locations were near the blast face as well as near the foundation of structures in the nearby villages and in the structures itself.

4. FIELD STUDIES

Ten production blasts and two signature blasts were conducted. The number of blast holes varied from 1 to 80. The diameters of deep blast holes were of 150 mm. The burden and spacing were ranged from 4.5 to 5.5 m and 7.0 to 9.5 m respectively. The depth of production blast holes varied from 5 to 11.5 m and the explosives loaded in a hole varied from 50 to 149 kg. The explosives weight per delay ranged between 74 and 300 kg. Total explosive weight detonated in a blast round varied between 74 and 6150 kg. All the blasts were conducted with Nonel initiation system. The details of blast design parameters experimented is presented in Table 1.



Photograph 1. The overview of the Injepalli Limestone Mine of Vasavadatta Cement.

Table 1. Summary of trial blasts conducted at Injepalli Limestone Mine.

Strata blasted	Hole dia. [mm]	Hole depth [m]	Bench height	Burden [m]	Spacing [m]	Top stemming [m]	Explosives per hole [kg]	Charge factor [kg/m^3]
Limestone (High Grade)	150	11.5	11.0	5.5	9.5	2.5	144	0.25
	150	10.0	9.5	5.5	9.5	2.5	134	0.27
	150	10.0	9.5	5.5	9.5	2.5	129	0.26
	150	10.3	9.5	5.5	9.5	2.5	149	0.30
	150	5.0	4.7	4.5	8.5	3.0	50	0.28
	150	10.0	9.5	5.5	-	2.5	139	Signature blast
Limestone (Low Grade)	150	7.5	7.0	4.5	8.5	2.5	72	0.27
	150	7.5	7.0	4.5	8.5	2.5	67	0.25
	150	10.0	9.5	5.0	7.0	2.5	110	0.33
	150	7.0	6.7	4.5	8.5	2.5	90	0.35
	150	7.5	7.0	4.5	9.0	2.5	79	0.28
	150	7.0	6.7	4.5	-	2.5	74	Signature blast

5. MONITORING OF IN-THE-HOLE VELOCITY OF DETONATION (VOD) OF EXPLOSIVES

A uniform VOD is essentially required throughout the blast holes to produce sufficient detonation pressure to the blast hole walls to yield uniform fragmentation. The in-the-hole VOD of explosives were determined for two sets of trial blasting. In first set, hole is only charged with primer and column explosives while in second hole, the ANFO was used with primer. The VOD was recorded for first hole (primer and column explosive) was 4681 m/s (Figure 1) whereas in case of second hole (primer and ANFO), it was

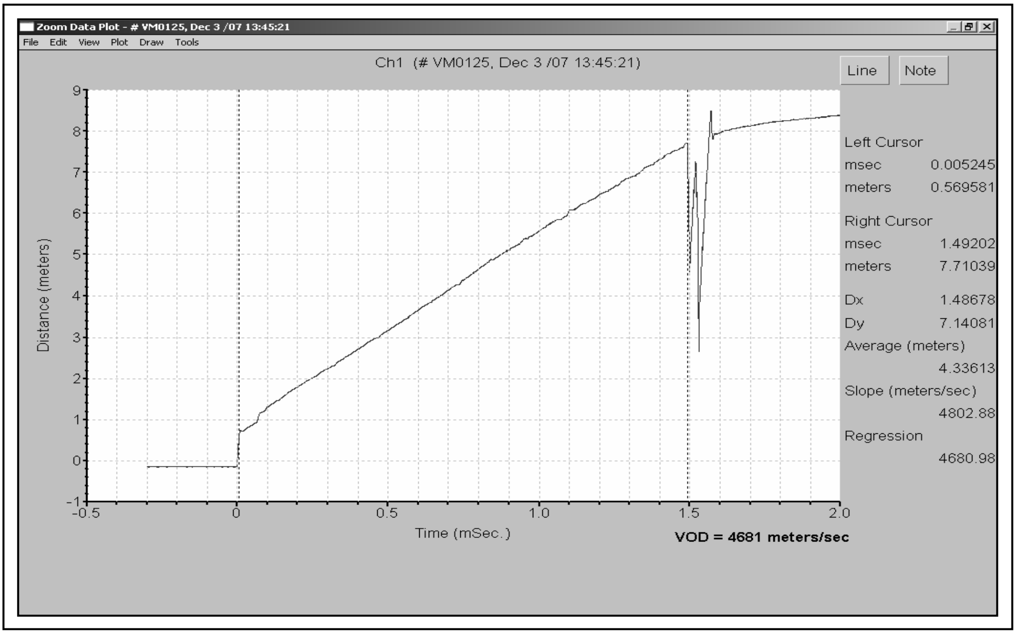


Figure 1. Trace of recorded in-the-hole VOD of explosives (primer and column).

only 3728 m/s.

6. DELAY INTERVAL OPTIMIZATION WITH THE HELP OF SIGNATURE HOLE BLAST

The optimum delay timing plays an important role in adequate rock fragmentation, swelling, displacement, fly rocks, over breaks control and in minimizing ground vibration & air over-pressure. Complete avoidance of superposition and amplification of the vibrations in a production blast is impossible to achieve because the duration of the vibration is always considerably larger than the effective delays used between the charges in smaller blasts (Valdivia *et al.* 2003).

The most effective way to minimize the effects of poor blasting practices and to reveal the effects of geology on the blasting-induced ground vibration propagation is to use a signature blast (Wheeler 2001; Adlas & Bilgin 2004). Therefore, two signature blasts were conducted at 415 RL bench and 425 RL bench of Injepalli Limestone mine. The blast hole of 425 RL bench was loaded with 74 kg of explosives whereas at 415 RL bench, it was 139 kg. The blast wave signatures were recorded at three to four locations varying from 100 to 410 m. The attenuation characteristics of blast wave were documented. The typical time history of blast wave signature recorded at 100 m from the blast hole is presented in Figure 2.

The frequency spectra of the signature blast was analysed. Linear superposition of the waves were done to simulate the waveform characteristics. For the determination of optimum delay interval it was taken in multiple of ½ or ¼ of the wavelenth of the signature blast (Rudenco 2002). The analyses revealed that very short delay times between the holes and very long delay times between the rows should be avoided. The analyses further concluded that the mean time needed to start the movement of rock face is 4.1-5.2 ms of effective burden. The delay timing of 8.2-17.6 ms/m of effective burden will give better results. The blast designs were optimised considering the out put of linear superimposition techniques.

In a few blasts, back-break was noticed which was further reduced by optimizing delay interval between the rows. The optimized delay intervals were of 65 ms to 84 ms for a burden of 4.5 m to 5.5 m (11.8 ms/m to 18.7 ms/m) of effective burden.

7. OPTIMIZATION OF BLAST DESIGN PARAMETERS

The blast design parameters for each blast were

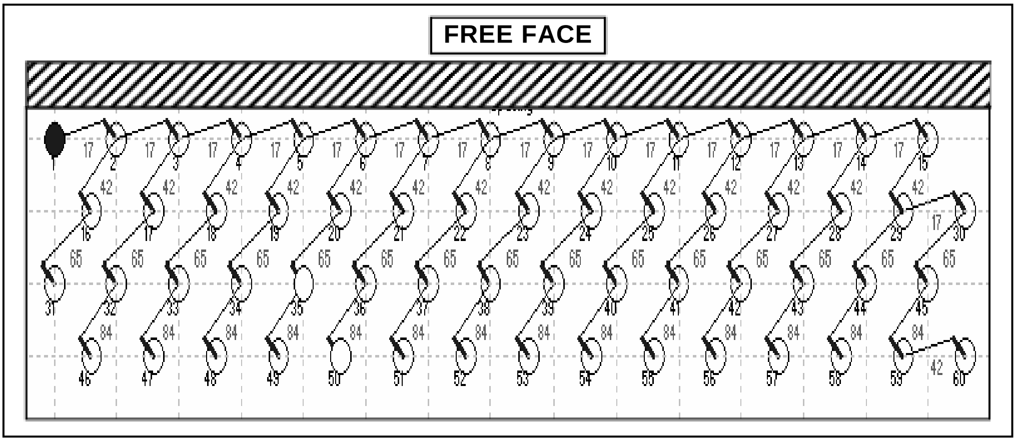
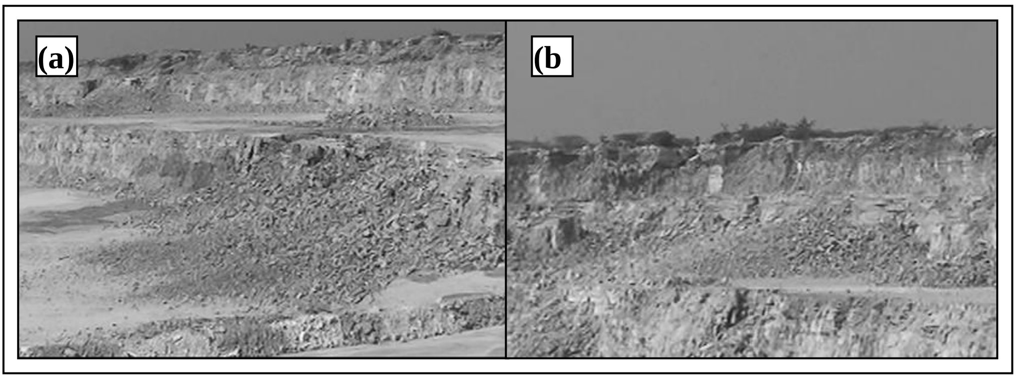


Figure 3. Delay sequence of the blast conducted at 405 m RL bench with 5 ^{0} inclinations of the blast holes.



Photograph 2. View of the throw of the blasted rock with (a) 5 ^{0} and (b) 15^0 inclination of the blast holes.

analysed in order to get optimal fragmentation suitable for loading, hauling and crushing machine. The charge factors used during the experimentation were of 0.25 to 0.35 kg/m^3. All the blasts were conducted with nonel initiation system. Rock fragmentation analyses were carried out for each blast using photo-analysis system.

A photo-analysis system and sieving are the widely used technique for measuring a size distribution of blasted rock. Under this program, photo-analysis system was adopted with the help of size distribution software "WipFrag". About 8-10 photographs of blasted muck for each of the blasts were taken from the digital camera with standard scale (1m aluminum rod). The burden, spacing and inclination of the blast holes were changed to optimise the fragment size distribution and suitable blasted muck profile for loading equipment. The blast conducted with an inclination of holes of 50 resulted in better fragmentation and muck profile compared to the blast having 150 inclined blast holes. The blast layout of the holes and their initiation sequences for the holes drilled with 50 inclinations is presented in Figure 3.

The blast holes drilled with 15^0 inclinations resulted into excessive throw of blasted material whereas the blast holes drilled with 5 ^{0} inclinations yielded very well shaped muck pile suitable for loading. The bucket fill factor of the loading equipment and cycle time of hauling equipment was enhanced significantly in case of the blast performed with inclination of holes of 5^0. The Photograph 2 depicts the view of the throw of the blasted rock due to 5^{0}and 15 ^{0} inclinations of the blast holes.

The fragmentation resulted due to blasts conducted at 405 RL bench having inclination of

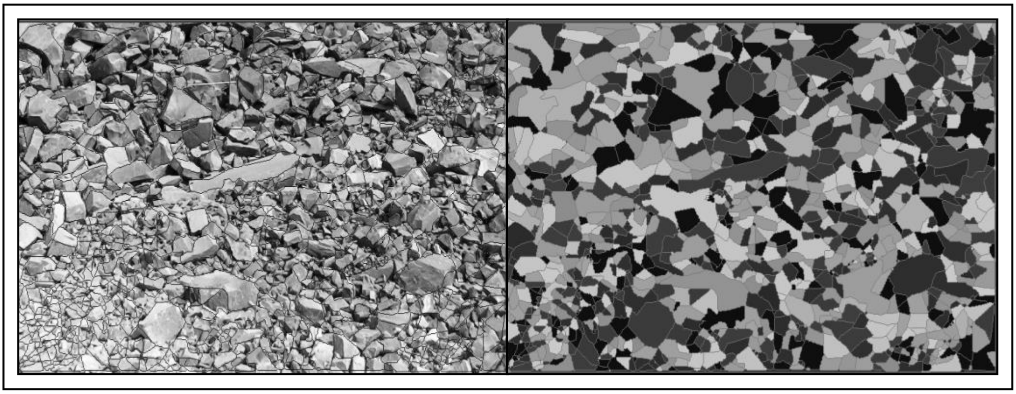


Figure 4. Netting and countering of block sizes of fragments at 405 RL bench of Pit A which yielded due to the blast holes drilled with inclinations of 5^{0}.

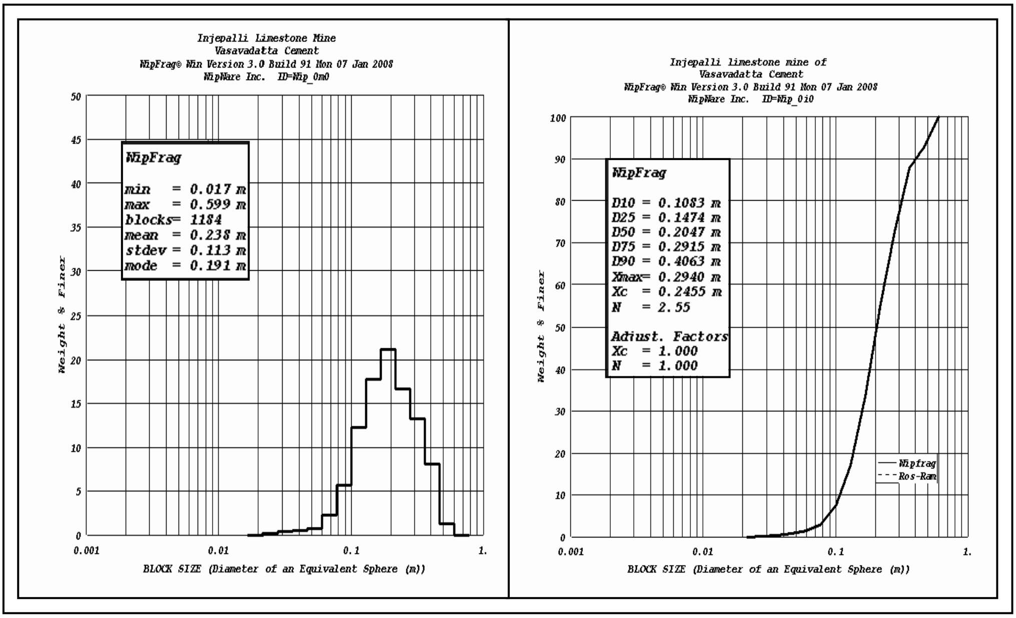


Figure 5. Histogram and cumulative size curve view of fragmented block sizes at 405 RL bench of Pit A which yielded due to the blast holes drilled with inclinations of 5^{0}.

holes upto 5^{0} and 15^{0} were analyzed with the help of WipFrag software. The output of the analyses are in the form of number of exposed fragmented blocks, maximum, minimum and mean size of the fragmented blocks, sieve analysis as per the requirement i.e. at different percentile size viz. D_{10}, D_{25}, D_{50}, D_{75} & D_{90}. (Percentile sizes: for example D_{10} is the ten-percentile, the value for which 10% by weight of the sample is finer and 90% coarser. In terms of sieving, D_{10} is the size of sieve opening through which 10% by weight of the sample would pass). The details fragment size analysis for the blast is depicted in Figures 4 and 5 for the blast having inclination of the blast holes of 5^{0}. The average mean size of the block is 0.238 m (diameter of an equivalent sphere) and the most common size of the block is 0.191 m. (diameter of an equivalent sphere). The maximum size of the boulder found in the analysis is of 0.599 m (diameter of an equivalent sphere). It is evident from the fragmentation analyses that the blast holes drilled with 15^{0} inclinations yielded coarser fragmentation compared to the blast holes drilled with 5^0 inclinations.

The Similar analyses were also carried out at different benches for the varying burden and spacing. The strata were of well jointed rock mass. The spacing to burden ratio were kept in the range of 1.56-1.73 for different benches. The ratio is much higher in comparison to those standard ranges of 1.15-1.25 times. The recorded blast vibration due to signature holes and due to production blasts were group together for statistical analyses. The recorded vibrations are plotted against their respective scaled distances. It is evident from Figure 6 that the recorded

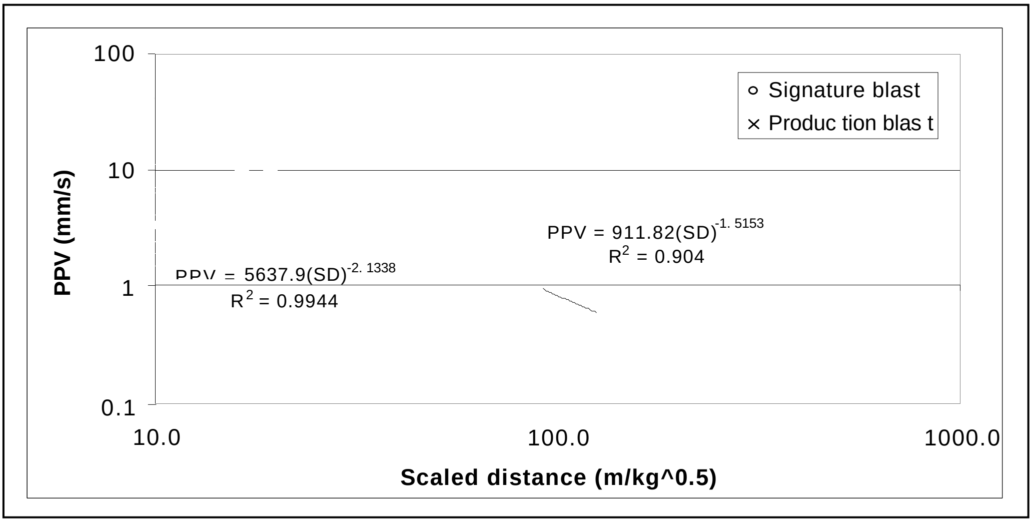


Figure 6. Plot of vibration generated due to signature hole blasts and production blasts at their respective scaled distances.

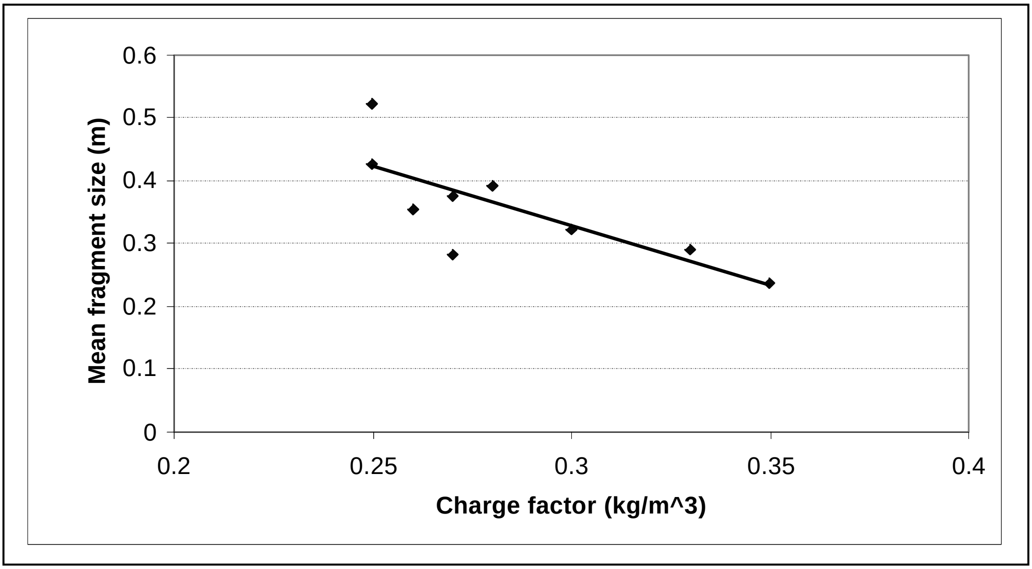


Figure 7. Impact of charge factor on mean fragment size.

vibration lev els at respective scaled distances are in agreement of each other which confirms that charge weight per delay holds-goods in prediction of vibration levels.

The hole depths was upto 11.5 and spacing was in the range of 7 to 9.5 m. It was recorded that coarser fragmentation were encountered when charge factor was 0.25 kg/cm^3 whereas when charge factor was increased by 40% there was significant improvement in the fragmentation size (Figure 7). A few experiential blasts were conducted with higher burden to spacing ratio (1.56 to 1.73). It has been documented that even if the spacing is about 95% of the depth of the hole there is no significant increase in mean fragment size (Figure 8).

8. CONCLUSIONS

It has been found that the standard scaled distance formula is valid in prediction of level of vibration. The predicted level of vibration due to single hole blasts was in agreement with the vibrations generated due to production blasts. The ratio of burden to spacing may be enhanced up to 1.8 to get desired fragmentation. The higher spacing length will economise the drilling cost and will lead to cost effective production of minerals. The inclinations of the blast holes should be minimised to 5^0-7^0 in stead of 15^0 to avoid excessive throw of the blasted material from the bottom portion of the blast face. In case of 5^0-7^0 inclined holes, explosive energy will be utilised more which improved fragmentation and ultimately reduced vibration. The recorded data of in-the-hole velocity of detonation of explosives revealed that the primer charge should be 25-30% of column charge. The quality of the column charge should be improved so that in-the-hole VOD of full column charge should be more than 4000 m/s. The placement of booster in blast holes is very important. The booster should be placed at one location when the explosive column is less or equal to 4 m.

9. ACKNOWLEDGMENTS

The authors express their thankfulness to the mine officials of Injepalli Limestone Mine of M/s Vasavadatta Cement for providing necessary facilities during field investigations. The permission of Director, Central Institute of Mining & Fuel Research, Dhanbad, India to publish the paper is thankfully acknowledged.

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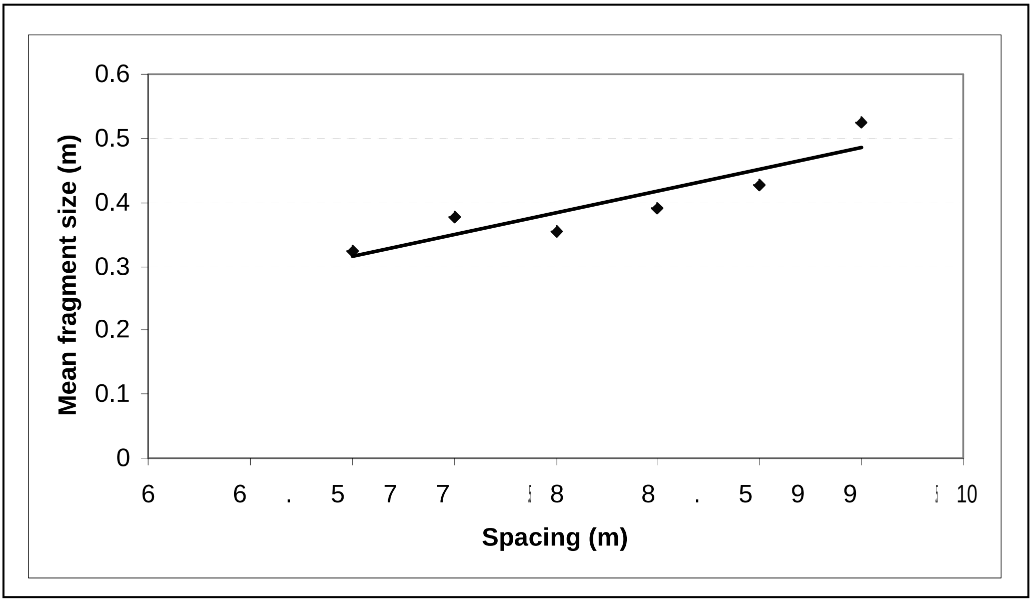

Figure 8. Impact of spacing on mean fragment size.

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Demolition of a drying factory by blasting in TPP Maritza Iztok 2

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ABSTRACT: At the moment in Bulgaria a large scale investment project for rehabilitation of certain energy units is being carried out. It also includes building of sulphur filtering installations for all of the six units. To start execution of the project it was necessary to destroy the old drying factory which included industrial buildings, reinforced concrete foundations and two reinforced concrete chimneys, both 185 m high. The chimneys were situated in the immediate vicinity of the operating workshops of the plant and their demolition was performed through blasting. The chimneys had to fall on a pre-selected area where there were no buildings or structures. Blasting works were executed according to the method of small charges in blasting holes. Special attention was paid to safety while blasting and restriction of the seismic effect during the fall of the chimneys.

1. INTRODUCTION

TPP Maritza Iztok 2 is the biggest thermal power plant in Bulgaria. It is one of the three electric power plants from the Maritza Iztok complex, which is situated in the south-east of the country. The plant is located 280 km from Sofia and 60 km from Stara Zagora. It is built on an area of 512 hectares in the immediate proximity of Radetsky village, and east of it the Ovcharitza dam is located. The average power output in TPP 2 is 1,465 Megawatts. The TPP 2 consists of eight generating units, two of which – the seventh and the eighth, have some built-in sulphur filtering equipment. Maritza Iztok 2 works with brown coal, gathered in the Maritza Iztok mines. TPP Maritza Iztok 2 is a Sole Proprietor Joint Stock Company (Sole Proprietor JSC) with 100% state participation and its purpose is producing electrical power, as well as construction and repair activities in the field of power production and heat power production. At the moment a large scale investment project for rehabilitation of power units is being carried out. The project also includes the construction of sulphur filtering installations from the first to the sixth power producing units. The aim is to reduce the noxious emissions and dust concentration, as the emissions of sulphur dioxide will be reduced by 94%. With the rehabilitation of units 1-6 the production power of the plant will increase by 156 MW.

Officially, the first project was to start on 4 October 2004, but in reality it started at the end of 2005, with the stopping of powers for rehabilitation – unit 1 was stopped on 4 October, and unit 2 – on 1 December. The average value of the project was 226 million Euros, 191 m of which are provided by the Japanese bank for

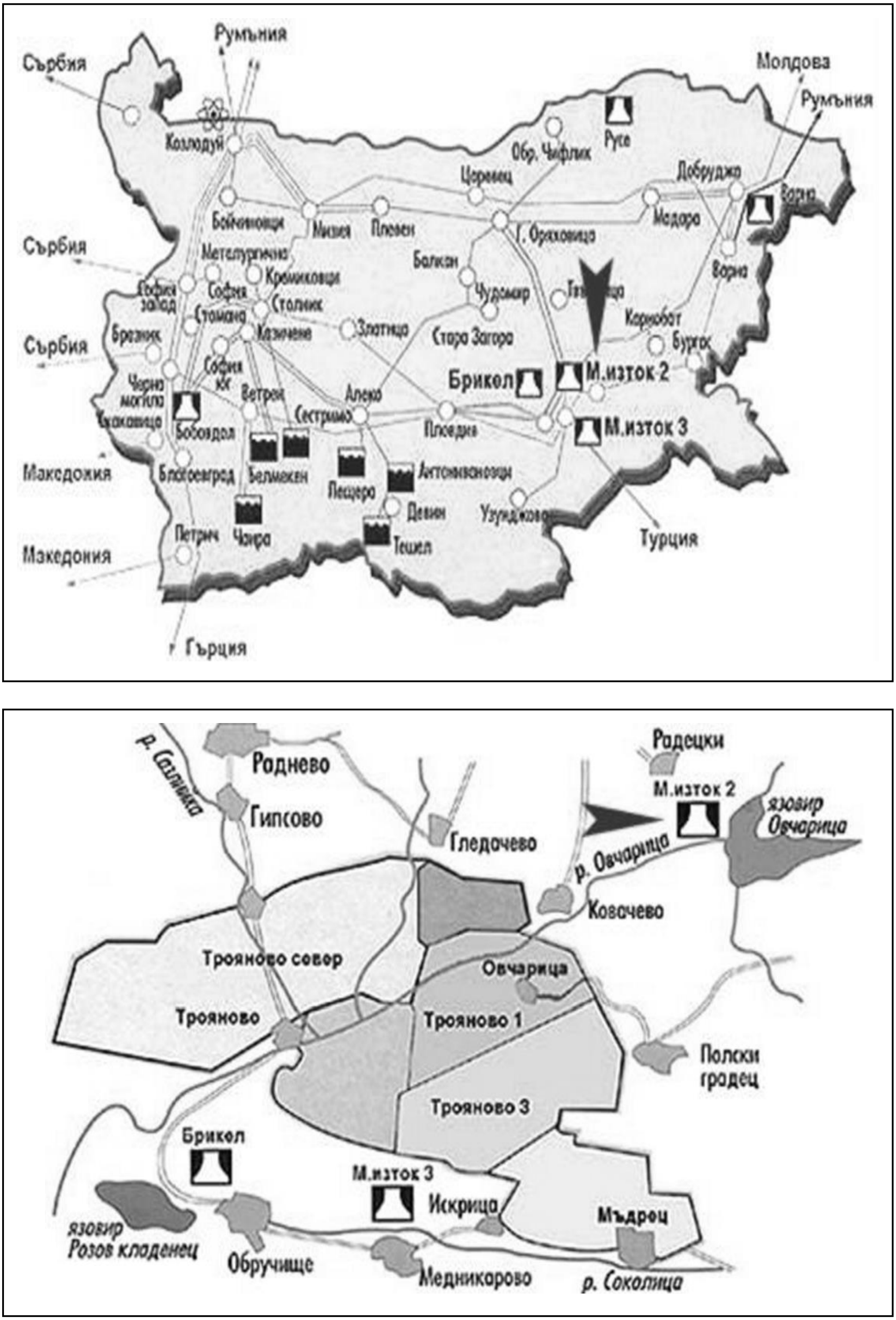

Figure 1. TPP Maritza Iztok 2 location.

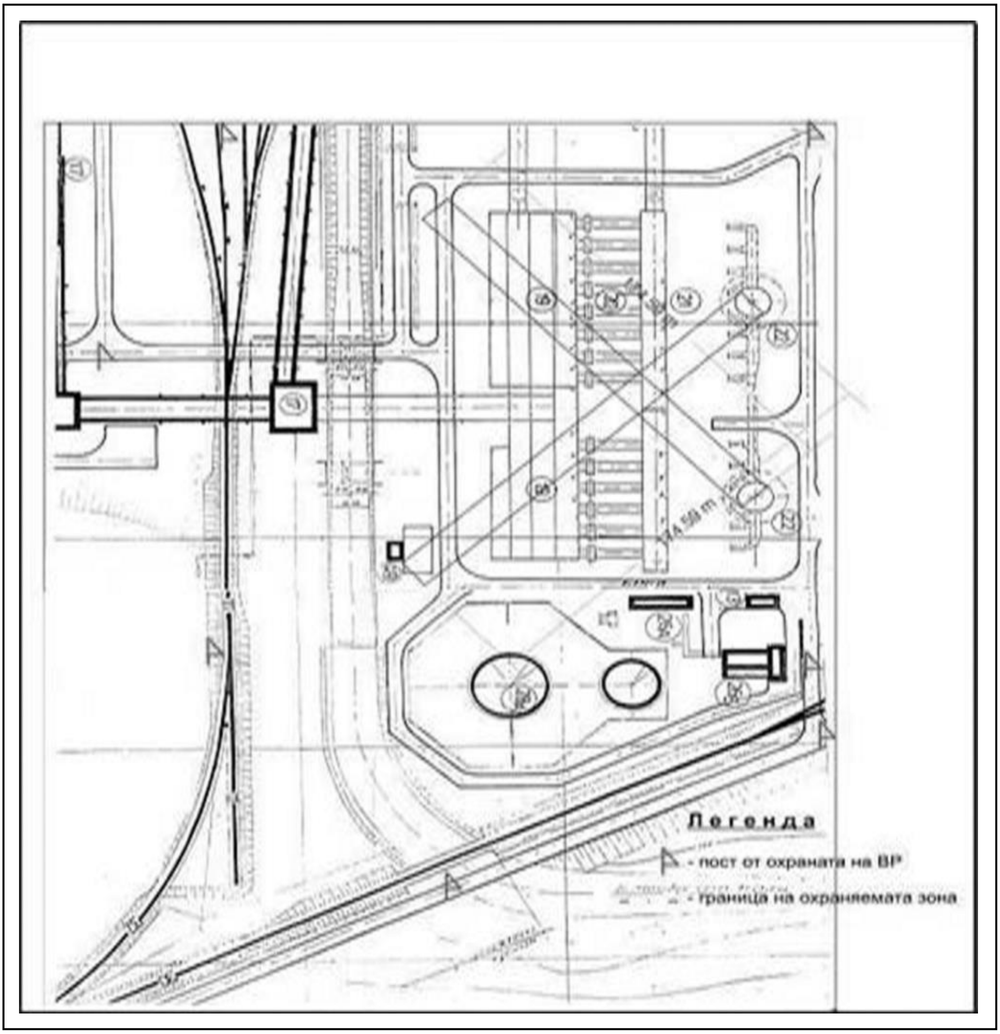

Figure 2. Plan-situation of the objects.

international cooperation through a state guaranteed loan, and the remaining 35 m are financed by means of credit.

Replacement of turbine units 1, 3, and 4, as well as of generator units 1-4 with new ones by TOSHIBA was contemplated. Unit 2 has being exploited since the beginning of 2007, and unit 1 – since the first days of August the same year. Construction and assembly works of unit 3 have being executed and the unit was stopped for rehabilitation on 12 July 2007. Unit 4 was closed 4 months later. Unit 3 came back in operation on the 1st of December 2008, whereas unit 4 was planned to resume operations on the 1st of April 2009.

The project for the construction of sulphur removing installations for units 5 and 6 is another large scale ecological investment in the plant. The project cost is 80.3 m Euros, 36.1 m of which are provided as aid under the program ISPA of the EU, 34 m as a loan by the European Bank for Reconstruction and Development, and 10.1 m are proper funds of TPP Maritza Iztok 2. The sulphur removing installations of units 5 and 6 are expected to come into use in 2010.

In terms of those projects realization it was necessary to destroy the buildings, the fundaments and two chimneys – both 185 m high. The buildings were grouped in a common ensemble, consisting of two main structures, situated one opposite another, and co-bound by a trestle, and the distance between them was 50 m. The constructions are made of unified similar elements like steel concrete beams, posts, and slabs. According to the task, the aim of drilling-blasting works is the destruction of support posts of the buildings. The subsequent destruction in full will be completed in parallel with the clearance, mainly through water-hammer. Considering the construction, the most effective and safe way for the demolition is by means of delayed blasting under the method of small charges. The most close-standing guarded objects, situated near the place of accomplishing of blasting works, were the actual industrial halls and water-mains, owned by TPP Maritza Iztok ", and standing from blasting works at 50 m, as it is shown in the plan-situation. Because of the lack of precise building documentation, the constructive sizes, the facades, and the buildings geometry were shot by the designer.

The drilling-blasting works were completed in accordance with the operative state legal orders, regarding the reflecting of documentary processes, authorization and coordination regime, reflected in the Work Safety Regulations at blasting works; Explosives, Weapons and Ammunition Control Act and the Regulations for application of Explosives, Weapons and Ammunition Control Act.

The following documentation had to be prepared and kept in the case:

A project for special blasting works – worked out by a specialized team of designers, along with a license issued by the General Labor Inspection with the Ministry of Social Welfare, which allows the team to prepare projects for special blasting works. The project is coordinated with the municipal authorities as well as with the owners of buildings and equipment, coming in the scope of blasting works if the latter are executed in built-up areas.

License for drilling-blasting works, which is issued on the basis of the project by the Sectional Inspection on labour safety in the region where these works are accomplished.

License for explosive materials – it is issued by the Department of the National Police Service, sector 'Control over means of general risk'.

License for transport of explosive materials – it is issued by the Regional Police Department, sector 'Control over means of general risk', where the explosive materials will come in.

License for purchasing of explosive materials – it is issued by the Regional Police Department, sector 'Control over means of general risk', where the explosive materials will come in.

2. DRILLING WORKS

The drilling of blast holes was performed by means of hand-held Bauer rock drills, equipped with pneumatic stands. They are fed by compressed air, which is passed through elastic hoses from a mobile diesel compressor PR-10/8f. The blasting holes were driven in with a drill g 39 mm diameter of the drill bit. The middling slime of the holes was blown away by the compressed air. The blast holes were drilled according to the scheme shown in Figure 3.

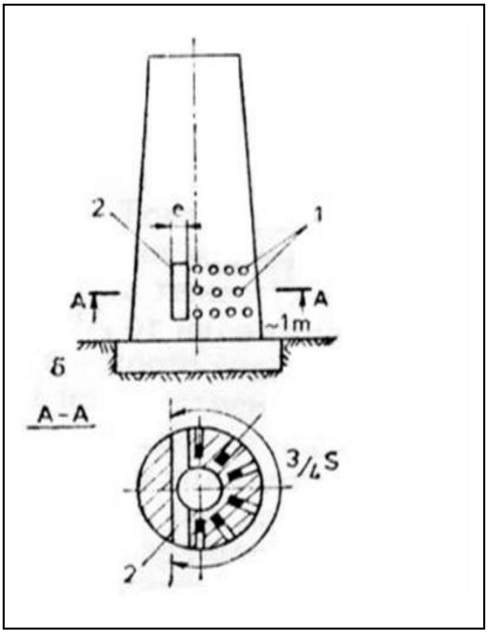

Figure 3. Scheme for drilling of blasting holes.

3. TECHNOLOGY OF EXPLOSIVE WORKS

The explosive works were performed by the method of small charges in a blasting hole. The explosive used was cartridged Ammonite, diameter of cartridges - g 28 mm. Clay was used for tamping. The construction of the charge was continuous.

The buildings and chimneys had been demolished and destroyed one-by-one, after being prepared for that. Each building had been independently demolished after the other demolished buildings before it were cleared out. The initiation of the explosive had been performed by means of electric detonators from P class with seconds delay. In order to insure demolition in the desired direction, the sequence in which the initiation was performed, pointed in the work project, was strictly followed. The desired direction of fall, by the investor, was towards the inner courtyard, between the two buildings. To limit the dispersion of explosive fragments, around the outer ends of the outer posts of the blasted building, protected covering of old rubber bands was used.

The demolition of each of the objects was accomplished one by one, in two stages, as follows:

1st stage: Oriented demolition of the entire construction of the object.

2nd stage: Crushing the fixtures and steel-concrete pieces, breaking the big pieces in full with hydraulic hammer, pushing, loading and transporting the crushed material.

4. BLASTING WORKS PARAMETERS

The results from the calculated parameters of the blasting works for one of the chimneys are presented in Table 1. The total amount of blast holes and energetic materials needed are calculated for every single object as the total sum of blast holes and energetic materials needed for similar elements which are contained in the object.

5. BLASTING SYSTEM

As we have already remarked, the initiation may be performed by means of electric detonators type "hh", with delay between degrees of 500 msec. and length of conductors 2.4 m. Blasting will be performed through main bunched conductor, with section 1.5 sq. mm, by means of blasting device, allowed for use in the country, type L – 500, having the following blasting parameters:

Tension 1,500 V

Energy 24.75 J

At cascade connection, where the electric detonators in the group are connected in series and the groups are connected together in parallel with the extent of the current, which will pass through each detonator, is as follows:

$i = I_{ij.} / m, k$

Where:

m – number of the parallel groups

$I_{ij.} = U_{lm} / R_{nmo}, k$ – extent of the current in the point of the bundle

$R_{nmo} = R_o + R_l + (R_p / m)$, - resistance of electro blasting system

$R_p = n.r/m + R_{lp}$, - resistance of the group of electric detonators, connected together in series.

R_{lp}, - resistance of district conductors used in the connection of electric detonators from the group.

As electric detonators with raised security were used, the current which has to pass through each of the detonators should not be less than 2 A.

An important and obligatory condition is passing current with equal power through all the electric detonators. To fulfil this condition in the parallel scheme of connection in series, it is necessary that the resistance of all the parallel groups after the bundle be the same.

6. ORGANIZATION OF CHARGING AND BLASTING

After the final drilling of all the blast holes is completed, they have to be charged. The charging was completed with the strict adherence to the project of blasted objects. The leaders of TPP Maritza Iztok 2 informed in advance all the people working in the area.

The exact time of blasting, region isolation and secession, is coordinated and determined together with the leadership of TPP Maritza Iztok – 2.

It is charged through mobile step-ladders.

Charging is completed by means of wooden ramrod at strict adherence to the construction of charges and the quantity of explosives for each blasting hole.

The charging is directly guided by the leader of blasting works and is accomplished by certified blasters and instructed assistants for the purpose.

The charges are prepared on the spot just before they are loaded.

The detonators are connected by the blaster after everyone else has left the blasting field and are outside the danger area.

Table 1. Blasting works parameters for chimney N°1.

		Number of the holes, items	Length, m	Number of cartridges in a hole	Stemming cm	Explosives Quantity, kg	Delay
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
1	**Left of the axe** 2 vertical rows – the first near the axis	20	0,60	2	10	6,800	0
2	2 vertical rows – the next	20	0,60	2	10	6,800	1
3	2 vertical rows – the next	20	0,60	2	10	6,800	2
4	2 vertical rows – the next	22	0,60	2	10	7,480	3
5	2 vertical rows – the next	22	0,60	2	10	7,480	4
6	2 vertical rows – the next	20	0,60	2	10	6,800	5
7	2 vertical rows – the next	20	0,60	2	10	6,800	6
8	2 vertical rows – the next	20	0,60	2	10	6,800	7
9	2 vertical rows – the next	20	0,60	2	10	6,800	8
10	2 vertical rows – the next	20	0,60	2	10	6,800	9
11	1 vertical row – next to the col- umn	10	0,60	2	10	3,400	10
12	1 vertical row – in the column	10	1,35	5	10	8,500	10
13	3 subsidiary holes – in the column	3	1,35	5	10	2,550	10
	Total left of the axis:	**227 items**				**83,810 kg**	

	Right of the axe:						
1	2 vertical rows – first to the axe	20	0,60	2	10	6,800	0
2	2 vertical rows – the next	20	0,60	2	10	6,800	1
3	2 vertical rows – the next	20	0,60	2	10	6,800	2
4	1 vertical row – next to the column	10	0,60	2	10	3,400	3
5	1 vertical row – in the column	14	1,35	5	10	11,900	3
6	1 vertical row – in the column	10	1,35	5	10	8,500	6
7	2 vertical rows – the next	20	0,60	2	10	6,800	7
8	2 vertical rows – the next	20	0,60	2	10	6,800	8
9	2 vertical rows – the next	20	0,60	2	10	6,800	9
10	2 vertical rows – the next	20	0,60	2	10	6,800	10
	Total right of the axe:	**174 units**				**71,400 kg**	

Total for chimney Nº1 **401 units,** **155,210 kg**

including	0	40 units,	13,600 kg
	1	40 units,	13,600 kg
	2	40 units,	13,600 kg
	3	46 units,	22,780 kg
	4	22 units,	7,480 kg
	5	20 units,	6,800 kg
	6	30 units,	15,300 kg
	7	40 units,	13,600 kg
	8	40 units,	13,600 kg
	9	40 units,	13,600 kg
	10	43 units,	21,250 kg
		401 units,	**155,210 kg**

The result from blasting of the first chimney is presented in Fig. 3, and the moment of leaning of the second chimney is shown in Fig. 4.

7. MAIN CONCLUSIONS

In view of the organization of the accomplished blasting works, we may conclude that an important moment of planning is the proper position of the blasting holes and determination of the necessary delays for the right guidance of the direction of chimney’s fall. It is of great importance in cases like the one described when it is in close proximity to costly buildings and equipment in use. In order to decrease the seismic effect from the chimney’s fall, at the place of fall, a sandy pillow 50 cm thick was put on the place, determined for the fall. The last action prevented any possible failures of the metric instruments and neighbouring workshops.

Figure 4. Scene of chimney Nº 1 fall.

Figure 5. Leaning of chimney Nº 2.

Those were the two highest chimneys in our country, ever demolished by means of blasting, and situated in the immediate vicinity of the workshops in the plant, operating without turning the duties off.

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