# A New Approach to Blasting Induced Ground Vibrations and Damage to Structures

***Sasa Stojadinovic*[1]*, Miodrag Zikic* and *Radoje Pantovic***

*This article presents a new methodology for determination of causal connection between open pit blast induced ground vibration and damage to structures and liability for the damage. The basic hypothesis of the methodology is that damage is a result of joint action of both blasting and non-blasting related factors and that neither of these factors can be neglected regardless of its small strength. Additional hypothesis is that it is possible to calculate the percentage of blasting factor influence in total damage to structure. The methodology is primarily based on crack response monitoring approach but also defines and exploits the principle of partial liability which states that blaster should be held liable for the damage but only if the causality is proven and only to an amount equal to the percentage of blasting influence in total damage. The article gives guidelines for determination of causality and explains the procedure of blasting influence percentage calculation on an example.*

**Key words**: *methodology, blasting, ground vibrations, partial liability, percentage; damage*

## Introduction

Blasting at open pits is potentially dangerous operation if it is not carried out in accordance with regulations and if all necessary protection measures are not undertaken. When there are residential and other structures in the proximity of open pit, there is additional danger that they will be jeopardized by negative effects of blasting i.e. flyrock and vibrations.

Most homeowners consider this jeopardy as invasive and complaints and law suits are common. The subjects of the complaints go from disturbance to damage to structures. In cases of law suits starts the whole process of confirmation and disproof of liability for jeopardy of structures.

## Traditional approach to the problem of liability for blasting vibrations caused damage

The liability for damage to structures is confirmed or disproved upon a testimony of expert witnesses. Their primary task is to determine the existence of cause-consequence (causality) connection between the blasting operations and damage to structures.

According to traditional approach (Adhikari, 2005, Adhikari, 2007, Stark, 2004, Svinkin, 2010), causality between the blasting operations and damage to structures, and hence blasters liability, exists only if the particular damage appeared during passing of the blast induced seismic wave. In other words, if damage existed prior to the blast there is no liability. Proof and disproof of the causality according to this approach, comes up to providing answer to two basic questions

1. Was the peak particle velocity of ground motion during blast induced vibrations higher than the maximum allowed?
2. Did the particular damage exist prior to the blast or not?

The answer to the first question indicates the potential of blast induced ground vibrations to cause damage while the answer to the second question gives affirmation or negation of causality existence.

This traditional approach, for determination of causality, considers only the moment of appearance of new cracks in the structures. From that point of view, this approach can be justified only in the case of undamaged, solid and vibration resistant structures. However, a problem arises when the particular structure is already damaged and hence weakened or its resistance degraded. In that case, a new, crack response monitoring approach is used. This approach was suggested by C.H. Dowding at all, and is now widely accepted in various vibration caused damage research (Aimon-Martin at all, 2003, Aimon-Martin and Dowding, 2004, Mann, 2003, Meissner, 2010, Siskind at all, 1980). The crack response is in the form of changes in crack width and causality is determined based on the magnitude of this response. If the magnitude of crack response is smaller than response to other, non-blasting related influences, the seismic wave and resulting ground vibration have no potential to cause damage to structure so there is no liability.

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A number of researches (Aimon-Martin at all, 2003, Aimon-Martin and Dowding, 2004, Mann, 2003, Meissner, 2010, Nicholls at all, 1970, Siskind at all, 1980) point out the significant influence of non-blasting related factors such as daily temperature changes, variations in air humidity, foundation settlement and human everyday house activities. Results of these researches indicate that the influence of blast induced vibrations to structures is much smaller compared to other non-blasting related factors and when that is the case, the blasting influence should not be taken into account. However, even in those cases when blasting influence is considerably smaller than other influences it can not, and should not be neglected because the fact is that it exists.

The facts that were not emphasized enough in the mentioned researches are:

1. The monitored cracks were cosmetic damage in plaster and not structural cracks in constructive elements of the structure.
2. The influence of non-blasting related factors is, by rule, a slow process lasting from several hours (daily temperature changes) to several days or even years (ground settlement and subsidence)
3. Non-blasting related influences give rise to static stresses in the constructive elements of the structures. Dynamic stresses can appear as a result of everyday human activities but are highly localized and can not excite the structure in whole.
4. The influence of some non-blasting factors is often periodical, progressive during one period and regressive during the next.
5. Daily temperature changes and humidity variations do not have the potential to cause structural damage and can only cause cosmetic damage. Only the soil and foundation settlement can cause structural damage.
6. Blast induced ground vibrations are short term, lasting several seconds, but can transmit significant amount of energy to the structure.
7. Stresses that appear in constructive elements of structure as a result of ground vibrations are dynamic.
8. Passing of the seismic wave can excite the structure in whole.
9. Crack responses to blast induced ground vibrations, the changes in crack width are, as a rule, progressive.

These weaknesses in the crack response monitoring approach were the cause for the development of a new methodology for determination of causality between blast induced ground vibrations and damage to structures. In addition, an integral part of this new methodology is the model of calculation of percentage of blast induced vibration in total amount of damage.

## Determination of the causal connection between blasting vibrations and damage to structures

The basic hypothesis of the methodology is that damage to structure is a result of joint action of both blasting and non-blasting related influences. The term damage is defined as the appearance of new cracks in structures as well as the increase in dimensions of already existing cracks. Of course, it does not mean that in all cases of blasting operations in the proximity of residential and other structures the blasters should be held liable. The liability exists only in those cases when it is possible to prove the existence of the direct influence of blast induced ground vibrations to structure and liability is proportional to the percentage of blast related influence in total amount of damage. In accordance to that, this methodology defines and exploits **the principle of partial liability** which states

**If the causality between blast induced ground vibrations and damage to the structures is proven, then the blaster should be held liable for the damage, but only to the amount equal to the percentage of blast related influence in total amount of damage.**

The existence of causality is proven by direct measurements of crack response to ground vibrations. The crack response is defined as any permanent deformation or change in crack dimensions (commonly crack width). The measurements should be performed by placing displacement gauges (preferably LVDT-s) on the structural cracks, and avoiding superficial, cosmetic cracks. The reason for that is difference in the responses of structural and cosmetic damage to both blasting and non-blasting related factors. Cosmetic crack are located in the plaster. Large surface of plaster, in relation to its thickness makes the plaster highly subjected to environmental changes (temperature and humidity). Thermal spreading or humidity caused shrinkage of plaster result in significant linear deformations considering that two dimensions of plaster (length and width) are considerably larger in relation to the third (thickness). Structural cracks are located in the constructive elements of the structure (walls, beams…). Due to a different dimensions ratio, deformations resulting from environmental changes are volumetric. On the other hand, excitation of plaster, as a constructive element cover, by ground vibration is weakened since it is partially dumped in the constructive element and partially in the juncture wall-plaster. The response of structural cracks to excitation by ground vibration is direct.

Therefore, the results of deformation measurements on cosmetic cracks will be larger than measurements on structural cracks. Due to this nature of deformations the response of structural cracks is much more realistic than the response of cosmetic cracks.

Reliable determination and measurement of structural crack response considers anchoring and bolting of the instrument into the constructive element of the structure since, in this way, the instrument will register the excitation of the wall and not the plaster. This procedure is direct and explicit. If the crack does not respond to excitation from ground vibrations it can be reliably stated that there is no influence, hence there is no causality and liability.

**The influence of non-blasting related factors**

Non-blasting related factors that can influence the appearance and development of damage are, before all, ground and foundation settling, daily temperature changes and variations in air humidity. Considering the duration of these influences, resulting stresses in constructive elements of structure should be considered as, and are static. Oposed, dynamic stresses can be result of everyday human activities but are localized and can not affect the entire structure (Nicholls at all, 1970).

Daily environmental changes and air humidity variations have periodical character and are progressive over some periods and regressive in the next (Aimon-Martin at all, 2003). Deformations of cracks, i.e. crack responses are not permanent and are dependant on the type and duration of the influence.

Foundation and ground settling is the only non-blasting related factor that has the potential to cause structural damage. However, when structures are founded in the soil subjected to subsidence, or in loose sends saturated with water, blast induced ground vibrations can be a trigger and initiate subsidence and settling of the soil that would otherwise remain stable. This phenomenon is known as vibration or dynamic settling and triggering particle velocities of ground vibrations can be as low as 2 mm/s (Lacy and Gould, 1985, Svinkin, 1999).

**The influence of the blasting vibrations**

During the blast induced seismic wave passing, vibrations are transferred from ground to structure and the structure begins to oscillate in the regime of forced oscillator. The ground vibrations are exciting and the structure vibrations are forced motions. During these vibrations, constructive elements of structures are deformed and dynamical stresses appear. If magnitudes of these stresses exceed the values of corresponding constructive element material strengths, deformations become plastic resulting in damage appearance. In dependence on constructive characteristics of structure, quality of building materials and general condition of structure its resistivity to vibrations can be higher or lower. The problem arises with structures that are already damaged to some extent.

In the zone around the structural crack, stress/strain condition of material is disturbed and its resistivity degraded. Considering that internal connections in the material and its consistency, which would normally resist stressing, are weakened or broken, crack sensibility to stress is increased. The crack then responds even to those stresses that, in normal conditions, would not cause plastic deformations. During the excitation duration, crack width changes in accordance to movements of constructive element governed by ground vibrations. When ground and resulting structure vibrations stop, depending on the intensity of excitation, crack width can be changed compared to the width before the vibrations. This change in crack width represents the residual deformation of the crack $\mathbf{\Delta s_i}$ (Fig. 1).

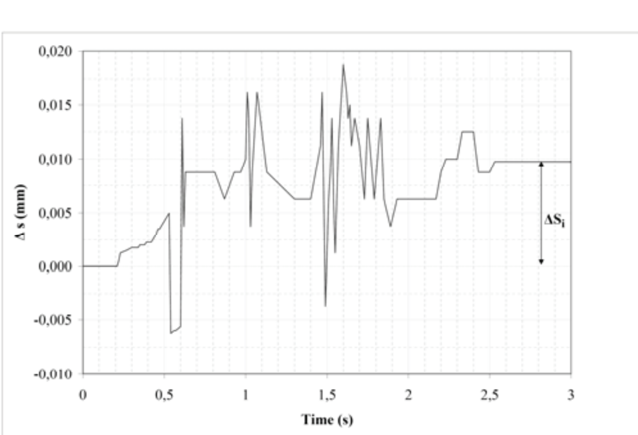


*Fig. 1. Typical crack response to blast induced vibrations excitation and residual deformation.*

Considering the fact that, if non-blasting related influences are excluded, there is no mechanism that would cause the crack to return to previous condition, this residual deformation due to the blast induced ground vibrations is permanent. Hence, with every new blast and resulting ground vibrations, crack width increases according to Eq. (1) and the damage is progressive (Fig. 2).

$$s_{i+1} = s_i + \Delta s_i \text{, (mm)} \quad (1)$$

where:

**$s_{i+1}$** - crack width after the **i** blast,
**$s_i$** - crack width prior to the **i** blast (mm),
**$\Delta s_i$** - residual deformation after the **i** blast (mm).

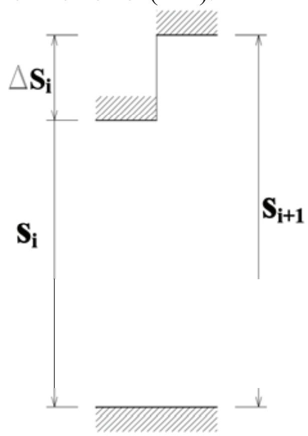


*Fig. 2. Increase of the crack width.*

This of course does not mean that all cracks respond to every, even smallest vibrations. There is a limit level of ground vibrations intensity that causes crack response. In other words, cracks respond only to the ground vibrations intensity, expressed via the particle velocity, which has the potential to excite them. Considering that, from an engineering point of view, it is necessary to determine the minimum particle velocity of ground vibrations that causes crack response and the relationship between the particle velocity of ground vibrations and magnitude of residual deformations. In order to achieve that it is necessary to simultaneously measure the particle velocity of ground vibrations and corresponding magnitudes of crack responses. In those cases when low intensity ground vibrations which can not trigger the seismograph are expected, or it is not possible to directly measure the ground vibrations particle velocity, corresponding particle velocity is calculated upon the propagation law, for the 50 % confidence line. The 50 % confidence line should be used because these values of particle velocities of ground vibrations have the highest probability of appearance.

Since the damages of different intensities have different reaction to excitation (cracks with larger width have larger residual deformations), the results of the measurements should be brought down to relative values, in order to achieve comparability. Therefore, the term relative crack deformation **$(\Delta s/s)_i$** is defined as the ratio of absolute value of the change in crack width (residual deformation **$\Delta s_i$** ) after the excitation has stopped and the value of initial crack width (**$s_i$**) prior to the excitation as:

$$\left(\frac{\Delta s}{s}\right)_i = \frac{\Delta s_i}{s_i} \text{, (mm/mm)} \quad (2)$$

Relationship between residual deformation given as relative value and ground vibration particle can be expressed as

$$\left(\frac{\Delta s}{s}\right) = f(v_{50}) \text{, (mm/mm)} \quad (3)$$

The function **$f(v_{50})$** is determined upon statistical analysis of the field measurements data.

It should be noted that if Eq. (3) gives negative result it does not mean that crack width decreases but simply that the excitation is not strong enough to trigger crack response.

The limit, minimal value of particle velocity of ground vibration that causes the crack response can then be calculated from the condition that relative deformation is equal to zero.

### The model for calculation of percentage of blasting vibrations in total damage

In principle, the basics of the calculation of percentage of blast induced vibration influence in total amount of damage can be presented graphically (Fig. 3). In this case partial influences of both blasting and non-blasting related factors are for the reason of clarity displayed as straight lines, whilst in reality these are complex curves (detail **a** in Fig. 3).

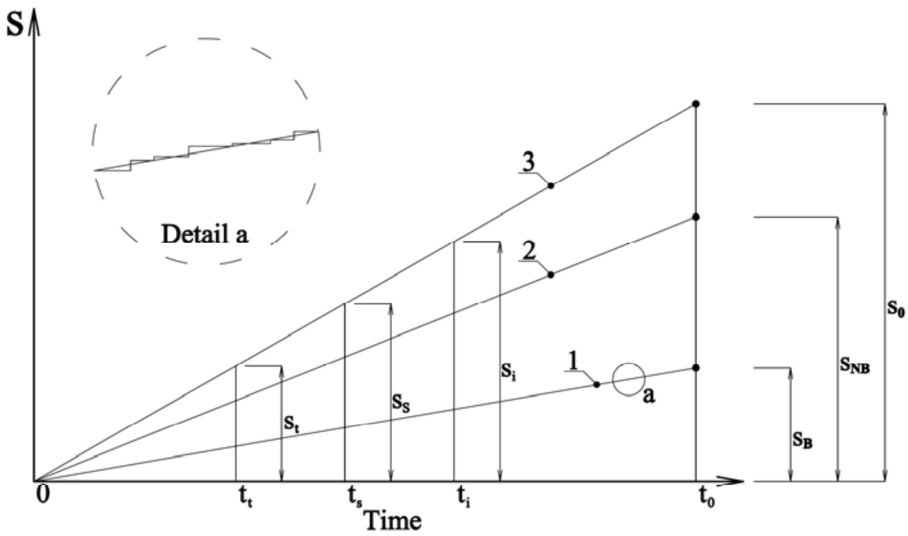


*Fig. 3. Graphical presentation of the principle of partial liability.*

The point $\mathbf{t_t}$ in Fig. 3 represents the moment of appearance of threshold damage in a form of hairline (76 μm) crack with the width $\mathbf{s_t}$. From that moment on, crack grows due to a joint influence of blasting and non-blasting related factors. In a point of time $\mathbf{t_s}$ damage becomes structural and crack reaches width $\mathbf{s_s}$. The trend of growth or development of the damage is displayed as the line 3 and the crack in the moment of observation $\mathbf{t_0}$ reaches width $\mathbf{s_0}$.

If the blast induced vibrations influence did not exist the crack would grow under the influence of environmental factors only, following the trend displayed as line 2. In those conditions, in the moment of observation $\mathbf{t_0}$ the crack would have the width $\mathbf{s_{NB}}$. Also, under the isolated influence of blast induced ground vibrations the crack would grow following the trend displayed as the line 1 and in the moment of observation $\mathbf{t_0}$ would reach the width $\mathbf{s_B}$.

The ratio of width that crack would reach under the isolated influence of blast induced ground vibration and the width reached under the joint influence of blasting and non-blasting related factors represents the participation of blast induced ground vibration influence in total amount of damage (Eq. 4).

$$P_B = \frac{s_B}{s} \cdot 100\text{, } (\%) \qquad (4)$$

In accordance to that, the blaster should be held liable for equivalent percentage of total value of damage to structure. It is important to point out that, if precise moment of damage appearance and precise data on prior blasts are known, it is not necessary to know the trend of crack growth under isolated influence of non-blasting factors.

In order to calculate the percentage of blasting factors in total amount of damage to structure it is necessary to define the trend of crack development. To that effect, according to the methodology, the starting point is the crack width in the moment of observation $\mathbf{t_0}$ in Fig. 3. Further analysis follows the crack "in reverse" towards the moment of its appearance, and crack width is practically decreased in accordance with the (reversed) trend of growth. The reason for this reversed observation lies in the fact that, in most cases, it is practically impossible to precisely determine the exact moment of crack appearance which would allow normal (forward) crack growth observation.

With known limit value of particle velocity of ground vibrations, known relationship between the residual deformation and particle velocity of ground vibrations, and with know blasting parameters for previous blasts, it is possible to determine to which extent did the blasts from any observed period in the past influence the development of damage. Mathematical interpretation of this model can be expressed as follows

$$s_1 = \frac{s_0}{1+\left(\frac{\Delta s}{s}\right)_1}, \quad \left(\frac{\Delta s}{s}\right)_1 = f(v_{501})$$

$$s_2 = \frac{s_1}{1+\left(\frac{\Delta s}{s}\right)_2}, \quad \left(\frac{\Delta s}{s}\right)_2 = f(v_{502})\ldots$$

$$s_e = \frac{s_{e-1}}{1+\left(\frac{\Delta s}{s}\right)_b}, \quad \left(\frac{\Delta s}{s}\right)_e = f(v_{50e})\ldots$$

$$s_b = \frac{s_{b-1}}{1+\left(\frac{\Delta s}{s}\right)_b}, \quad \left(\frac{\Delta s}{s}\right)_b = f(v_{50b})\ldots \tag{5}$$

where:

- $\mathbf{s_o}$ - initial crack width in the moment of observation;
- $\mathbf{s_1}$ - crack width before the last blast;
- $\mathbf{s_e}$ - crack width at the end of the observed period;
- $\mathbf{s_b}$ - crack width at the beginning of the observed period;
- $\mathbf{v_{50i}}$ - peak particle velocity of ground vibrations during **i$^{th}$** blast, registered at the location of the structure or calculated according to the propagation law, for 50 % confidence line.

Graphical interpretation of the model is given in Fig. *4*.

This mathematical model excludes the influence of non-blasting related factors and its application defines the amount of damage expressed as the increase in crack width which is the result and consequence of isolated influence, in the observed period, of blast induced ground vibrations only. The percentage of damage for which, in given period, the blaster should be held liable, is calculated as:

$$P_P = (1-\frac{s_b}{s_e})\cdot 100, \; (\%) \tag{6}$$

It should be pointed out that Eq. 6 gives the percentage of blasting factors influence in total amount of damage for a specific observed period. The period can be set to cover total lifespan of the crack or to cover a shorter period. The length of the observed period depends on the specific demands (the need to define the blasting influence for the last several years or in some predefined period in the past…). The length of the observed period also depends on the precision of the input data, especially the parameters of the blast (weight of explosive per delay and the location of the blasting series). The precise knowledge of blasting parameters is essential for calculations of PPV during previous blasts, for the location of the structure. It is pointless to set the observation period over the period of time with unknown blasting data. Once more, it is pointed out that the precision and validity of the estimate of percentage of blasting factors in total amount of damage depend on the precision of input data.

In those cases when it is possible to precisely determine the exact moment of the crack appearance, and with reliable blasting data, it is possible, with high accuracy and for the whole crack lifespan, to estimate the percentage of damage for which the blaster should be held liable.

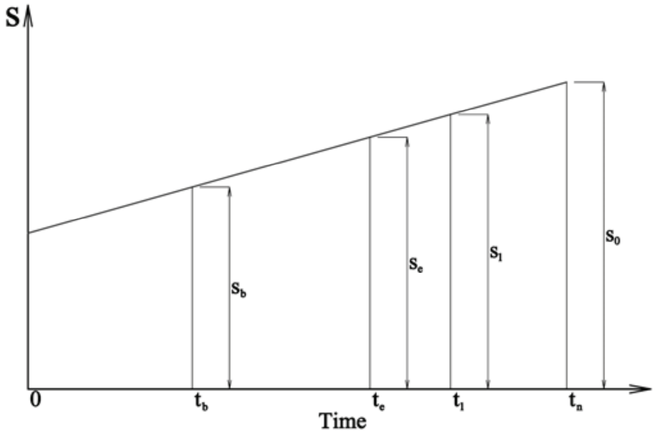


*Fig. 4. The model of crack development trend.*

## Validation of the methodology

The methodology was developed for the purposes of expert witness analysis for the Municipality court in Paracin, Serbia. The subjects of the analysis were 115 structures in the Popovac village, covering 50 individual complaints and joined into 30 court cases. The structures are located between two active quarries, Cokoce (limestone) and Tresnja (marlstone). Both quarries were opened in 1955. The average annual number of blasts was 13 on Cokoce and 26 on Tresnja. The blasting activities at Tresnja quarry stopped in 2003. The distance between the quarries is 2 km and the closest structures are located at 250 m from Tresnja and 600 m from Cokoce.

In order to validate the methodology it was necessary to test the results on a structure with known history. The chosen structure is located at 600 m from the Cokoce quarry. It was built in 1980 as one story masonry structure with reinforced concrete frame. It has a concrete foundations up to 1,5 m deep. The owner had the construction project and building license. During the inspection of the structure no signs and indication of ground and foundation settling were found. There are no underground waters and the soil is not subjected to saturation.

The inspection of the structure revealed several structural cracks characteristic for vibration damage. The cracks were diagonal, spreading from the corners of windows and doors. Also, several cracks at the joints of door and window frames with walls were registered. No damage indicative to non-blasting related factors was noticed.

During the period from 1980 to 1990 at almost every blast on Cokoce more than 1300 kg of ANFO detonated instantaneously. Several blasts had more than 2000 kg of ANFO detonated instantaneously and one blast from May 5$^{th}$ 1980 had 6580 kg of ANFO detonated instantaneously. Average amount of explosive per blasting series was 15000 kg and average amount of explosive per blasthole was 350 kg.

Since the inspection of the structure did not reveal any signs of non-blasting related factors and that all registered cracks were characteristic for vibration damage the conclusion was that cracks appeared and developed under the influence of blast induced vibrations. The exact moment of crack appearance was not known but, considering the strength of the blasts, it was most probably in the late ‘80s. To confirm the validity of the methodology, the analysis “backwards” as previously described needed to show that hairline crack appeared sometimes between 1987 and 1990, considering the time needed for strains to accumulate and reach critical values. A hairline crack is a 76 µm wide crack and it represents the threshold of cosmetic damage. Such a result would be considered as a close match.

The first step was the definition of the propagation law for estimation of particle velocities of ground vibration for the blasts in the past. Considering the frequency of the blasts, deadline given by the Court and number of available seismographs the propagation law was defined upon the measurements from six locations, during four blasts. The propagation law for 50% confidence line was defined as

$$ppv = 336{,}13\left(\frac{d}{\sqrt{Q}}\right)^{-1{,}2997}, \quad (mm/s), \quad R^2=0{,}6031 \qquad (7)$$

where

**ppv** - peak particle velocity of ground vibrations;
**d** - distance from the blasting series, (m);
**Q** - mass of explosive detonated per delay, (kg)

The second step was to determine the relationship between residual deformation and ground vibrations particle velocity. Crack responses were monitored and measured on structural cracks, with displacement gauges anchored into constructive elements of the structures at six locations (Fig. 5). The results of the crack responses measurements are given in Table 1.

The results of the measurements 7, 8, 9 and 10 from the table were not used to determine the relationship between residual deformation and ground vibrations particle velocity since locations were not secured and the validity of those measurements could later be questioned at Court.

The relationship between residual deformation and particle velocity of ground vibrations was determined upon regression analysis of the measurement data. The obtained best fit trendline with satisfactory correlation factor is linear, and its formulation is given by Eq. 8. Graphical interpretation of the relationship is shown in Fig. 6.

$$\left(\frac{\Delta s}{s}\right) = f(v_{50}) = av_{50} - b = 0{,}0051v_{50} - 0{,}003, \quad (mm/mm) \qquad (8)$$

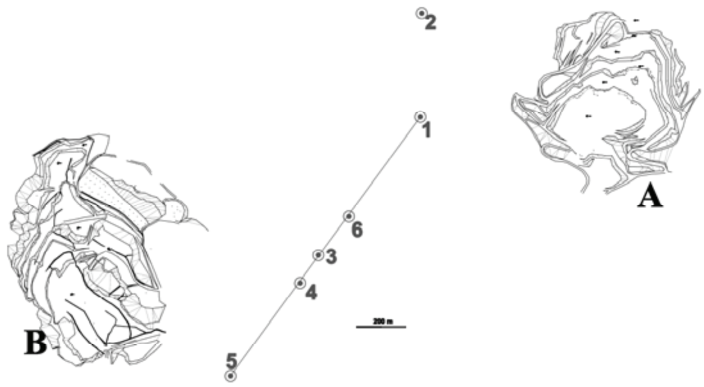


*Fig. 5. The position of the quarries Cokoce (A) and Tresnja (B) with the location of measuring points.*

Limit, minimal particle velocity of ground vibrations that can trigger crack response was calculated from the condition that relative deformation is equal to zero as:

$$\left(\frac{\Delta s}{s}\right) = 0 \Rightarrow v_{\lim} = \frac{b}{a} = \frac{0{,}003}{0{,}0051} = 0{,}6 \text{ , } mm/s \tag{9}$$

*Tab. 1. Results of the crack response monitoring.*

| № | Date (dd.mm.yyyy) | d (m) | $s_0$ (mm) | Δs (mm) | (Δs/s) (mm/mm) | v (mm/s) | location |
|---|---|---|---|---|---|---|---|
| **1** | 27.07.2006. | 1 929 | 2.0 | 0.0020 | 0.00100 | 0.73 | 5 |
| **2** | 27.07.2006. | 1 489 | 8.0 | 0.0160 | 0.00200 | 1.02 | 4 |
| **3** | 27.07.2006. | 1 368 | 10.0 | 0.0325 | 0.00325 | 1.14 | 3 |
| **4** | 06.11.2006. | 857 | 1.6 | 0.0925 | 0.00578 | 1.79 | 2 |
| **5** | 06.11.2006. | 817 | 1.4 | 0.0975 | 0.00696 | 1.91 | 1 |
| **6** | 26.12.2006. | 1 214 | 1.7 | 0.0225 | 0.00132 | 0.98 | 6 |
| **7** | 27.07.2006. | 825 | 1.4 | 0.0155 | 0.01107 | 2.14 | 2 |
| **8** | 27.07.2006. | 783 | 1.0 | 0.0062 | 0.00620 | 2.35 | 1 |
| **9** | 06.11.2006. | 1 518 | 5.0 | 0.0182 | 0.00364 | 1.42 | 4 |
| **10** | 26.12.2006. | 805 | 1.4 | 0.0027 | 0.00193 | 0.95 | 2 |

This low value of particle velocity is a result of overall general condition of the observed structures, suffering from severe structural damage. It also indicates higher sensitivity to blast induced vibrations of structural damage compared to cosmetic damage.

With defined propagation law and residual deformation over particle velocity of ground vibration relationship it was possible, in the third step, to perform the analysis of crack development "in reverse" as described previously.

The blasting data for the analysis were collected from the blasting logs of both Cokoce and Tresnja quarries and covered the period from 1969-2006. Total number of blasts carried out during this period was 1141.

The blasting data and coordinates of the structure and blasting series were imported into MS Excel worksheet. In order to step up the analysis, functions were inserted into worksheet to calculate distances, ground vibration velocities, relative and absolute values of residual deformations and finally resulting crack width. Due to large amount of blasting data tabular presentation of entire procedure of the analysis was not possible but the results are shown as graph presented in Fig. 7.

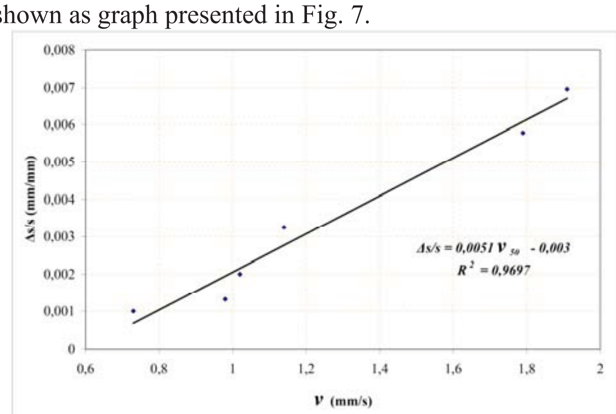


*Fig. 6. The relationship between residual deformation and particle velocity of ground vibrations.*

The analysis showed that a 76 μm wide crack, a hairline crack appeared in March of 1989 (Fig.7). This result confirmed the assumption that the cracks on the observed structure appeared in the late 1980's and hence validated the used methodology.

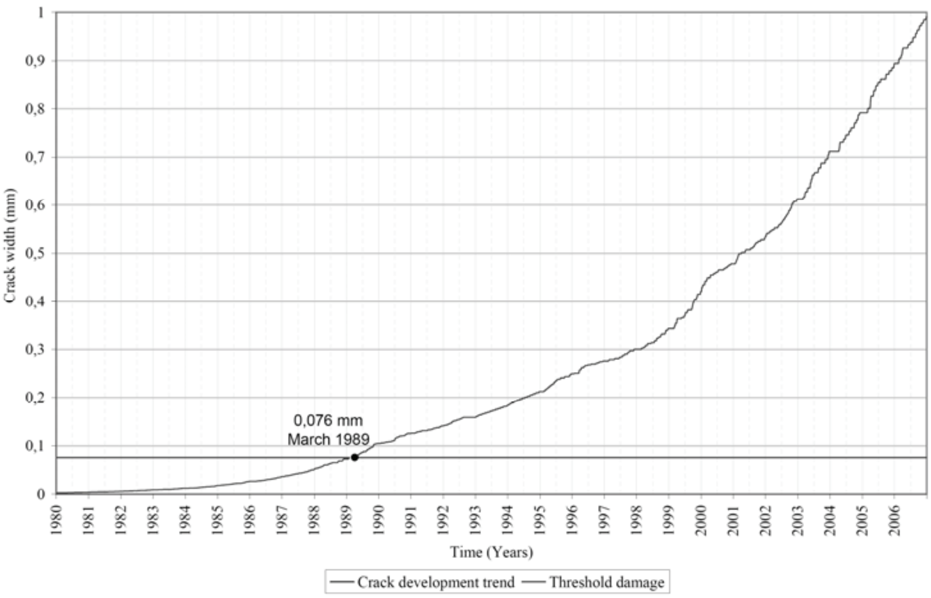


*Fig. 7. Graphical presentation of the results of the methodology validation analysis.*

**Example on percentage calculation**

The illustration of the use of the methodology is given on a practical example. The Municipality Court in Paracin, Serbia, gave the order to determine the percentage of blast induced vibrations in the damage to the specific structure for the period of three years before the date of the owner's complaint. The complaint was filed on December 26$^{th}$ 2001 so the start of the period in question was on December 26$^{th}$ 1998. The structure is located at the minimum distances of 1302 m from the Cokoce quarry and 817 m from Tresnja quarry. The structure is one story masonry residential building. The exact year of construction was not known but the structure was constructed in the late 1970's.

The inspection of the structure revealed several cracks, with the width ranging from 0.8–2 mm. The starting point of the analysis "in reverse" was the date of the last blasting operation, December 26$^{th}$ 2006.

Know data on blasting and known distances from the blasting series were used to calculate the ground vibrations particle velocities in accordance with Eq. 7. Then, according to Eqs. 8 and 9, relative residual deformations and corresponding relative crack widths were calculated, which made it possible to plot a graph of crack width over time (the crack development trend) shown in Fig. 8.

The date of July 7$^{th}$ 2003 displayed in the graph corresponds to the date when blasting operations on Tresnja quarry were stopped and extraction of the marlstone continued with hydraulic excavators. At that point the crack width was 0.8725 mm, according to the calculations. At the date of complaint the crack width was 0.8158 mm and three years earlier the crack width was 0.7054 mm.

In accordance to Eq.3-3, percentage of blast induced vibrations in damage to structure in the observed period was

$$P_P = (1 - \frac{0,7054}{0,8158}) \cdot 100 = 13,5\text{, } (\%)$$

This means that, if other non-blasting influences did not exist, the crack width would, under isolated influence of blast induced vibrations, increase for 13,5 % of its initial width. Hence, the blaster should be held liable for 13,5 % of damage to structure over the observed period.

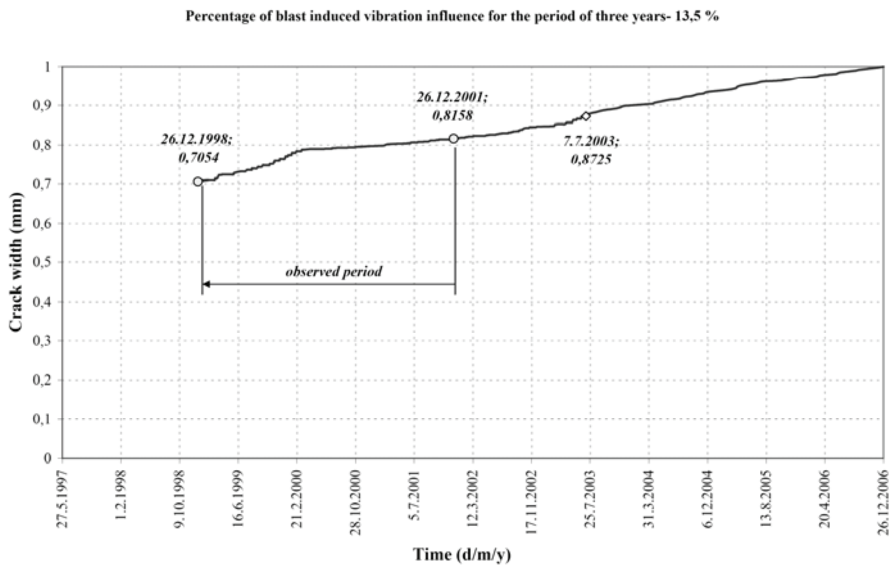


*Fig. 8. The crack development trend for calculation of the percentage of blast induced ground vibrations in damage to structure for the observed period.*

## Conclusion

The traditional approach to the problem of blast induced vibration caused damage to structures recognizes and considers only the moment of appearance of a new damage to structure. The crack response approach, on the other hand, often neglects the influence of blast induced vibrations justifying that action with considerably larger influence of non-blasting related factors. In both cases, the final decision on the liability for damage depends on the subjective opinion of expert witness that could in some cases be harmful for either homeowner or blaster. If, according to traditional approach, expert witness proves the causal connection between blasting operations and damage to structure, the blaster would be held liable for total amount of damage regardless of the true cause of the damage. On the other hand, if according to crack response monitoring approach expert witness neglects minor blasting related influence the homeowner will be harmed because the blasting operations maybe boosted the crack growth.

The methodology presented here gives the procedure for objective determination of causality between blasting operations and damage to structures and gives procedure for precise calculation of percentage of blast induced vibrations in total damage. According to this methodology, blasters liability exists only if causality is firmly proven, and the liability is equal to the percentage of blasting factors in total damage to structure. In other words, if other factors have influenced the appearance and development of the crack the blaster will not be liable for total damage but only for a small part.

Vice versa, the homeowner, only if causality is proven, can be entitled compensation for the part of damage caused by blast induced ground vibrations regardless of how small this influence can be because the fact is that this influence exists.

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