Vibrations induced by blasting in rock: a numerical approach

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Summary

Blasting is a powerful excavation method in terms of both production efficiency and economical costs, but its high environmental impact due to noise, vibrations, and potential damage to surrounding structures may limit its extensive application. The use of explosives as excavation tool is usually ruled by the national codes of practice, in which tolerable limits for induced motion are given for the different structure classes. In order to predict the vibrations induced in the ground at a given distance from the blast centre, attenuation laws, derived from either in situ measurements or analytical solutions of simple elastic wave propagation problems, are adopted. As the attenuation laws usually refer to homogeneous continua, they may not be adequate as a predictive tool for complex geological sites. In such cases, numerical analysis provides a valuable alternative, as the whole propagation history of stress waves could be simulated in principle, irrespective of the geological complexity of the specific site. To describe the progressive effects of underground blasting on the surrounding site, a finite element approach is presented. The explosion energy is translated in a time history of pressure at the boundary of the blast hole. Cracking of the nearby rock mass is modelled according to a cohesive crack model, while elastic behaviour is assumed for the non-cracked rock mass and soil deposits. Propagation of stress waves from the blast hole is simulated by a time domain 3-D finite element analysis, which is able to provide the time history of all the relevant quantities describing the motion at any given distance. The numerical results can be post processed in order to derive attenuation laws for the most relevant quantities to which the codes of practice usually refer to, i.e., peak particle velocity and principal frequency of the vibration. The model is energy-conserving, thus the energy supplied by the explosive is correctly partitioned into fracture energy of the rock mass close to the blast hole, elastic energy providing the stress wave propagation and kinetic energy of the fragmented rock blocks. Numerical simulations of two literature case-histories are presented, and the numerical results are compared to the available experimental data. Experimental peak particle velocity could be captured remarkably. Principal frequencies for the rock mass could be reproduced as well. In layered sites, the ratio between the stiffness of the different media where stress waves propagate seems to play a key role in the determination of principal frequencies, while less influence is observed on peak particle velocities.

Keywords: blasting, dynamic wave propagation, induced vibrations, finite element analysis, cohesive fracture.

Introduction

The use of explosives is still a powerful excavation method, thanks to flexibility, robustness and high production rates achievable [see, e.g., HUDSON, 1993]. Blasting technology has been refined to a great deal, although the practical methods have been optimised empirically. Many factors hindered a deep knowledge of all the dynamic effects produced by explosive blasts, such as the debate on whether the resulting rock breakage is due to the large amplitude stress waves travelling through the medium, or to the very high gas pressures. Several research works published over the past 40 years supported either one or the other hypothesis [FOURNEY, 1993]. For very high charges detonated nearby an exposed plane free surface, BHANDARI [1979] suggested that the most likely fragmentation mechanism could be the spalling induced by the reflection of the compression wave generated by the explosion in the form of a tensile wave. On the contrary, gas penetration into already opened fractures around blast holes seems to be more a symptom than a cause of fracture [MC KENZIE, 1993].

Whatever fracture mechanism prevailed, part of the explosive energy is transmitted outward in the far field through elastic stress waves. Vibrations are induced inside the rock mass and on the free boundaries, where surface waves and reflection may amplify the incoming body stress wave. Commonly, peak particle velocity, i.e. the time derivative of the vibration displacement, is adopted as a convenient measure of disturbance level, although other alternative parameters may describe the effects of blast induced vibration on structures. DOWDING [1993] provides a comprehensive treatise of all the technical aspects concerning the interaction between blast vibrations and structures.
The use of explosives is usually ruled by the national codes of practice, in which tolerable limits for induced motion are given for the different structure classes (e.g., Anderson, 1993). As a consequence of the increasing environmental constraints on tolerable levels of disturbance induced by blasting operations upon nearby residents, blast is designed so as to achieve levels of ground vibration and overpressure disturbance as close as possible to the permissible levels. In order to predict the vibrations induced in the ground at a given distance from the blast centre, attenuation laws, coming from either in situ measurements or analytical solutions of simple elastic wave propagation problems, are adopted. As the attenuation laws usually refer to homogeneous continua, they may not be adequate as a predictive tool for complex geological sites.

Numerical analysis provides a valuable alternative in these cases, as the whole propagation history of stress waves could be simulated in principle, irrespective of the geological complexity of the specific site. Grady and Kipp [1980] and Ang and Valliappan [1988] were among the first researchers tackling wave propagation arising from detonation of explosives via finite element analysis. Recently, a few 3-D numerical analyses were proposed, including continuum-based [Liu et al., 1997; Hao et al., 1998; Jia et al., 1998; Hao et al., 2002; Torano and Rodriguez, 2003; Wu and Hao, 2004; Torano et al., 2006] and discontinuum-based approaches [Hart, 1993; Chen and Zhao, 1998].

To describe the effects of underground blasting on the surrounding site, a finite element approach is presented here. The explosion energy is translated in a time history of pressure at the boundary of the blast hole, based on Friedlander’s equation. Cracking of the inner portion of the rock mass is modelled according to the cohesive crack model proposed by Ortiz and Pandolfi [Pandolfi and Ortiz, 1998; 2002; Ortiz and Pandolfi, 1999; 2004], while elastic behaviour is assumed for the non-cracked rock mass and soil deposits. Propagation of stress waves from the blast hole is simulated by a 3-D finite element analysis, which is able to provide the time history of all the relevant quantities describing the motion at any given distance. Numerical simulation of two published case histories is presented, allowing for capabilities and limitations of the proposed numerical model to be discussed.

2. Blasting and blast wave

The energy released by a conventional chemical explosive in air produces, with respect to atmospheric pressure, an overpressure wave, $P$, called blast wave. The blast wave profile in air evolves in the direction of travel (Fig. 1a), due to the different velocities of the overpressure components [Kinney, 1962]. Irrespective of the actual original overpressure shape, at a reasonable distance from the centre of the explosion, blast waves can be described by a sharp overpressure front followed by a decaying tail, eventually dropping to zero or to negative values (Fig. 1b). At a fixed distance, $r$, from the explosion centre, Friedlander’s equation may be adopted to describe the homothetic decaying pressure $p(t)$ in time:

$$p(t) = p_0 P(1 - \tau) \exp(-b\tau)$$

where $p_0$ is the atmospheric pressure, $P$ is the peak overpressure due to blasting, $t_r$ is the arrival time at distance $r$, and $T_d$ is the duration of the blast induced positive overpressure. The parameter $b$ governs the decaying velocity of the overpressure.

Peak pressure, arrival time and duration of blast overpressure at distance $r$, following the actual explosion, may be calculated by scaling laws on the ba-
ysis of experimental data relative to a standard explosion of 1 ton TNT. Assuming that the explosion is characterised by the same temperature, reference pressure $p_0$ and sound velocity as the standard conditions, the relevant parameters may be calculated through a scaling parameter, $\lambda$, called yield, given by the cube root of the ratio between the effective explosion energy, $W$, and the standard reference explosion energy of 1 t of TNT, $W_{ref}$:

$$\lambda = \left( \frac{W}{W_{ref}} \right)^{1/3}$$  \hfill (2)

Distances may be simply scaled by the relation

$$r_{ref} = \frac{r}{\lambda}$$ \hfill (3)

and the relevant overpressure parameters by the relations

$$P = p_0 P_{ref} \quad t_r = \lambda t_{ref} \quad T_d = \lambda T_{dref}$$  \hfill (4)

The non dimensional parameter $b$ is an empirical function of the scaled distance,

$$b = b \left( \frac{r}{\lambda} \right)$$ \hfill (5)

which can be derived from the tabular values reported by Kinney [1962]:

The scaling laws indicate that a real explosion, characterised by a yield $\lambda$, generates the same overpressure as the one generated by the standard explosion at a distance $r = \lambda r_{ref}$, with arrival time $t_r = \lambda t_{ref}$ and duration $T_d = \lambda T_{dref}$.

This approach may be adopted to describe the effects of blasting in rocks, provided an air gap between the explosive and the surrounding rock mass allows for treating the pressure wave in the blast hole similarly to a blast wave in air.

### 3. Numerical Modelling

To provide a description of the effects induced by blasting in the surrounding environment, a finite element numerical model is proposed, based on previous work by Ortiz and Pandolfi [Pandolfi and Ortiz 1998, 2002; Ortiz and Pandolfi 1999, 2004], which will be briefly summarised in the following. A detailed description of the relevant algorithms can be found in the original papers.

#### 3.1. Constitutive Laws

A non-linear kinematic approach is adopted in the formulation of the model, due to the necessity of describing processes involving fracture and possible motion of the fractured rock blocks.

The intact rock mass, in which blast holes are drilled, may be modelled as an elastic brittle homogeneous medium. This choice allows for capturing the most relevant features governing the overall deformational response to a blast load, i.e., intense fracturing of the inner zone near the blast hole and propagation of elastic waves in the outer region. For the sake of simplicity, in the analyses presented here material damping in the intact zone is neglected.

An isotropic, non linear, hyper-elastic model is adopted [Ogden, 1984; Ortiz and Pandolfi, 2004] for the rock mass. The model is characterised by the initial tangent values of the two independent elastic constants. Non linearity of the elastic law is a direct consequence of the finite kinematic approach.

Failure of the rock mass is described by crack formation and propagation and modelled by means of the cohesive element proposed by Ortiz and Pandolfi [1999]. The cohesive model is based on the definition of an effective fracture opening displacement, $\delta$, work-conjugate to an effective fracture, $t$. The two quantities are defined based on the respective vectors, $\delta$ and $t$ (Fig. 2), assigning different weights to their normal and tangential components:

$$\delta = \sqrt{\beta^2 \delta_n^2 + \delta_t^2} \quad \delta_n = \delta \cdot n \quad \delta_t = |\delta - \delta_n n|$$ \hfill (6)

![Fig. 2 – Relative displacements and traction over the cohesive surface (CS).](image)

![Fig. 2 – Spostamenti relativi e trazioni sulla superficie coesiva (CS).](image)
where $\beta$ is defined by means of the tensile strength, $\sigma_t$, and the uniaxial compressive strength, $\sigma_c$, of the rock:

$$\beta = \frac{\sigma_t}{\frac{2 \sigma_c}{\sigma_t}}$$  \hspace{1cm} (8)$$

A simple model for cohesive fracture may be obtained by assuming a free energy density dependent only on the effective opening displacement. The uniaxial effective irreversible cohesive law implemented in the numerical code is visualised in Figure 3. If the maximum attained effective opening displacement, $q$, is chosen as internal variable, a load path implying fracture propagation is described by the conditions $\delta = q$ and $\delta > 0$. The critical energy release rate, $G_c$, is represented by the area bounded by the linear law depicted in Figure 2(b), i.e.:

$$G_c = \frac{\Phi(q)}{\delta}.$$  \hspace{1cm} (9)$$

The ratio between the energy dissipated in the current fracture opening, $\Phi(q)$, and the critical energy release rate defines an a posteriori damage measure, $D$, in the range between 0 and 1:

$$D = \frac{\Phi(q)}{G_c} \equiv \frac{\Phi(q)}{G_t} \geq 0.$$  \hspace{1cm} (10)$$

As depicted in Figure 3(a), irreversibility characterises an unloading process. Upon closure, the two surfaces of the fracture are subjected to a unilateral contact condition with friction. Contact and friction are defined independently from the cohesive law and implemented by a penalty approach [Vu et al., 2002].

Besides the two parameters necessary to calibrate the isotropic elastic law (shear modulus, $G$, and Poisson ratio, $\nu$, are adopted here), three independent constants are needed to fully describe the cohesive law. A common choice is represented by the tensile strength of the rock, $\sigma_t$, its uniaxial compressive strength, $\sigma_c$, and the critical energy release rate $G_c$.

3.2. Boundary value problem formulation and discretisation

The numerical code adopted for the analyses allows for a finite kinematics three-dimensional analysis of dynamic boundary value problems in the time domain. Explicit integration in time is performed by means of Newmark’s algorithm, by adopting $\beta_N = 0$ e $\alpha_N = 1$ [Hughes, 2000]. This choice implies algorithmic damping, which partially compensates the lack of material damping in the constitutive law.

The finite element model is discretised into an initially coherent mesh of 10-node solid tetrahedral elements. Fractures are allowed to propagate only between the faces of the original elements, when the effective traction reaches the critical value defined by the cohesive model. Fracture is then simulated via the local introduction of 12-node cohesive ele-

Fig. 3 – Graphical representation of the irreversible cohesive law.
Fig. 3 – Rappresentazione grafica della legge coesiva irreversibile.
ments (Fig. 4a) in between the faces of the original element mesh (Fig. 4b), through an auto-adaptive remeshing procedure [PANDOLFI and ORTIZ, 1998; 2002]. The cohesive law governs the subsequent effective opening process along the new opened surface.

For the process zone of the cohesive fracture to be correctly modelled without introducing spurious scale effects, the characteristic size of the finite elements in the numerical mesh should be smaller than the characteristic length, $l_c$, introduced by the cohesive approach:

$$l_c = \frac{E\sigma_c}{\sigma_f^2}$$  \hspace{1cm} (11)

where $E$ is the Young modulus of the intact rock mass. Besides, the integration time step should not be larger than the critical time step for the stability of the numerical algorithm, $t_z$:

$$t_z = \frac{h}{c} = \frac{h}{\sqrt{\rho E}}$$  \hspace{1cm} (12)

where $h$ is the minimum mesh size, $\rho$ is the rock mass density and $c$ is the rock longitudinal wave speed [COURANT et al., 1967].

3.3. Boundary conditions

Different boundary conditions are imposed on the different portions of the boundaries of the discrete domain.

Along the blast hole boundary, exposed to the explosion, a time history of overpressure is imposed, based on the considerations summarised in Section 2. Given the explosion yield, a pressure time history is calculated through the scaling law as a function of the distance between the centre of the explosion and the surface element considered. The resulting overpressure time history at any given node of the blast hole surface follows the shape depicted in Figure 5.

On the free boundaries of the domain, a zero traction condition is imposed throughout the analysis. Fictitious outer boundaries, closing the numerical domain, should in principle work as perfectly absorbing boundaries. Among the different suggestions in the literature (e.g. ENquist and MAjDA, 1977; CLAYTON and ENquist, 1977; KAUSEL, 1988; JIAO et al., 2007), in order to limit artificial reflection of the stress waves, absorbing boundaries based on the proposal by CLAYTON and ENquist [1977] were
4. Numerical simulations

The numerical model summarised in the previous paragraph was adopted previously by Ortiz and Pandolfi [2004] to estimate damage induced on concrete buried structures by a blast wave due to an explosion over the soil surface. The present work is aimed at verifying whether the model was able to simulate rock fragmentation due to blasting and induced vibrations in the outward rock mass and soil deposits. To this aim, two case-histories are considered and the results of the numerical simulations are compared with some published experimental data. The first case shows that the amount of energy dissipated for rock fracturing is well caught by the numerical model. The second example focuses on the characteristics of the elastic waves propagation in the outer zone, and on the influence of the geometry and the material properties of the rock-soil deposit on the overall vibration process.

4.1. Hydropower plant service tunnel excavation [BERTA, 1985]

The first case simulates one advancing step of the excavation of a service tunnel for a hydropower plant in a granite rock mass [BERTA, 1985]. A schematic view of the tunnel geometry and of the excavation stages is given in Figure 6.

The tunnel section is excavated in three main steps. In the first one (bounded by the inner circle in Fig. 6) the central section is opened by first manually drilling the central relief hole, and then by controlled blasting in the inner blast holes. The second stage is the proper production stage, where a relevant part of the section (in between the inner and the outer circle) is excavated by controlled blasting, and most of the total blast energy is consumed. Finally, in the third stage, sequential blasting of multiple small explosions are used to refine the external profile of the tunnel.

The geometry and the finite element mesh adopted in the numerical analysis are described in Figure 7 (a-d). It is worth noting that only the second stage of excavation was simulated numerically.

To this aim, a single blasting episode was considered, in which the total amount of explosive effectively used in the practice was concentrated at the centre of the blast hole, irrespective of the real blasting sequence. The simplifying assumption is acceptable, as the aim of the first simulation was to verify the global energy balance of the dynamic process. The numerical mesh is built with 13024 tetrahedral elements and 19585 nodes. Mesh refinement was provided close to the blast hole in order to satisfy the requirement on the characteristic length (Eq. 11), although the typical size of the elements was increased outward from the front face.

The explosion yield was calculated on the basis of the design parameters of the production stage. The volume of rock to be excavated in the analysed stage was equal to 12.6 m³. Chosen the explosive, the equivalent mass of TNT to obtain the desired volume of excavation during the production stage was estimated to be \( M_{TNT}^{eq} = 19 \) kg, whose equivalent explosive energy is \( W = 79 \text{ MJ} \), hence the yield of the explosion is \( \lambda = 0.267 \). The rock mass parameters were chosen on the basis of literature data and are summarised in Table I.

Relevant results of the numerical analysis are shown in Figures 8-9. The final damage level is represented in Figure 8(a) on the cross section of the advancing tunnel. The numerical simulation correctly predicts the extension of cracking in the inner tunnel section, which was the aim of the production stage blasting. The outer part of the excavated section is yet damaged, but to a smaller level. It is worth noting that, in practical applications, a third profiling stage is expected.

Figure 8(b) shows that the advancement depth is well caught, although a slight mesh dependency can be appreciated from the fractured zone being bounded to a significant extent by the surfaces of
the smaller elements. This mesh dependency could be eliminated by reducing the size of the elements to values smaller than the characteristic length in a bigger volume on the front of the tunnel. Nevertheless, in this analysis the focus was on the global energy balance rather than on following the actual fragmentation profile.

Figure 9 shows how the total blast energy is partitioned during fragmentation and stress wave propagation. The dots represent the total external work provided by the pressure wave on the boundary of the blast hole. Total energy is almost conserved by the numerical model, although the effects of computational damping may be appreciated starting from about 15 ms after the explosion. The analysis predicts that most of the energy provided to the system is transformed into kinetic energy during the whole time interval analysed. Fracture process is
almost completed after the first 10 ms, and only a small part of the total energy is transmitted to the outer rock mass as elastic propagating waves. Figure 9(b) shows the numerical prediction of the fracture area created by fragmentation of the inner rock mass. The ratio between the total volume of the fragmented rock and the fractured area may be used as a measure of the dimension of the blocks isolated by the excavation procedure. This information proves to be a relevant parameter in the design, for the removal, transport and storage of the excavated rock blocks.

Figure 10 shows how stress waves are transmitted to the surface of the rock mass, in correspondence of the three sampling points (forward, central and backward) indicated in Figure 7(b). The figure reports the vertical velocity component as a function of time in the first 10 ms, before reflected waves from the artificial lateral boundaries may interfere with the incoming stress waves. The different arrival times of stress waves in the different sampling nodes can be clearly detected as a function of the distance from the blast hole. Stress waves propagating from the blast hole through the fractured zone reach first the central control point, located on the vertical to the blast hole centre, where the ground vertical velocity is a result of the longitudinal stress waves mainly. A longer time lapse is needed for the stress waves to reach the rock mass surface forward and backward.
backward sampling nodes. In these points, the initial amplitude of the local velocity is smaller, due to geometric damping. The influence of surface stress waves on these sampling points may be appreciated by observing increasing amplitudes and lowering frequencies with time. It is worth noting, anyway, that computational damping may contribute to filter the highest frequencies with distance from the blast pressurised boundary.

Effectiveness of the dashpots introduced to damp the outgoing waves may be evaluated by the comparison of the results of the previous analysis (Fig. 11a) with the results of an identical analysis without absorbing boundaries (Fig. 11b). The three velocity components at the central control node are depicted for the analysis time interval of 40 ms. Comparison between the time evolution of the three velocity components demonstrates that the dashpots allow for reducing the effect of spurious reflected waves although they are not able to eliminate them completely. Differences between damped and non-damped analysis may be fully appreciated after about 15 ms, while initially (until about 10 to 15 ms), before reflected waves begin to become significant, the numerical velocities are almost comparable.

The results presented in this section aim at demonstrating the potential capabilities of the numerical model in the description of the different processes involved in a rock mass when blasting is adopted as excavation technology. Energy is almost conserved, fracture area may be calculated, and in principle, the acceleration, velocity and displacement histories can all be described at any distance from the blast centre, at least before reflected waves introduce spurious effects in the result of the analysis. Usually, due to the characteristics of the stress waves induced by blasting, which is an impulsive load, the most relevant effects on ground acceleration and velocity are observed upon stress wave arrival, before spurious reflected waves may affect the numerical results.

Reliability and limitations of the proposed model may be much better appreciated with reference to the following literature example, where part of the numerical results could be compared to experimental measurements.

4.2. Small-scale field blast test in a layered rock soil site [Wu et al., 2003]

To investigate stress wave propagation from detonation of explosives in a quarry, a series of field blast test was designed and performed in a granite rock mass [Wu, et al., 1998; Hao et al., 2001; Wu et al., 2003]. Before the blast test, a 1.5 m thick compacted soil layer was backfilled onto the flat surface...
of the test quarry site (Fig. 12a). The site was instrumented with accelerometers, with a sampling rate of 500 kHz, arranged inside the rock mass, on the rock surface (section A-A in Fig. 12c), and on the soil surface (Fig. 12d). The accelerations following coupled and decoupled blast tests with different charge weights were recorded.

A careful geological and geotechnical investigation had been carried out before the blast tests, giving a rather detailed characterisation of the rock mass and of the compacted soil layer. All model parameters could be determined from the investigation data. The sole critical energy release rate of unweathered granite was calibrated, based on literature data and on a preliminary numerical simulation of the fracture process in a granite disk [Kutter and Fairhust, 1971]. Although an elastic-plastic model could be equally adopted for the compacted upper layer [Ortiz and Pandolfi, 2004], the soil was assumed to behave elastically, due to the small vibration level expected. The rock and the compacted fill parameters adopted in the numerical analysis are summarised in Tables II-III.

The charge hole, of 11 m total depth, was located at the centre of the test field. The explosive (PETN) was buried at a depth of 8.5 m from the rock mass surface, in a chamber 0.8 m wide. Different blast tests were performed with different charge weights, ranging from 2.5 to 50 TNT equivalent kg. The 12.5 TNT equivalent kg test was simulated numerically, as most of the published data refer to this case.

Different views of the initial solid mesh adopted in the analysis are provided in the Figures 12(b), 13-14. The finite element model has 7702 elements and 11700 nodes. As in the previous example, the boundary of the charge hole was loaded with a scaled pressure time history, calculated from the actual explosion yield $\lambda = 0.232$. Absorbing dashpots
were located at the fictitious outer boundary of the domain, and no tractions were imposed elsewhere.

Figures 15-17 show time histories of horizontal and vertical velocity components at three control points, at comparable distances from the charge hole, inside the rock mass, at the rock mass surface and at the upper soil surface, respectively. Comparison between the three figures confirms that the blast-induced stress waves differ substantially at the three locations. In the rock mass and at the rock surface the vertical and horizontal components are of the same magnitude. The largest amplitudes are observed at the rock surface (Fig. 16), due to wave reflection and surface wave generation. On the contrary, due to refraction at the rock-soil interface, on the upper soil surface the vertical component is definitely higher than the horizontal one. Although the compacted fill soil is only 1.5 m thick, the attenuation properties of the backfill are rather evident when the peak velocities on the soil surface are compared to those of the rock surface.

When environmental effects are of concern, a convenient way to describe the effect of vibrations induced by blasting is to visualise the attenuation of stress waves with the distance from the blast centre, choosing suitable parameters for the description of the stress wave. The most relevant parameter to infer potential damage on constructions is the peak particle velocity, i.e., the maximum value of the displacement rate at a given location. Peak particle velocity is therefore adopted as a convenient measure of the strain level induced on the structure by the striking wave.

Peak particle velocity can be defined in different ways, as the different velocity components are not in phase, in general. Here, it is defined as the modulus of the vector built with the independent peak values of the horizontal and vertical components. Although this definition has no physical counter-

Tab. II – Parameters adopted in the numerical analysis of small blast test [Wu et al. 2003]: rock mass.

<table>
<thead>
<tr>
<th>Rock mass density</th>
<th>Young’s modulus</th>
<th>Poisson’s ratio</th>
<th>Tensile strength</th>
<th>Uniaxial compressive strength</th>
<th>Critical energy release rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho ) ( \text{kg/m}^3 )</td>
<td>( E ) ( \text{GPa} )</td>
<td>( \nu )</td>
<td>( \sigma_t ) ( \text{MPa} )</td>
<td>( \sigma_c ) ( \text{MPa} )</td>
<td>( G_c ) ( \text{N mm}^{-2} )</td>
</tr>
<tr>
<td>2610</td>
<td>74 (or 25)</td>
<td>0.16</td>
<td>16.1</td>
<td>186</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Tab. III – Parameters adopted in the numerical analysis of small blast test [Wu et al. 2003]: compacted soil fill.

<table>
<thead>
<tr>
<th>Soil mass density</th>
<th>Young’s modulus</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho ) ( \text{kg/m}^3 )</td>
<td>( E ) ( \text{GPa} )</td>
<td>( \nu )</td>
</tr>
<tr>
<td>2000</td>
<td>1.5</td>
<td>0.3</td>
</tr>
</tbody>
</table>
part and may overestimate the real peak value of ground motion (i.e. the modulus of the instantaneous velocity), the choice allows for comparison with the published experimental data.

Figures 18-19 compare experimental peak particle velocities with the results of the numerical simulation. On the horizontal axis, the scaled distance, defined as the ratio between the actual distance of the point under consideration from the blast centre, $r$ in m, and the cube root of the equivalent TNT weight, $W$ in kg, is reported. This definition does not seem totally appropriate, as the scaled distance

Fig. 15 – Velocity time history inside the rock mass, at a distance of 10.0 m from the blast centre: (a) horizontal component and (b) vertical component.

Fig. 15 – Storia temporale di velocità all’interno dell’ammasso roccioso, a distanza di 10.0 m dal centro dell’esplosione: (a) componente orizzontale e (b) componente verticale.

Fig. 16 – Velocity time history at the rock mass surface, 13.1 m from the blast centre: (a) horizontal component and (b) vertical component.

Fig. 16 – Storia temporale di velocità sulla superficie dell’ammasso roccioso, a distanza di 13.1 m dal centro dell’esplosione: (a) componente orizzontale e (b) componente verticale.

Fig. 17 – Velocity time history at the soil surface, 14.1 m far from the blast centre: (a) horizontal component and (b) vertical component.

Fig. 17 – Storia temporale di velocità sulla superficie del terreno, a distanza di 14.1 m dal centro dell’esplosione: (a) componente orizzontale e (b) componente verticale.
Fig. 18 – Peak particle velocity at the rock surface: (a) comparison between numerical results and experimental data and (b) comparison between respective linear fits.

Fig. 18 – Velocità di picco sulla superficie dell’ammasso: (a) confronto fra i risultati numerici e i dati sperimentali e (b) confronto fra le rispettive interpolazioni lineari.

Fig. 19 – Comparison between numerical and experimental peak particle velocity at the upper soil surface.

Fig. 19 – Confronto fra le velocità di picco numerica e sperimentale sulla superficie del rilevato in terra.
turns out to be a dimensional parameter, but it is widely adopted as a measure of the actual distance scaled by the explosion energy (see, e.g., Dowding, 1996), and was adopted here to compare the numerical results with the experimental data [Wu et al., 1998; Wu et al., 2003].

Figure 18(a) demonstrates that the calculated peak particle velocities at the rock surface fall entirely inside the range of experimental data, characterised by quite a large scatter. Both experimental data and numerical results show that attenuation is non-linear in the log-log plane. The peak particle velocity does not seem to decrease substantially, for scaled distances lower than 5, while an almost linear decrease is observed at larger distances. In Figure 18(b), replicating a common practice in the design, linear fits of both experimental and numerical data are drawn over the whole range of analysed scaled distances. Comparison between the two linear fits shows that a linearization over the entire scaled distance range is somehow misleading, as the attenuation laws seem to differ much more than the direct comparison presented in Figure 18(b) shows.

In Figure 19 the numerical results for the peak particle velocity at the upper surface of the compacted soil fill are compared to experimental data. Again, the numerical results fall in between the range of measured data for scaled distance lower than 15, while for larger distances the numerical analysis seems to underestimate the recorded peak particle velocity.

Figure 20(a) presents the numerical peak particle velocity of the rock mass. Although accelerometers were actually placed inside the rock mass, in the reference papers the experimental peak particle velocities of the rock mass were not reported. The numerical results confirm that a non linear attenuation law is expected inside the rock mass too, as damage induced by blasting influences the overall stiffness of the rock mass nearby the blast hole.

To analyse the sensitivity of the results of the numerical model on the assumed rock mass stiffness, a second analysis was run, assigning a Young’s modulus for the rock mass three times lower than that of the unweathered granite. The comparison between the results of the two numerical analyses shows that the rock Young’s modulus has little influence on the peak particle velocity of the rock mass itself, except in the proximity of the explosion centre (Fig. 20(b)). A slightly higher influence may be appreciated at the soil surface, although the two solutions tend to converge as the distance from the blast hole increases.

The experimental data collected inside the rock mass were elaborated by Wu et al. [1998], to give the attenuation law of the principal frequency of the acceleration. Experimental data, presented in Figure 21, refer to three tests in which equivalent TNT charges of 10, 20 and 40 kg were exploded in sequence. The results for the different explosive charges were presented separately as a function of the actual distance, r, of the accelerometer from the charge hole. The authors observed that the principal frequencies of acceleration decrease dramatically in the near-field of detonation, and slowly for radial distances exceeding 10 m. This implies that the high frequency components are damped out in the rock mass within a rather small zone surrounding the explosion centre.

The blast tests were designed in order not to induce significant damage in the rock surrounding the charge hole, so that multiple tests could be run at the same site [Wu et al., 2003]. Nevertheless, observing that smaller charge weights corresponded to higher principal frequencies, the authors themselves attributed the experimental evidence to the possibility of damage around the blast chamber, which attenuated the high-frequency components of the shock wave nearby the charge chamber.

The results of the two numerical analyses, with two different Young’s moduli, seem to corroborate this hypothesis. It is worth noting that the principal frequencies are well caught by the analysis, although the numerical attenuation is more gradual than the experimental one. The analysis with a reduced Young’s modulus, roughly simulating a weathered rock mass, matches very well the experimental data for the second and third explosions nearby the blast hole.

The extension of the damaged zone around the charge hole, derived from the numerical analysis, is represented in Figure 22. The result confirms that some damage is likely to have occurred around the blast hole, even for an equivalent TNT weight of 12.5 kg. The damaged zone has a limited extension, but it may justify the smaller dominant frequencies observed in the following experimental tests.

Finally, the principal frequencies of the acceleration at the backfilled soil surface calculated from the numerical analyses and from the experimental data are compared in Figure 23. In this case, Wu et al. [2003] chose to represent the experimental data by means of the mean value of the frequency in the range bounded by the maximum spectral peak and half of the maximum spectral peak. Hence, a direct comparison between the experimental results and the numerical values may be misleading to a certain extent. Nonetheless, it appears clearly that the principal frequencies calculated from the numerical results are definitely higher than those recorded experimentally. As the principal frequencies depend on the stiffness of the medium (see e.g. Fig. 21), the discrepancy may possibly come from overestimating the compacted soil fill Young’s modulus. Furthermore, decreasing the stiffness would increase...
Fig. 20 – Sensitivity of peak particle velocity to the rock stiffness (a) inside the rock mass and (b) at the soil surface.

Fig. 20 – Sensibilità della velocità di picco alla rigidezza della roccia (a) all’interno dell’ammasso e (b) sulla superficie del rilevato in terra.

Fig. 21 – Attenuation law for principal frequency of acceleration in the rock mass: comparison between experimental data and numerical results.

Fig. 21 – Legge di attenuazione per la frequenza principale dell’accelerazione nell’ammasso roccioso: confronto fra i dati sperimentali e i risultati numerici.
slightly the peak particle velocity, apparently underestimated in the present analysis, at least far from the blast hole (see Fig. 19).

5. Concluding remarks

The proposed finite element approach proved to be satisfactory to model the progressive effects of underground blasting on the surrounding site. The explosion energy was translated in a time history of normal pressure on the boundary of the blast hole. The pressure history, derived from theoretical consideration on the blast wave propagation in air, can be adopted in analysing the effects of blast in rock masses, when an air gap is present between the explosive and the blast hole surfaces.

Fragmentation of the rock mass around the blast hole can be accounted for with a suitable cohesive model, defined by three constitutive rock parameters. The numerical simulations performed confirmed that the percentage of explosive energy transmitted to the outer portion of the rock mass is relatively low. This justifies the adoption of an elastic constitutive law, characterised by the initial values of the elastic moduli, for the intact rock mass. Neglect-
ing material damping does not influence substantially the peak particle velocity of the rock mass, as damping of the highest frequencies is accounted for by introducing a convenient algorithmic damping.

In the numerical analyses performed, the dashpots, located at the fictitious outer boundary, were sufficient to avoid significant reflection of the outgoing waves as the comparison between numerical results and experimental data demonstrated.

The numerical approach provides useful information for practical applications. Thanks to the energy-conserving approach, partitioning of the explosive energy into fracture energy, stress wave propagation and kinetic energy can be estimated. The total fracture area, an important parameter in the design for transport and storage of the fractured rock volume, may be calculated. A full description of acceleration, velocity and displacement histories is provided at any location of the domain. The results may be post-processed to obtain spectral densities and principal frequencies of the relevant quantities.

Following a common approach, the numerical results were elaborated to give attenuation laws of peak particle velocity and principal frequency of the acceleration with scaled distance. The resulting attenuation laws are not necessarily linear in non homogeneous media, as wave reflection and refraction at the strata interfaces modify the dominant characteristics of the motion. Differently from empirical laws or simple analytic solutions, the numerical approach allows for tracking the stress waves characteristics in general non homogeneous domains, hence providing a valuable tool in the prediction of the environmental effects of blast waves in practical applications.

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References


Vibrazioni indotte da esplosivo in roccia: un approccio numerico

Sommario

Lo scavo mediante esplosivo è una metodologia versatile, efficiente ed economica. Tuttavia, la sua applicazione può essere fortemente limitata dal forte impatto ambientale associato al disturbo, alle vibrazioni e ai potenziali danni indotti sulle costruzioni circostanti. L’uso di esplosivo come tecnica di scavo è regolato dalle normative nazionali, che definiscono i limiti di tollerabilità delle vibrazioni trasmesse alle strutture circostanti, in funzione della tipologia di struttura stessa. Per stimare l’entità delle vibrazioni indotte nel terreno a una data distanza dal centro dell’esplosione, vengono usualmente impiegate leggi di attenuazione basate su misure effettuate in sito o su soluzioni analitiche semplificate di propagazione di onde. Le leggi di attenuazione sono riferite, in genere, a trasmissione in mezzi omogenei e, pertanto, possono non risultare del tutto adeguate come strumento predittivo in siti geologicamente complessi. In questi casi, l’analisi numerica può rappresentare una valida alternativa, poiché, in principio, consente di simulare l’intera storia di propagazione delle onde, qualunque sia la complessità del sito analizzato. In questo lavoro viene presentato un approccio agli elementi finiti per descrivere gli effetti provocati da un’esplosione sull’ammasso roccioso e sui terreni circostanti, mano a mano che si allontana dal suo centro. L’energia dell’esplosivo è tradotta in una storia temporale di pressione sul contorno esposto all’esplosione. Un modello di frattura coesiva consente di riprodurre la frammentazione della roccia in prossimità dei fori dove è posizionato l’esplosivo, mentre per l’ammasso roccioso non fratturato e per i terreni circostanti è assunto un comportamento elastico. La propagazione delle onde elastiche dal centro dell’esplosione è simulata mediante una analisi tridimensionale ad elementi finiti nel dominio del tempo, che fornisce tutte le variabili che caratterizzano il moto dell’esplosivo. L’energia viene così correttamente ripartita fra la quota dell’esplosivo e la quota rocciosa, che sembra che è possibile che risulti del tutto adeguata come strumento per calcolare la velocità di picco e la frequenza principale di vibrazione. Il modello numerico proposto è in grado di conservare l’energia. L’energia dell’esplosivo viene correttamente ripartita fra la quota del terreno a una data distanza dal centro dell’esplosione, l’energia di propagazione delle onde elastiche e l’energia cinetica dei frammenti di roccia. Vengono presentate le simulazioni di due casi di trasmissione, confrontando i risultati dell’analisi con i dati sperimentali disponibili. Le velocità di picco ottenute dalla simulazione risalgono al disturbo, alle vibrazioni e ai potenziali danni circostanti, che si aggirano attorno a 0.1 m/s e che possono risultare del tutto adeguate come strumento per calcolare la velocità di picco.

Parole chiave: esplosivi, propagazione dinamica di onde elastica, vibrazioni indotte, analisi ad elementi finite, frattura coesiva.