Shotcrete: Elements of a System – Bernard (ed)

A comparison of models for shotcrete in dynamically loaded rock tunnels

L. Ahmed & A. Ansell
Royal Institute of Technology, Stockholm, Sweden

ABSTRACT: During blasting in tunnels and mines, the shotcrete-rock interaction is influenced by propagating stress waves. Shotcrete support in hard rock tunnels is here studied through numerical analysis and comparisons with previous numerical results, measurements and observations in situ. The stress response in the shotcrete closest to the rock when exposed to P-waves striking perpendicularly to the shotcrete-rock interface is simulated. The first model tested is an elastic stress wave model, which is one-dimensional with the shotcrete assumed linearly elastic. The second is a structural dynamic model that consists of masses and spring elements. The third model is a finite element model implemented using the Abaqus/Explicit program. Two methods are used for the application of incident disturbing stress waves: as boundary conditions and as inertia loads. Results from these three types of models are compared and evaluated as a first step before a future extension to more detailed analyses using 3D models.

1 INTRODUCTION

The main design principle for rock support is to help the rock carry its inherent loads. The rock support is generally designed for static loading conditions but in many cases, however, the openings are also subjected to dynamic loads. One example is rock bursts that can cause serious damage to underground structures. Another source of dynamic loads is detonation of explosives during excavations of tunnels and underground spaces. These detonations give rise to stress waves that transport energy through the rock and these may, depending on the magnitude of the waves, cause severe damage to permanent installations and support systems within the rock, such as shotcrete.

In tunnelling, the search for a more time-efficient construction process naturally focuses on the possibilities of reducing the time periods of waiting between stages of construction. As an example, the driving of two parallel tunnels requires coordination between the two excavations so that blasting in one tunnel does not, through vibrations, damage temporary support systems in the other tunnel prior to installation of a sturdier, permanent support. There also arise similar problems in mining. To be able to excavate as much ore volume as possible, the grid of drifts in a modern mine is dense. This means that supporting systems in one drift are likely to be affected by vibrations in a neighbouring drift.

This research project was initiated to study how close in time and distance to shotcrete safe blasting can take place. It involves development of sophisticated dynamic finite element (FE) models which will be evaluated and refined through comparisons between calculated and measured data. At first, existing simple prototype models will be evaluated through calculations and comparisons with existing data. The models will then be further refined and re-modelled using FE programs and in the following step the prototype models will be replaced by a model based on solid FE elements. Non-linear material properties will be introduced, replacing the initial, elastic approach. The model will be used to study real in situ cases with realistic and complex geometry, such as shotcrete on tunnel walls and ceilings behind a tunnel front where blasting takes place. The results will lead to the establishment of recommendations and guidelines for practical use in e.g. civil engineering underground work, tunnelling and mining. This paper presents preliminary results, ongoing and future research within the project, assuming fully hardened shotcrete.

2 SHOTCRETE AND BLASTING

There are few published reports and papers on tests conducted in tunnels and mines where shotcrete has been subjected to vibrations from large-scale blasting. Due to the lack of knowledge, unnecessarily strict guidelines for acceptable vibration levels from blasting close to hardened as well as newly sprayed shotcrete are often used. For safe underground construction work, this leads to longer production times and higher costs. The guidelines often contain allowed vibration velocities, expressed as peak particle velocities (ppv), which are difficult to translate into minimum distance and shotcrete age. In the following a short description of stress waves in rock is given, followed by a summary of earlier interesting experiences and research on vibration resistance of shotcrete.

2.1 *Stress waves in rock*

Detonations in rock give rise to stress waves that transport energy through the rock, towards possible free rock surfaces. Wave motion can be described as movement of energy through a material, transportation of energy achieved by particles translating and returning to equilibrium after the wave has passed (Bodare 1997). The propagation velocity is governed by the type of rock and is different for different types of waves, such as compression waves (P-waves) and shear waves (S-waves). The P-wave propagates faster than the S-wave and is therefore the first to reach an observation point when both wave types have been generated simultaneously at a distant source, e.g. an earthquake or a blast round. There are also other types such as Rayleigh waves that appear on surfaces, see e.g. Dowding (1996).

As each wave passes, the motion of the particles in the rock can be described in three dimensions, either as displacements, velocities or accelerations. When a wave-front reflects at a free surface, such as that of a tunnel, the particle velocities are doubled and the stresses are zero over the surface. This means that a compressive wave reflects backwards as a tensile wave, etc. Shotcrete sprayed on rock exposed to blasting will thus be affected by incoming stress waves that reflect at the shotcrete-rock interface. When exposed to incoming stress waves, the inertia forces caused by the accelerations acting on the shotcrete give rise to stresses at the shotcrete-rock interface which may cause adhesive failure. It is also possible that the shotcrete may fail due to low tensile strength. The particle velocities that can be measured remote from a detonation in rock will show a decrease in magnitude with increasing distance to the source of explosion. This decay is caused by geometrical spreading and hysteretic damping in the rock (Dowding 1996) and is governed by a relation of the form

$$v_{\max} = a_1 \left(\frac{R}{\sqrt{Q}} \right)^{-\beta} \tag{1}$$

where $v_{\max}$ is the peak particle velocity (ppv) at a distance R from the point charge with the weight of the explosives Q and the constants a_1 and β. Equation (1) is valid only for situations where R is large compared to the length of the explosive charge, thus assuming a concentration of the explosive charges. As previously mentioned $v_{\max}$ of an incoming wave will be doubled when reflected at the free surface of, for example, a tunnel, and therefore must be obtained from Equation (1) through multiplication by two (Ansell 1999).

The time-dependent stress from a propagating longitudinal wave is

$$\sigma(t) = \rho_{\text{rock}} c_{\text{rock}} v(t) \tag{2}$$

where ρ_{rock} is the rock density and $v(t)$ the particle velocity. The wave propagation velocity in elastic materials is

$$c_{\text{rock}} = \sqrt{\frac{E_{\text{rock}}}{\rho_{\text{rock}}}} \tag{3}$$

where E_{rock} is the elastic modulus of the rock material. It should be noted that the allowable dynamic load on a structural system also depends on the frequency content, ie. higher ppv levels can be accepted for high frequencies. The highest frequencies are however filtered out as the waves propagate through the rock.

2.2 *Dynamic load capacity*

A couple of interesting reports present in situ tests conducted in tunnels and mines where fully hardened shotcrete on rock have been subjected to blast induced vibrations. Kendorski et al. (1973) carried out in situ tests to determine how a shotcrete lining was affected by standard drift blasts at various distances from the lining. A standard blast that consisted of 409 kg premixed ammonium and fuel oil (ANFO) showed that there was no bond failure at the shotcrete–rock interface. The tests revealed that cracks started to appear in the shotcrete when the detonations occurred at a distance of 16.5 m and that the function of the lining was considerably reduced when detonating from 12.2 m. No vibration levels were recorded during these tests. McCreath et al. (1994), Tannant & McDowell (1993), and Wood & Tannant (1994) presented results from tests carried out in a Canadian goldmine where steel fibre-reinforced and steel mesh-reinforced shotcrete linings were subjected to vibrations from explosions. During the tests, it was found that steel fibre-reinforced shotcrete can maintain its functionality even though exposed to vibration levels of 1500–2000 mm/s. It was seen that mesh-reinforced shotcrete performs better than steel fibre-reinforced shotcrete under very severe dynamic loading conditions. This is due to its ability to retain broken rock even when extensively cracked, which is not the case with fibre-reinforced shotcrete. It is reported that the shotcrete linings were partially cracked and that no shotcrete slabs were displaced or ejected by the blasting, nor was any significant increase in drumminess found by manual hammer sounding as the blasting continued. The latter indicates that the shotcrete–rock adhesive bond was undamaged. Further, it is suggested that mesh-reinforced shotcrete may remain partially functional when subjected to particle velocities as high as 2000–6000 mm/s (McCreath et al. 1994). These conclusions were based on empirical observations by the ground control staff at operating mines in the Sudbury area (Ontario, Canada).

The function of shotcrete in the support of burst prone ground has further been investigated through static and dynamic testing of shotcrete panels by Beauchamp (1995), also presented by Tannant et al. (1995, 1996). In their test program, impacts of rock blocks ejected from medium-sized rock bursts were simulated by using weights dropped on shotcrete panels. Different loading rates and panel configurations were tested. The result was presented as a relationship between panel damage, energy dissipation and displacement.

3 PRELIMINARY RESULTS

As a continuation to the research described in the previous section in situ tests with small and larger scale detonations were conducted, followed by numerical modelling (Ansell 1999, 2004a, 2004b, 2005, 2007). These results are briefly summarized and commented on in the following.

3.1 *In situ tests*

Tests on young shotcrete were performed on site in the Kiirunavaara iron-ore mine, situated in the north of Sweden (Ansell 1999, 2004a). The test sites were situated on both sides of a tunnel that was not originally reinforced by shotcrete. A total of four tests were carried out, each performed with a unique type of explosive charge not repeated in any of the other tests. Two shotcrete areas of varying age, one young and one very young, were subjected to vibrations in each test and no shotcrete area was subjected to vibrations more than once. The geometry of a test site is described in Figure 1.

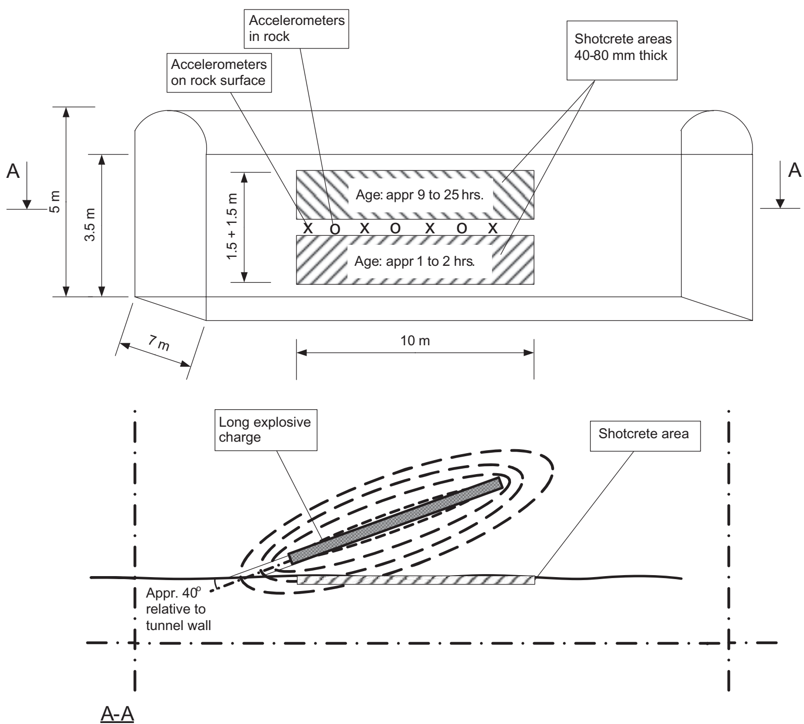

Figure 1. Schematic view of a test site. Explosive charge in rock behind shotcrete areas (Ansell & Holmgren 2001).

Measurement points for placement of accelerometers were located in the same horizontal plane as the charge holes, four points placed on the tunnel surface and three points 0.5 m into the rock. Three plastic pipes for mounting of accelerometers inside the rock were applied through injection by cement grout and four aluminium plates for accelerometers were bolted to the rock surface.

At each measurement point two accelerometers were mounted, normal to and parallel with the tunnel walls, i.e. accelerations were recorded using a total of 14 channels in each test. In the evaluation model used, it was assumed that only the first peak in each record provided a correct maximum velocity, i.e. that no reflections and superpositions had at such an early stage distorted the signals.

This method of determining the maximum particle velocity, or peak particle velocity (ppv), is sometimes referred to as the "first-arrival principle". In the ideal case, the ppv will coincide with the point where both of the two measured (in reality, three) acceleration components pass through the zero level simultaneously after the first half period of vibration. An example of a measured acceleration signal is shown in Figure 2. The equation

$$v_{\max} = 700\left(\frac{R}{\sqrt{Q}}\right)^{-1.70} \quad [\text{mm/s}] \tag{4}$$

was derived through regression analysis (Ansell 1999) from all the numerically determined ppv values and is valid for estimation of the vibration velocities inside the rock at Kiirunavaara.

3.2 *Numerical modelling*

A first step in modelling of shotcrete exposed to vibrations was taken by Ansell (1999). The in situ tests with young shotcrete were followed by numerical modelling with a first approach based on elastic stress wave theory. This resulted in a model for one-dimensional analysis of shotcrete on rock through which elastic stress waves propagate towards the shotcrete. Acceleration–time histories recorded in situ were used as loads. In a following project, an elastic finite element model was proposed (Ansell 2004b, 2005). In contrast to the earlier one-dimensional stress wave model, this finite element model is two-dimensional. This facilitates the calculation of a two-dimensional displacement field instead of the displacement at an isolated node. The model consists of beam elements that are used for representation of the flexural stiffness and mass of the shotcrete and the fractured rock closest to the rock surface. Spring elements are used to obtain elastic coupling between shotcrete and rock. The modelling approach is similar to that of a building during an earthquake, with accelerations measured in situ used as loads.

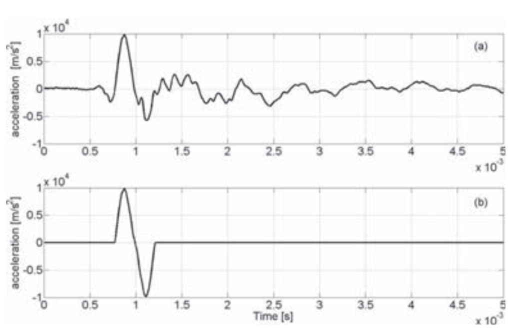

Figure 2. Example of measured accelerations (a) and part of the signal used for numeric analysis (b). Direction of measurement is normal to the rock surface.

3.3 *Recommendations*

The preliminary numerical results have been verified through comparisons with observations and results in situ, from full scale tests (Ansell 2004a) and from mining operations (Ansell & Malmgren 2003 and Ansell 2007). The results showed that the main failure mode is loss of the adhesive bond between shotcrete and rock. The work resulted in recommendations of peak particle velocities (ppv) calculated from the scaling relation given by Equation (4). These are here given in Table 1, also including a summary of the results referred to in section 2.2.

Table 1. Vibration peak particle velocities (ppv) when shotcrete damage occurs. Values from observations, measurements and calculations (Ansell 2007).

ppv at damage (mm/s)		Comments
From measurements:		
Kiirunavaara tests (Ansell 1999, 2004a)	500–1000	Young shotcrete
Japanese tunnelling (Nakano et al. 1993)	700–1450	
Mining, full scale (Kendorski 1973)	1000–1800	Estimated
Canadian tests (Wood & Tannant 1994, McCreath et al. 1994)	1500–2000	Steel fibre-reinforced
Recommendation:		
(Ansell 2005, 2007)	2300–3000	25 mm shotcrete
	700–710	50 mm shotcrete
	230–250	100 mm shotcrete

4 NUMERICAL MODELS

The evaluated models are one-dimensional and the point of detonation was assumed to be positioned within the rock, perpendicular to the point of observation on the free surface. Hence, all the models describe the stress response at one discrete point. The incident disturbing stress wave used in the comparison of the different types of models was derived from the scaling law for rock, Equation (1), together with the presumption of a characteristic frequency. That is, the applied disturbance is a theoretically defined approximation to those observed in situ. It thus provides a clear relation between the disturbance expressed as acceleration, velocity, displacement and stress and therefore enables comprehensible comparisons to be made between the different types of models and methods for defining the incident disturbing stress wave. The evaluation was made for a detonation of 2 kg of explosives concentrated at one point positioned 4 m below the rock surface. The shotcrete thickness was kept constant at 0.05 m.

4.1 *Material properties*

A simplification that was made throughout this study was that both the rock and the shotcrete were considered to behave in a strictly elastic manner. That is, no plastic deformations or permanent failure, e.g. crushing, cracking or debonding, were considered within the models and a linear elastic relationship between stress and strain was assumed.

It is difficult to define material parameters for a fractured rock mass since its behaviour is site-specific and this affects the characteristics of propagating stress waves (Wersäll 2008). Here, a Young's modulus of elasticity of 40 GPa was chosen for the rock material, assumed to be linearly elastic. Furthermore, the rock properties in the numerical examples were chosen to represent typical Swedish granite; a density of 2400 kg/m^3, a compressive strength of 160–240 MPa and a Poisson's ratio set to zero, since the problem treated here was purely one-dimensional. The longitudinal wave velocity was 4000 m/s and the particle velocity was assumed to follow the relation given in Equation (4).

A shotcrete lining can be either un-reinforced or reinforced, for instance with steel fibres, and its strength will be dependent on the composition of the shotcrete, the age of the maturing shotcrete and its ability to adhere to the rock surface. The adhesion strength perpendicular to the rock surface often lies within 0 and 2 MPa (Ansell 1999). Since this strength often is lower than both the compressive and the tensile strengths the adhesive bond to the rock will be crucial. In the numerical examples fully hardened (28 days) un-reinforced shotcrete was considered and the adhesion strength towards granite should for comparisons with older results be set to approximately 0.5 MPa. The shotcrete was furthermore assumed to have a density of 2100 kg/m^3 and a modulus of elasticity of 30 GPa. The chosen material properties were similar to that of the shotcrete used by LKAB in the Kiirunavaara mine, see Ansell (1999).

4.2 *Elastic stress wave model*

In elastic analysis, the material is often assumed to be infinite (or semi-infinite), linear elastic, continuous and isotropic. Rock mass is usually none of the above but these assumptions can be valid for deep tunnels subjected to small strains, e.g. from blast loads. An elastic stress wave model for analyses of shotcrete on vibrating rock, based on these assumptions, has been tested by James (1998) and Ansell (1999). The model was implemented with the numeric software Matlab (Mathworks 2009). Since the longitudinal P-wave was presumed to cause the first and major stress influence, this type of wave was the only one considered in the model. The effect of damping was disregarded and the stresses were assumed to follow Equation (2).

A graphical description of the model is shown in Figure 3 where the shotcrete lining is divided into n elements, the thickness of each element, Δx, being determined so that in each time increment the wave has propagated a distance of one element. The stress wave was assumed to be transmitted and reflected at boundaries and subsequently the stresses travelling in the positive and negative directions were superimposed to give the total stress within each element of the shotcrete lining.

4.3 *Structural dynamic model*

This mass-spring model includes fractured rock and the shotcrete lining, as lumped masses positioned

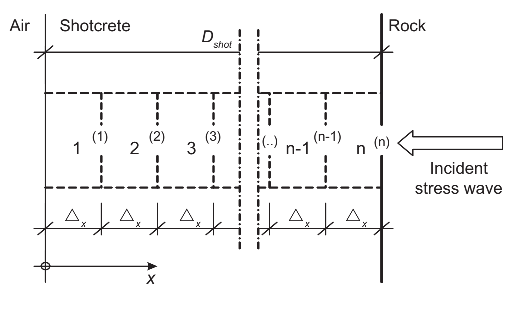

Figure 3. Elastic stress wave model of a shotcrete lining with thickness D_{shot} applied to a rock surface and exposed to a stress wave. The shotcrete lining is divided into n elements, each with thickness Δx (Nilsson 2009).

at the centres of gravity of each segment of the model, as seen in Figure 4. These were connected to the adjacent masses with springs corresponding to the axial stiffness of the shotcrete or fractured rock element. The degree of discretisation of the shotcrete has almost no influence on the maximum stress level, which developed within the shotcrete for both models. The greatest stress level is obtained in the shotcrete closest to the rock, and thereafter the stress level descended to zero towards the free surface. Since the total number of elements required is small for a model with masses at the centre of gravity, compared to if the masses are positioned at the interfaces (Nilsson 2009), this type of model was considered here. For the numerical analyses presented the shotcrete was modelled with one element, whilst the rock was divided into 40 elements. Also in this case Matlab (Mathworks 2009) was used for the analyses.

4.4 *Finite element model*

The engineering simulation software Abaqus (Simulia 2009a, 2009b) was used to create finite element models analyzed with the Abaqus/Explicit solver. A condition of plane strain was presumed in the directions parallel to the free surface of shotcrete. This implies that three-dimensional solid elements can be used to model the rock and shotcrete. Furthermore, when modelling a rectangular rod of rock and shotcrete with an incident longitudinal wave striking the inner surface of rock (or shotcrete) three-dimensional solid elements can be used as well. Simplifying the model to a rectangular rod of rock and shotcrete also means that the displacements in the directions transverse to the wave propagation can be constrained using displacement boundary conditions, thus restricting

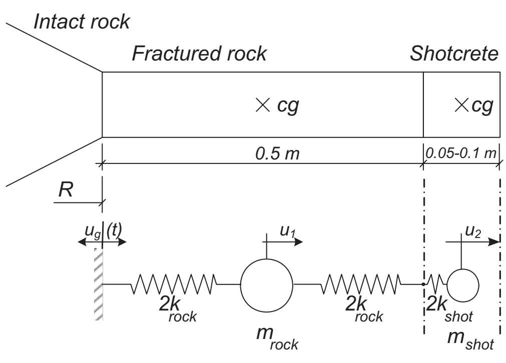

Figure 4. Structural dynamic model including both fractured rock and shotcrete exposed to ground motion, with masses at the centre of gravity of each element. R denotes the distance used in the scaling law for rock and u_i the displacements normal to the rock surface.

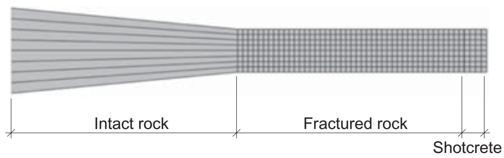

Figure 5. An Abaqus-model with intact rock (infinite elements), fractured rock (finite elements) and shotcrete (finite elements) modelled with plane strain elements. Displacements in the vertical direction are restricted (Nilsson 2009).

Table 2. Properties for Abaqus/Explicit model consisting of shotcrete lining and fractured rock.

	Part		
Specific properties	Shotcrete	Fractured rock	Intact rock
Cross section	0.1×0.1 m^2	0.1×0.1 m^2	–
Thickness	0.05 m	0.50 m	–
Elements	8×4	8×40	8
Element type	C3D8R*	C3D8R*	infinite

* C3D8R Explicit, continuum, 3D stress, 8-node linear, reduced integration, hourglass control.

the model to primarily describe particle displacements in the wave propagation direction.

Figure 5 shows the tested Abaqus-model with plane strain elements and features according to Table 2. The fractured rock and the shotcrete were modelled with finite elements and the intact rock with infinite elements. Tie constraints between rock and shotcrete were used and the infinite elements simulate a boundary with no reflection, also called a quiet boundary. The element size within the model was chosen to be 0.0125 m (Nilsson 2009). The incident disturbing stress wave, caused by the explosion within the rock, was applied in two principally different ways; as a boundary condition, i.e. as acceleration, velocity or displacement, and as a stress-load.

It is to be noted that the amplitudes describing the stress wave are dependent on the distance R to the point of detonation.

5 RESULTS

Results using theoretically determined (forecasted) particle accelerations and in situ measured accelerations as loads will be shown. First are the stress responses from all three types of models—elastic stress wave model, structural dynamic model and finite element model—using forecasted particle accelerations presented together with a comparison of the results. Thereafter, the elastic stress wave model is used to simulate the stress responses from two acceleration-time histories, measured in situ.

5.1 *Model comparisons*

The incident disturbing waves used as loads in this comparison were assumed sinusoidal acceleration variations with peak particle acceleration of either 2259 m/s^2 at 0.5 m below the rock surface or 1849 m/s^2 right under the rock surface, respectively. A frequency of 2000 Hz was considered, see also Nilsson (2009). In the examples of the elastic stress wave model and the Abaqus/Explicit models the stress responses were obtained by using an initial velocity conditions equal to zero, as well as an initial velocity condition equal to the maximum amplitude, as in Figure 6. The incident stress variation was obtained using Equation (2). When boundary conditions were used to describe the incident disturbance the stresses were set to zero after passing of the wave, to prevent the particle (node) acceleration at the interface between the intact and the fractured rock increasing at an unreasonable rate.

The stress responses at the shotcrete element closest to the rock surface are plotted for all models in Figures 7 and 8, with the incident disturbance defined at the interface between intact and fractured rock and between rock and shotcrete, respectively. In this case, the initial velocity condition was set to zero, as shown in Figure 6(b).

The case described in Figure 6(a) corresponds to a suddenly applied stress level followed by a cosine-period. This results in stress peaks that propagate through the elements of the elastic stress wave model and the Abaqus/Explicit finite element model. The stress responses in the shotcrete element closest to the rock surface when an incident disturbance is defined at the interface between intact and fractured rock and between rock and shotcrete, respectively, are as shown in Figures 9 and 10.

5.2 *From in situ measurements*

As part of the testing and evaluation of the models, in situ recorded acceleration-time history data from the Kiirunavaara mine (Ansell 1999) is also used as input. For the numerical examples, the accelerometer readings shown in Figure 2 were chosen. The sampling frequency f_{samp} of these in situ measured accelerations is 100 kHz, giving a time interval Δt of 10^{-5} s. For an adequate description of the sine-wave accelerations in the previous examples, a time increment Δt of 10^{-6} s was needed and therefore the same time interval was used also for the in situ measured accelerations. Thus, a linear interpolation

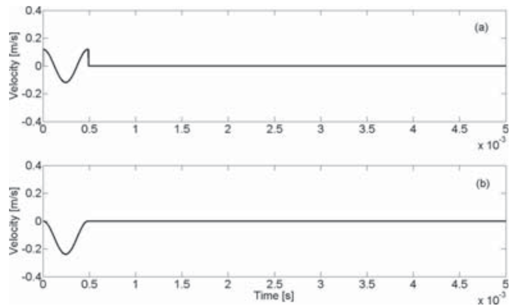

Figure 6. Particle velocity derived from accelerations, with the condition of initial velocity equal to (a) its maximum amplitude and (b) to zero.

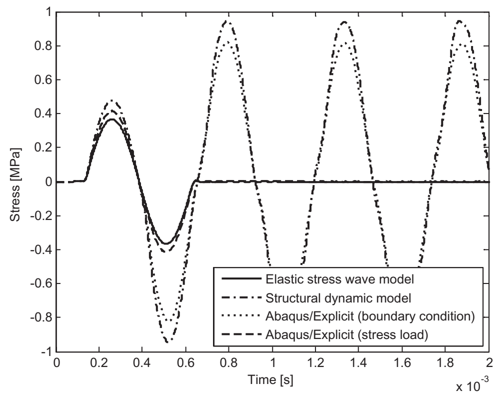

Figure 7. Stress responses in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between intact and fractured rock for all models but the elastic stress wave model. The initial velocity condition was set to zero (Nilsson 2009).

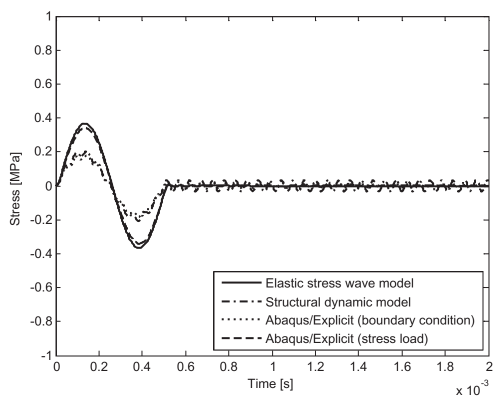

Figure 8. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between rock and shotcrete. The initial velocity condition was set to zero (Nilsson 2009).

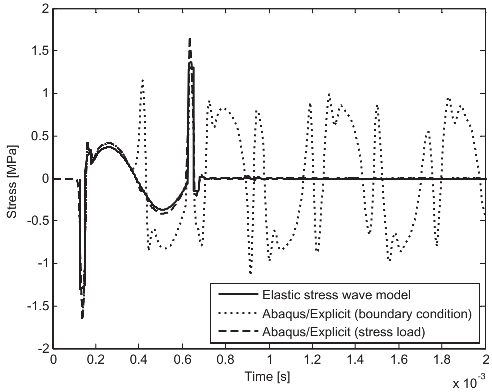

Figure 9. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between intact and fractured rock for all models but the elastic stress wave model. The initial velocity condition was set to its maximum amplitude (Nilsson 2009).

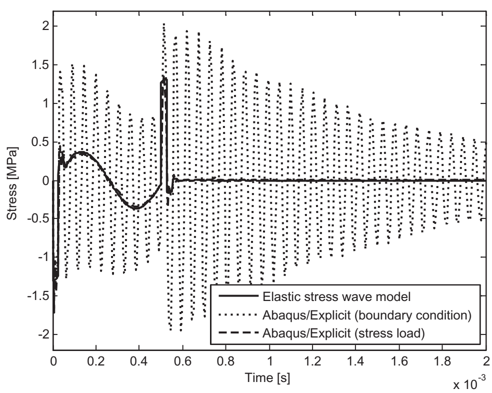

Figure 10. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between rock and shotcrete. The initial velocity condition was set to its maximum amplitude (Nilsson 2009).

was made between the measured points. The time history of the measured accelerations furthermore contain quite some noise and only the first part of the signal up to the peak particle velocity was considered, see also Ansell (1999). For the velocity to return to zero the acceleration-time history was then mirrored, see Figure 2(b).

The elastic stress wave model and structural dynamic model were used to simulate the stress response within the shotcrete lining from the in situ acceleration-time history data from Figure 2(b). The stress responses in the shotcrete elements

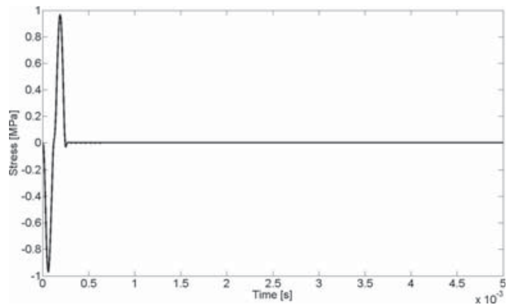

Figure 11. Stress response in the shotcrete element closest to the rock from in situ measured accelerations. Calculated with elastic stress wave model.

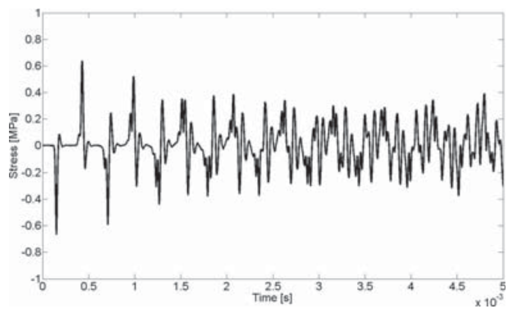

Figure 12. Stress response in the shotcrete element closest to the rock from in situ measured accelerations. Calculated with structural dynamic model.

closest to the rock surface are shown in Figures 11 and 12, respectively.

6 CONCLUSIONS

The three numerical models compared in this paper give similar results, with some exceptions. Despite the fact that the models only include wave propagation in one dimension the results are sometimes rather complex and difficult to interpret. The definition of the incoming stress wave must be adapted to each specific case studied and model used. The results do, however, show that it is possible to obtain similar results using the models for one-dimensional cases. It is important to note that the results from the older stress wave model and structural dynamic model also can be obtained with simple models created with the sophisticated FE program Abaqus/Explicit. It is therefore possible to continue the model development using FE models, while being able to compare the results to previous results.

6.1 *Discussion*

The comparison between the results from the tested models show good agreement in time and an acceptable coincidence in stresses when the incoming waves start from the zero level. This can be seen in Figures 7 and 8 from where it is also clear that the methods give different results depending on the definition of the load and place of insertion in the models. The elastic stress wave model is loaded by a stress wave that gives results that coincide with those from the similarly loaded Abaqus model. Application of the load as a boundary condition in Abaqus results in stresses that are very similar to those from the structural dynamic model.

It should be noted that there is a difference in time between when the stresses first appear in Figures 7 and 8, respectively, and this is due to the time it takes for the waves to propagate through the 0.5 m of rock. The lower stress-curves seen in Figure 8 are a result of the application of the loads at the interface between the rock and shotcrete. The structural dynamic model for such a case only consists of the shotcrete part, which gives a small mass and a relatively large stiffness. The model is thus stiffness dominated which can be seen in the high frequency vibrations that are superimposed over the stress curves, all the way until 0.5 ms. It is the positive stresses that are of interest here as they represent tensile stresses between rock and shotcrete, i.e. the adhesive bond stress. The comparison should therefore be done firstly with respect to the first arriving stress maxima.

When the stress load, calculated from velocities of the incoming wave, is suddenly applied and followed by a cosine-variation, the results become more complex but basically still comparable with those from the first load case. It is not practical to use such a load with the structural dynamic model and this was therefore omitted in the comparison made in Figures 9 and 10. These figures show large, sharp peaks with opposite signs, superimposed on sine-shaped stress waves similar to those in Figures 7 and 8. Such peaks correspond to a compressive impulse that is a result of the sudden increase in stress as the load is applied. When the load is applied as a boundary condition at the interface between rock and shotcrete this will dominate the results obtained from the Abaqus-model, as seen in Figure 10. It can be concluded that the maximum tensile stresses in principle coincide for the three models and for the different ways of applying the loads but that there exist superimposed disturbances and peaks that originate from dynamic effects.

The results obtained with measured accelerations demonstrate how in situ data can be used to evaluate the risk for bond failure between rock and shotcrete by using the elastic stress wave and structural dynamic models, respectively. The results in Figure 11 show that the response calculated with the stress wave model in this case kept the harmonic form of the input wave in Figure 2(b). This is not the case for the results from the structural dynamic model in Figure 12 where it can clearly be seen that the dynamics of the mass-spring system affects the results, as was the case in Figure 8. The in situ acceleration data used has previously been evaluated with the stress wave model by Ansell (1999), including all results from the four test sites. Using a future three dimensional finite element method it will be possible to simultaneously include data that describe accelerations in two or three directions at many points of the model.

6.2 *Ongoing research*

The ongoing research aim is to extend the finite element models in Abaqus/Explicit to describe three-dimensional geometries and displacements. Linearly elastic material models of dynamically loaded shotcrete have been described, evaluated and compared and non-linear properties and behaviour of the rock and shotcrete will also be introduced in these models. The point detonation can possibly be modelled as a cavity within the rock with an impulsive pressure load on its surface. To allow the stress wave to disperse a rather large volume of rock will have to be modelled, depending on the distance to the detonation of explosives. For evaluation of the models further in situ acceleration measurements will be undertaken. The aim is to describe the wave propagation along tunnel walls following blasting at a tunnel front. The further work will also study newly sprayed and hardening shotcrete. The goal is detailed dynamic analyses using 3D models.

ACKNOWLEDGEMENTS

The authors thank all those who have supported the project, especially Ms Cecilia Nilsson who contributed with the preliminary results presented in this paper.

REFERENCES

Ansell, A. 1999. *Dynamically loaded rock reinforcement*, Doctoral thesis, Bulletin 52. Stockholm: KTH Structural Engineering.

Ansell, A. 2004a. In situ testing of young shotcrete subjected to vibrations from blasting. *Tunnelling and Underground Space Technology*, 19(6): 587–596.

Ansell, A. 2004b. A finite element model for dynamic analysis of shotcrete on rock subjected to blast induced vibrations. In Bernard (ed.), *Second International Conference on Engineering Developments in Shotcrete*, Cairns, 4–6 October 2004. Rotterdam: Balkema.

Ansell, A. 2005. Recommendations for shotcrete on rock subjected to blasting vibrations, based on finite element dynamic analysis. *Magazine of Concrete Research*, 57(3): 123–133.

Ansell, A. 2007. Shotcrete on rock exposed to large-scale blasting. *Magazine of Concrete Research*, 59(9): 663–671.

Ansell, A. & Holmgren, J. 2001. Dynamically loaded young shotcrete linings. In Bernard (ed.), *International Conference on Engineering Developments in Shotcrete*, Hobart, 2–4 April 2001. Rotterdam: Balkema.

Ansell, A. & Malmgren, L. 2003. The response of shotcrete to blast induced vibrations in mining. In Handley & Stacey (ed.), *Technology roadmap for rock mechanics, Proceedings of the 10th Congress of the ISRM*, Sandton City, 8–12 September 2003. Johannesburg: SAIMM.

Beauchamp, K.J. 1995. *Impact testing of large scale shotcrete panels*, Master thesis. Waterloo: University of Waterloo.

Bodare, A. 1997. *Jord- och Berdynamik* (*Soil and Rock Dynamics*, in Swedish). Stockholm: KTH Dept. of Soil and rock Mechanics.

Dowding, C.H. 1996. *Construction Vibrations*. Upper Saddle River: Prentice-Hall.

James, G. 1998. *Modelling of young shotcrete on rock subjected to shock wave,* Master thesis. Stockholm: Royal Institute of Technology.

Kendorski, F.S., Jude, C.V. & Duncan, W.M. 1973. Effect of blasting on shotcrete drift linings. *Mining. Engineering*, 25: 38–41.

Mathworks. 2009. http://www.mathworks.com/products/matlab/

McCreath, D.R., Tannant, D.D. & Langille, C.C. 1994. Survivability of shotcrete near blasts. In Nelson & Laubach (ed.), *Rock Mechanics*. Rotterdam: Balkema.

Nakano N., Okada S., Furukawa K. & Nakagawa, K. 1993. Vibration and cracking of tunnel lining due to adjacent blasting (in Japanese, Abstract in English). *Proceedings of the Japan Society of Civil Engineers*, 462: 53–62.

Nilsson, C. 2009. *Modelling of dynamically loaded shotcrete*, Master thesis 276. Stockholm: KTH Structural Engineering.

Simulia. 2009a. http://www.simulia.com/products/abaqus_standard.html

Simulia. 2009b. http://www.simulia.com/products/abaqus_explicit.html

Tannant, D.D., Kaiser, P.K. & McCreath, D.R. 1996. Impact tests on shotcrete and implications for design for dynamic loading: field tests and modelling. In Aubertin, Hassani & Mitri (ed.), *Proceedings of 2nd North American Rock Mechanics symposium*, Montreal, 19–21 June 1996. London: Taylor and Francis.

Tannant, D.D., Kaiser, P.K., McCreath, D.R., McDowell, G.M. & Beauchamp, K. 1995. Large-scale impact tests on shotcrete panels. In *Proceedings of the 12th CIM Mine Operators Conference*, Timmins.

Tannant, D.D. & McDowell, G.M. 1993. *Dynamic testing of shotcreted drifts*. GRC Internal Report 93-12-IR. Sudbury: Laurentian University.

Wood, D.F. & Tannant, D.D. 1994. Blast damage to steel fibrereinforced shotcrete. In Banthia & Mindess (ed.), *Fiber Reinforced Concrete—Modern Developments*. Toronto: UBC Press.

Wersäll, C. 2008. *Blast-induced vibrations and stress field changes around circular tunnels*, Master thesis. Stockholm: KTH Dept. of Soil and rock Mechanics.