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Vibration vulnerability of shotcrete on tunnel walls during construction blasting

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ABSTRACT

The effect on shotcrete from blasting operations during tunnelling is studied, with focus on young and hardening shotcrete. A finite element model specially adapted for analysis of the shotcrete behaviour is tested, it is able to describe stress wave propagation in two dimensions which is important for cases where shear stresses are dominant. The modelling results are compared with in situ measurements and observations, from construction blasting during tunnelling through hard rock. The comparison shows that the model gives realistic results and can be used to investigate the vulnerability of shotcrete, aiming at compiling recommendations and guidelines for practical use. The given recommendations emphasize that blasting should be avoided during the first 12 h after shotcreting and that distance and shotcrete thickness are important factors for how much additional time of waiting is possibly needed.

1. Background

For safe excavation and tunnelling through hard rock it is often important to apply shotcrete (sprayed concrete) on the rock surfaces already at an early stage to secure potentially loose blocks and the shape of tunnels or other subspace openings. For a time-efficient construction process the time of waiting between stages of excavation needs to be minimized. Hardening concrete, and shotcrete, is vulnerable to disturbance during early age after casting or spraying, which can lead to failure or a reduction in strength. This is the case for the development of the bond between shotcrete and rock which is sensitive to vibrations, especially during the first 12–24 h after shotcreting. For safe tunnelling and underground work it is thus necessary to know when e.g. the bond between rock and newly sprayed shotcrete has reached an age where it can withstand vibrations at certain levels. This is particularly important since most work in hard rock requires blasting operations resulting in stress waves that transport energy through the rock and which may cause severe damage on permanent installations and support systems within the rock, such as shotcrete. An important example is the driving of two parallel tunnels that requires coordination between the two excavations so that blasting in one tunnel does not damage temporary support systems in the other tunnel prior to placing of a sturdier, permanent support. Similar problems also arise in mining operation where the need to excavate as much ore volume as possible leads to that the grid of drifts in a modern mine is dense. Established, detailed guidelines for acceptable vibration levels from blasting close to young and newly sprayed shotcrete is missing. Therefore, unnecessarily strict limit values are often used, leading to longer production times and larger costs. The limits set up are often expressed as maximum allowed vibration velocities, or peak particle velocities (ppv). It is often difficult to translate these into minimum distance and shotcrete age, allowed amount of explosives to be used, sequence of detonations, etc. For this to be possible information on geometry of e.g. the tunnel together with material properties of rock and shotcrete must be considered. Such detailed guidelines for how close, in time and distance, to young shotcrete blasting can take place would thus be an important tool in planning for safe and economical tunnelling projects.

2. Previous investigations

2.1. Numerical modelling

As a step towards detailed guidelines for practical use, a series of projects with analytical and numerical modelling using in situ observations and measurements for verification have been carried out. These projects have resulted in recommendations for fully hardened shotcrete exposed to small amounts of explosives at short distances (Ansell, 2005), large scale detonations at larger distance (Ansell, 2007a) and for young and hardening shotcrete

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(Ansell, 2007b). Additional recommendations are given by Ahmed (2012). The first models used and the cases studied were kept simple to facilitate the comparison between different modelling concepts and between calculated and in situ results, Ahmed and Ansell (2012a). With these simplified models, based on linear elastic material theory, the efficient analytical procedure makes it possible to compare large numbers of calculations with various combinations of input data. Also, the interpretation of the results become straightforward but the possibility to study various geometrical conditions is heavily restricted. It is thus not possible to describe partially damaged structures, e.g. partial de-bonding of shotcrete, only to identify the limit for damage through series of calculations. One model tested was a one-dimensional elastic stress wave model, capable of describing wave propagation in one direction, including reflections and transmissions at material interfaces and free surfaces. The other two models were based on structural dynamic theories with vibrating masses and beams connected through elastic springs. The comparison and evaluation of the three numerical models showed that their results were comparable, although the definition of the dynamic loads was different. For the stress wave model the dynamic load was defined as a time dependent velocity while a time dependent acceleration was used for the structural dynamic models. Based on the experience from these first models the further work focused on developing a more advanced two-dimensional (plane strain) finite element (FE) modelling concept, Ahmed et al. (2012). The use of established FE programs facilitates the study of more complex geometries and it will also be possible to include the effect of rock bolts and partially damaged shotcrete, as the model is developed further.

2.2. In situ measurements

The modelling results have been verified through comparison with in situ observations, measurements and laboratory tests. The latter were conducted as model tests using a shotcrete covered concrete beam with dynamic properties similar to good quality granite subjected to an impacting hammer that produced stress waves similar to that from blasting (Ahmed and Ansell, 2012b). Published, relevant in situ measurement data is scarce. In situ testing to determine how a shotcrete lining was affected by standard drift blasts at various distances from the lining was done by Kendorski et al. (1983), but no vibration levels were recorded during these tests. Measurements during the construction of parallel tunnels are presented by Nakano et al. (1993). Recorded accelerations or velocities are not available but particle velocities of 1450 mm/s are reported to have been reached with a smallest distance between blasting points and shotcrete that was 1.0 m. Results from tests carried out in a Canadian goldmine where steel fibre-reinforced and steel mesh-reinforced shotcrete linings have been subjected to vibrations from explosions are presented by Wood and Tannant (1994) and McCreath et al. (1994), reporting vibration levels of 1500–2000 mm/s. As these results are either incompletely documented or not compatible with the numerical models tested, results from tests on young shotcrete performed during 1999 on site in the Kiirunavaara iron-ore mine, in the very northern of Sweden (Ansell, 2004) have been used for the verification. The measurement set-up was similar to that used by Jinnerot and Nilsson (1998) who conducted tests aiming at determining damage zones in the rock due to blasting operations. Results from measurements during full scale production blasting in the mine has also been used, see Ansell (2007a). During these three measurement projects accelerometers were placed on a horizontal line along the length of the tunnel, two accelerometers at each measurement point to provide a two-axial description of the vibrations, parallel with and perpendicular to the length of the tunnel. Surface mounted accelerometers were placed on steel or aluminium plates bolted to the rock. For measurements inside the rock, accelerometers were mounted inside fully grouted pipes of PVC. The inner end of these pipes consists of a conical tip, of steel or aluminium that is in contact with the rock and contains holders for the accelerometers on the inside. The lengths of the pipes were in these cases adjusted so that the measurement points are about 300–500 mm behind the rock surface. The obtained acceleration measurements have provided a good set of data for the evaluation of the analytical and numerical models with respect to the risk for local shotcrete damage in the vicinity to the source of blasting. For further analysis of more advanced geometrical conditions the models need to be verified with respect to wave propagation along the length of the tunnel, caused by blasting at a remote point, e.g. at the tunnel front. This paper presents such an evaluation, through comparison with a series of measurements done in situ during tunnelling construction work.

2.3. Measurements during construction blasting

An attempt to characterize the vibrations that occur along tunnel walls during excavation blasting has been performed by Reidarman and Nyberg (2000). The measurements were done during construction of the Southern Link (Södra länken) road tunnel system in Stockholm, Sweden. The accelerometers used were positioned following the same system as for the Kiirunavaara measurements, described above. The results from this investigation are well suited for evaluation of the FE models described above. No shotcrete damage was observed following the blasting, due to the very restrict guidelines used, and it can thus be assumed that the shotcrete-rock system behaves elastically throughout the passage of the stress waves. The FE model which is based on elastic material properties can therefore be used for a numerical study of the stress wave propagation along these tunnel walls. The measurement of vibrations from four blasting rounds was done using accelerometers located along an axis stretching approximately 5–50 m behind the tunnel front. Accelerations were measured in two directions, parallel with and perpendicular to the tunnel walls, recorded and later numerically recalculated into corresponding velocity–time records. All measurement points were situated 300 mm into the rock. The layout of the test tunnel with the position for the measurement points is shown in Fig. 1. It should be noted that the advancement of the tunnel front is towards the left in the figure and that each blasting round results in 5 m new tunnel length, except for the third round which gave a 10 m extension. The figure also shows how some measurement points were abandoned in favour of new points closer to the tunnel face, thus approximately giving equal spacing between the points for each of the four rounds.

The maximum velocities for each point vs. the distances along the tunnel wall are shown in Fig. 2, with curves fitted using the method of least squares, in the direction parallel with and perpendicular to the tunnel wall $v_{\max,x}$ and $v_{\max,y}$, respectively, giving:

$$\begin{cases} v_{\max,x} = 74.81e^{-0.065x} \ (\mathrm{mm/s}) \\ v_{\max,y} = 18.48e^{-0.026x} \ (\mathrm{mm/s}) \end{cases} \tag{1}$$

with the distance x along the tunnel given in metres. It should be noted that the ppv from a single blast hole did not exceed 80 mm/s in any test point, in any direction.

3. Stress waves in rock

3.1. Wave types

Detonations in rock give rise to stress waves that propagate outwards from the detonation, in all directions through the rock and