In-situ dynamic testing of rock support at LKAB Kiirunavaara mine

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Abstract

A series of large scale dynamic tests were conducted at the LKAB Kiirunavaara mine using explosives to generate the dynamic load on the support system. This was done with the aim of developing a testing methodology for in-situ testing of ground support. Furthermore, the response of the installed rock support system to strong dynamic loading was evaluated. The results of the Tests 1, 2, 3, 4 and 5 indicated that the relation between the burden and the used amount of explosive had a vital role in either reducing or involving the effect of the detonation gases in the test results. Higher peak particle velocities were measured compared to those of similar large scale tests carried out in other countries. However, the level of induced damage in Tests 1 and 2 was limited to a fractured zone behind the support system while in Tests 4 and 5 the burden was unexpectedly destroyed. Based on the test results and preliminary numerical analysis, a modified test (Test 6) was designed at the same mine. The aim was to avoid the unexpected damage of burden as was observed in earlier tests, and to modify the dynamic loading leading to increase the depth of fractured zone and if possible pushing the support system beyond its limit. Results indicated that a larger fractured zone compare to earlier tests was developed behind the support system while the installed support system was still functional. Evidence from the damage to the tested cross-cuts in Test 6 indicated a reduction of radial cracks that provide access for the gas expansion. The results indicated that the installed support system, designed for dynamic conditions, performed well under the loading conditions which can cause ejection.

1 Introduction

In the design of support systems under seismic loading conditions, neither the demand nor the capacity of a support system can be satisfactorily defined leading to a case of design indeterminacy (Stacey 2012). Therefore, in order to quantify the performance of the rock support systems suitable for dynamic loading conditions, four main types of dynamic tests are considered including simulated large scale experiments by means of blasting, drop test facilities that apply an impact load on the reinforcement, laboratory tests applying dynamic loads on core samples, and passive monitoring and back analysis of case studies (Hadjigeorgiou and Potvin 2007). There are advantageous and disadvantageous with any of these approaches, but the provided data by different methods are complementary leading to the development of the data base for dynamic rock support performance (Hadjigeorgiou and Potvin 2007).

Within the framework of a research program focused on deep mining problems at Luleå University of Technology, in-situ dynamic testing of rock support using blasting as the seismic source was conducted in the Kiirunavaara underground mine, owned and operated by Luossavaara Kiirunavaara Aktiebolag (LKAB). The main objective of the tests conducted was to develop a large-scale in-situ testing method for evaluating rock support performance. This was done by exposing the rock support system to seismic waves generated by means of blasting. For this purpose, a series of large scale tests (Tests 1 to 7) were carried out in pillars/cross-cuts in the northernmost part of the Kiirunavaara mine. Results from Test 1, 2, 4 and 5 are presented in detail in Shirzadegan et al. (2016b), and Test 6 in Shirzadegan (2016a). This paper presents a summary of the all conducted 7 tests.

It should be noted that there are certain differences between loading conditions from a seismic event and a simulated rockburst by means of blasting. A real seismic event of shear type first initiates a compressive wave followed by a larger amplitude shear wave which carries more energy than the longitudinal wave. The event-induced waves comprise all frequencies but the high frequency components are rapidly attenuated with travelling distance while the low frequency components attenuate with a much slower rate with distance. This means that ejection is most likely to occur close to the seismic source while significant dynamic stress contributions (tensile and compressive) leading to damage caused by superimposed static and dynamic stresses can be expected at all distances from the seismic source. This is also true for the shaking damage mechanism. Blasts produce waves which are dominated by P-waves with higher dominant frequencies than those induced by real seismic events. It was therefore decided to focus on simulating the ejection damage mechanism using blasting as the seismic source.

Large scale seismic event simulations have been performed in different parts of the world in order to assess the performance of ground support systems since 1969 (Andrieux et al. 2005, Ansell 2004, Archibald et al. 2003, Espley et al. 2002, Hagan et al. 2001, Heal 2010, Heal and Potvin 2007, Heal et al. 2005, Ortlepp 1969, Tannant et al. 1994, Tannant et al. 1995, Ortlepp 1992). Different blast layouts (e.g. blasthole angle and burden) were used by the different researchers based on the objective of their tests. Different levels of success in obtaining the desired amount of damage to the rock support /rock mass were observed.

Stacey (2012) reviewed in-situ testing methods of rock support. He divided the design of the tests into the categories direct and indirect testing methods. Direct testing of rock support is considered as a severe test of rock support where the damage to the support system is directly due to the blast. In this method, the gas pressure has a strong influence which makes such a test very different from a rockburst. The test carried out by Ortlepp (1992b) was an example that falls within this testing category. Ortlepp (1992b) compared the damage from a real rockburst that occurred close to the site where the test was conducted. He concluded that the damage produced in a direct test is indistinguishable from the rockburst damage. Stacey (2012) concluded that, the results provided by this method should be compared to documented real rockburst damage, perhaps using a damage classification approach.

In indirect blasting tests the issue of gas pressure is almost completely solved by increasing the burden (Stacey, 2012). This type of test has been considered as the closest simulation of rockbursts. However, there are still questions that need to be resolved such as the difference in wave interaction, wave frequency, source mechanism, source location, and source magnitude. Further issues indicated by Stacey (2012) are that the tests are not repeatable at one site, they are too costly, and too inconvenient in an underground mine environment to be practical for comparison of the performances of different support systems.

A summary of some earlier tests with direct, indirect or a combination of them is presented in this section. In the tests carried by Andrieux (2005) and Tannant (1994) angled blastholes were drilled into the wall of the test drift (Figure 1). The main objective was to separate the effect of waves from the detonation gases. The angled geometry of the blastholes provided (according to Andrieux et. al (2005)) an easier path for the detonation gases to vent backwards. Only the part of the wall close to the collar was destroyed and small signs of damage were observed at the charged segment of the blasthole in all tests except in the second trial where the whole test wall was destroyed. Tannant (1994) described that the blasthole relative to the test wall in their tests was designed to generate a variation in the blast loading intensity along the tested wall.

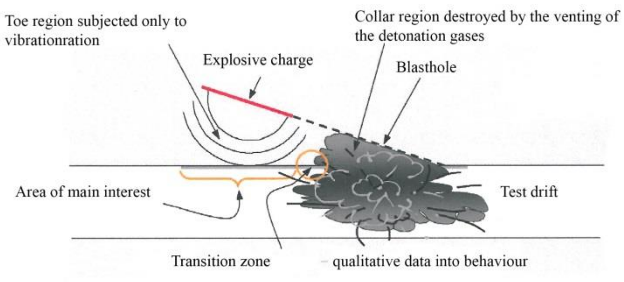


Figure 1 Schematic presentation of wave and gas separation (Andrieux et al. 2005)

In the tests conducted in South Africa (Hagan et al. 2001) a large burden of 5 m was considered to minimize the effects of expanding gases. Figure 2 shows a picture of the damage to the sidewall and the volume of the blocks ejected after the simulated rock burst. Reddy and Spottiswoode (2001) showed that the shape of the ejected blocks was determined by the pre-existing bedding separations and stress fractures. Post blast observations indicated that no direct gas expansion had affected the ejection of the rocks (Hagan et al. 2001).

Figure 2 Ejection of blocks of rock after simulated blast (Hagan et al. 2001)

In the tests conducted by Heal and Potvin (2007), three blasts were conducted at one site, with a burden of 5 m, 4 m, and 3 m in the first, second and third blast, respectively. Each blasthole was separately charged and detonated to allow successively larger dynamic loading upon the test wall. In these tests, successive blasts were designed to reproduce an actual seismic event as closely as possible.

Figure 3 Before and after blast image captured by high speed camera indicating the damage to the tested support system (Heal and Potvin 2007)

2 Tests 1 to 5

2.1 Design

Considering the mentioned difficulties and uncertainties associated with direct and indirect blasting methods including the penetration of explosive gases into the burden, the issues with the differences in loading conditions and source mechanism, and the cost of the tests which are all believed to be common in both approaches, it was decided to design the tests in the Kiirunavaara mine to possibly overcome some of the abovementioned problems. The aim was to design the tests so the rock support system could be tested with one blast and to ensure that the loading of the rock support was caused by seismic waves and thus the effect of the blasting gas had to be minimized. For this purpose, one blasthole was drilled in the pillar parallel to the test wall and four holes were drilled into the burden to measure the gas pressure in the burden.

A high VOD explosive type, NSP711 with VOD around 7900 m/s, was selected for the tests. The reason for selecting this type of explosive was the lower amount of gas production compared to commercial explosives. The only exception was Test 3 in which bulk emulsion was used.

In Test 1, the blast was designed to generate a magnitude +3 event (Richter scale) located 15 m from an opening. This would result in PPVs in the range of 1.5 m/s to 3.5 m/s according to the PPV – magnitude – distance relationship by Kaiser et al. (1996). The design magnitude was similar to that of the largest seismic event that had occurred in the Kiirunavaara mine until 2010 (Malmgren 2010). The burden and blasthole diameter in Test 1 were determined based on the experience from earlier studies in the Kiirunavaara mine by Olsson et al. (2009). This resulted in a theoretical burden of 3.5 m and a blasthole diameter of 115 mm. Two different charge concentrations, dc_1 = 76 mm and dc_2 = 45 mm, denoted as “high charge segment” and “low charge segment”, respectively, were used. The low charge segment was the first 5 m of the blasthole measured from the toe. The high charge segment was the subsequent 5 m of the blasthole. This was done in order to investigate the support performance under two different levels of energy during one test and reduce the number of trials in the search for the optimum blast design. The total length of the blasthole was 15 m. The last 5 m of the blasthole, from the high charge segment to the collar, was left empty.

The blast in Tests 2 to 5 were designed based on the observed results in the earlier tests. The burden in all these tests was kept constant while the amount of explosive was changed based on the experienced level of damage in previous test. Figure 4 shows a schematic plan view of the test sites and the blast design in Tests 1 to 5.

2.2 Rock support and instrumentation

The rock support system used in all tests comprised 100 mm steel fibre reinforced shotcrete (40 kg/m^3 steel fibre), 75 mm × 75 mm welded mesh with 5.5 mm diameter, and Swellex Mn24 rockbolts with a length of 3 m and 1 m spacing.

The instrumentation was designed to provide data for different objectives. As an example of the instrumentation, the monitoring layout in Test 1 is shown in Figure 5. The exact number of monitoring instruments in each test is summarized in Table 1. The reason for not using monitoring instruments in Test 3, 4 and only a few instruments in Test 5 was to speed up the process of finding the optimal burden and charge concentration.

The instrumentation was designed to provide data to assess the performance of the rock mass and the rock support system. For this purpose, uniaxial shock accelerometers, type PCB 350 B03, were used to measure the acceleration and to calculate the velocity and the displacement. Observation boreholes, inspected by Robicam 37 before and after the blast, were used to measure the depth of the damage created behind the surface of the test walls.

High-speed cameras (Casio EX-F1) were used in each test to record the response of the tested walls and to estimate the ejection velocity where possible.

Laser scanning (Leica HDS 6000) was conducted before and after each blast to measure the residual displacement of the test walls due to dynamic loading. The scanning also provided information on the deflection of the reinforced shotcrete. The measured deflections were then used to estimate the energy absorption by the reinforced shotcrete.

In some of the tests, a laser sensor type Sick OD Value Sensor was placed in the middle of the cross-cut to record the displacement versus time of the test walls during the blast. The aim was to study the dynamic response of the tested walls under different designs.

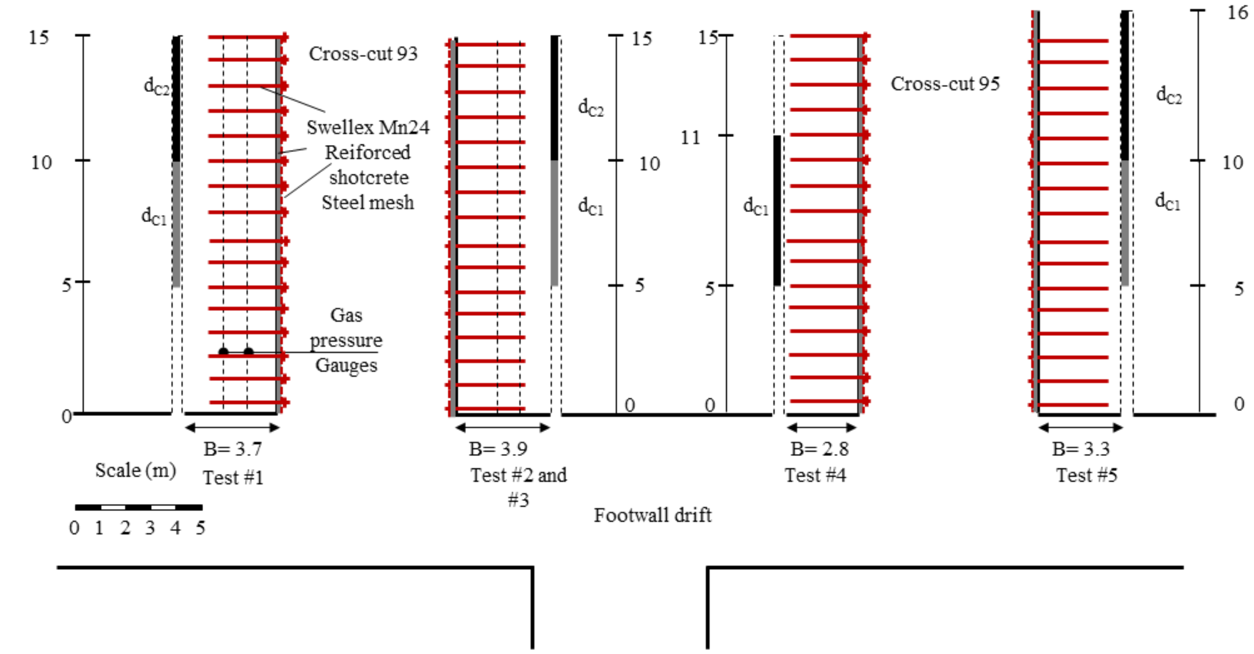


Figure 4 **Blast design in Tests 1, 2, 3, 4 and 5 (Shirzadegan et al. 2016b)**

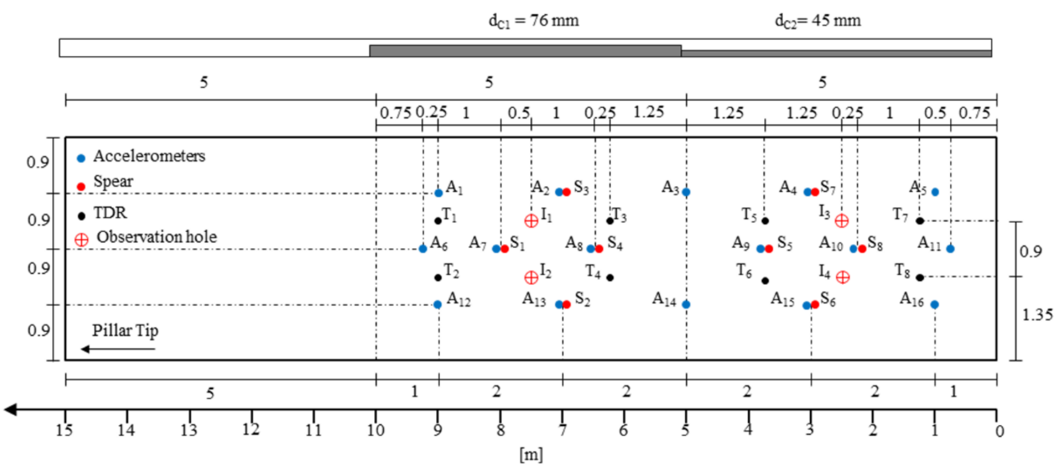


Figure 5 **Monitoring instruments lay out in Test 1** (Shirzadegan et al. 2016b)

Table 1 Summary of the number of monitoring instruments in Tests 1, 2, 5, 6, and 7

Test	No. of accelerometers	No. of observation boreholes	No. of laser sensor	Laser scanning	Gas pressure sensor
1	16	4	-	Yes	Yes
2	24	4	2	Yes	Yes
5	3	-	1	Before blast	No
6 crosscut 100	16	11	1	Yes	No
6 crosscut 103	5	9	1	Before blast	No
7 crosscut 100	16	11	-	No	No

2.3 Results

In Test 1, the post-blast damage mapping showed a few discrete cracks in the shotcrete but no damage to the mesh or the rockbolts was observed. The range of the measured PPV recorded by the accelerometers was between 1 and 6.7 m/s. The depth of the created fractured zone behind the shotcrete measured by the borehole camera was 0.3 to 0.4 m. The kinetic energy transmitted to the rock support was estimated to be in the range of 6.0 - 20 kJ/m^2.

The level of the input energy for Test 2 was increased by 50% compared to Test 1. Test 2 created larger number of, longer and wider cracks than in Test 1 and the cracks were distributed all over the tested wall. However, no damage of the rockbolts was observed despite the high PPV (up to 7.5 m/s measured by accelerometers). The depth of the created fractured zone was 0.5 m and the transmitted kinetic energy was in the range of 2.0 - 30 kJ/m^2.

Test 3 was carried out at the same location as Test 2 using the same blasthole. Therefore the burden was already fractured due to the blast in Test 2. Unlike the rest of the tests, the blasthole in Test 3 was filled with bulk emulsion. A complete destruction of the burden occurred in this test. The broken rock pieces were ejected and thrown to the other side of the cross-cut. Mapping of the rock pile showed that some of the ejected rockbolts did not show severe damage probably indicating that the distance from the tested wall surface to the release surface of the burden was greater than the length of the bolts (Figure 6a). Some of the rockbolts were severely damaged, mainly either deformed at 2 or 3 cross-sections along their 3.0 m length or broken into pieces of lengths from 0.2 to 3.0 m. The broken rockbolts showed signs of shear failure at the failure surface. The face plates were in a few cases detached, in some cases severely deformed indicating that they had been highly loaded, and in some cases remained undamaged. However it was difficult to judge with high certainty that the observed damage to the rockbolts was due to the blast load, gas pressure or a result of becoming broken under the rock pile after ejection.

In Test 4 the input energy was increased by 10% compared to Test 2. The burden was the same as in the earlier tests. The increase of charge concentration resulted in a complete destruction of the burden. Figure 6b shows the damage to the rockbolts in Test 4. The charge concentration for Test 5 was in between that of Tests 2 and 4 (dc_1 = 94 mm and dc_2 = 83 mm in a 16 m long blasthole). Complete destruction of the burden occurred in this test. The damage observed to the rockbolts in Test 4 and 5 was similar to that observed in Test 3.

A relatively high pressure (about 800 kPa) was recorded by one of the gas pressure sensors (approximately 1 m from the blasthole) in Test 1, 34 ms after the initiation of the detonation and a low pressure was recorded by the rest of the sensors in both Tests 1 and 2.

a)

b)

Figure 6 Severe damage to the rock support in tests 3 and 4

3 Analysis of the tests and improved blast design

3.1 Tests 1 to 5

Tests 2 and 5 were numerically analysed using a combination of LS-DYNA and UDEC (Zhang et al. (2013). The LS-DYNA was used to simulate the detonation stage of the blast. The velocity-time history calculated by LS-DYNA was applied as an internal boundary condition in UDEC. The simulation of Tests 2 and 5 resulted in thin yielded areas parallel to the tested walls near the free face and a large number of thin radial yielded areas from the crushed zone boundary created around the blasthole, forming a large cone-shaped volume. The conically shaped rock mass moved towards the cross-cut, with extensive yielding occurring along and within this volume. The results from the numerical analysis of Test 5 were quite representative of the conditions after Test 5 in which the test wall and part of the pillar were completely destroyed. The conclusion of Tests 1, 2, 3, 4 and 5 was that the loading condition needs to be improved to avoid the creation of radial cracks and the conical failure volume and to promote the generation of dominantly wall-parallel fractures mimicking the damage caused by a planar seismic wave.

3.2 Improved blast design (Test 6)

Numerical simulations with larger burdens, i.e., 5 m and 8.5 m (larger than in Tests 1, 2, 4 and 5 and the largest possible at the test site), were therefore conducted by Zhang et al. (2013). The number of blastholes was also increased to two in this analysis with the aim to generate a wave with more planar front, and increase the strength of the designed blast to develop a larger failure zone behind the test walls compared to earlier tests. The results from the numerical analyses showed that fractures parallel or sub-parallel to the wall surface were created without forming any cone-shaped volume for both 5 m and 8.5 m burden. Based on the results from the numerical analysis it was decided to use a burden of 8.5 m for Test 6. Therefore, two blastholes with zero delay and a designed burden of around of 8.5 m was proposed in order to generate a wave front which is sub-planar wave to the surface of the tested wall and to reduce the negative effect of radial fracturing, and increasing the strength of the blast. Figure 7 shows the design specifications in Test 6. The blastholes were drilled in the middle of the pillar between cross-cuts 100 and 103. The tested wall in cross cut 100 was supported by the support system mentioned in section 3 while the tested wall in cross-cut 103 was supported by only plain shotcrete.

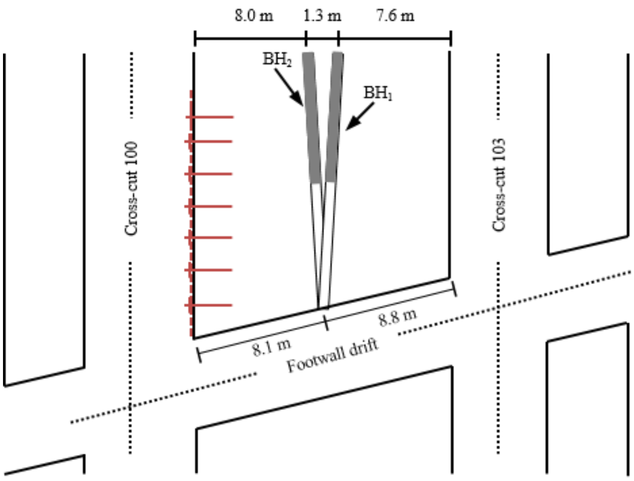


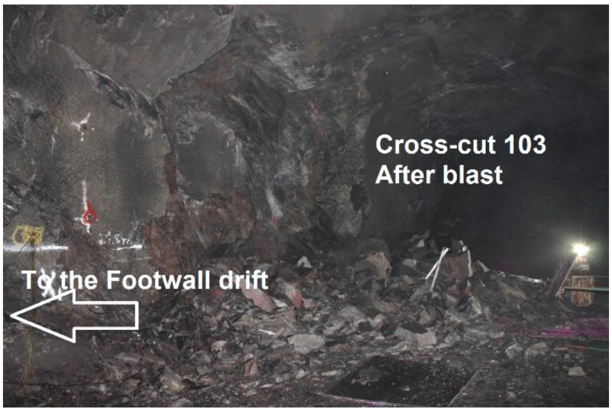
Figure 7 Schematic view of the blast design in Test 6 (Shirzadegan et al. 2016a)

3.3 Results

The lower blasthole did not detonate during Test 6. No major damage to the rockbolts and mesh was observed in cross-cut 100, after the blast. The shotcrete, showed a few very fine cracks. Clearly evident damage to the surface support, i.e., debonding of the shotcrete from the rock over an area of roughly 1 m x 1 m was observed at the end of the test panel (farthest from the footwall drift, see Figure 8a). However, the damaged material was kept in position by the mesh. Farther into the cross-cut, the areas not covered with the dynamic rock support, showed ejection of wall material with a depth of up to 30 cm. The range of the measured PPV in this cross-cut was between 1.3 and 4 m/s. The depth of the created fractured zone behind the shotcrete measured by borehole camera was 0.3 to 0.6 m. The maximum transmitted kinetic energy in this cross-cut was 10 kJ/m^2.

Cross-cut 103 failed to be functional and blocks of rock were ejected from the tested wall by the blast (Figure 8b). The large pieces of rock and shotcrete lying in the middle of the cross-cut were ejected from a location of 1.5 to 2 m above the floor and travelled a horizontal distance of about 2 m. The failed thickness per square meter of the test wall (range 0.1 – 0.8 m) was measured by direct observation after removal of the rock piles from the cross-cut. Based on analysis of the video recorded by a high speed camera, an ejection velocity of 3.6 - 8.1 m/s was estimated in this cross-cut. The maximum transmitted kinetic energy in this cross-cut was 29 kJ/m2.

a)



b)

Figure 8 Damage to cross-cut 100 after (Shirzadegan et al. 2016a)

3.4 Test 7

Since the lower blasthole did not detonate in Test 6, it was decided to recharge the upper blasthole and blast both holes a second time (Test 7). A similar number of accelerometers were installed in cross-cut 100. Post blast observations indicated that the number of cracks on the surface of the shotcrete increased. No significant damage to the rockbolts and weld mesh was observed. Lower range of particle velocity compared to Test 6 was recorded by the accelerometers (1-3 m/s). This could be due to the fact that the burden was fractured by the blast in Test 6.

4 Rock support performance

4.1 Swellex Mn24

The deep installed accelerometers, 1.5 m from the surface of the shotcrete, at each charge section in Test 2 were used to estimate the performance of the rockbolts. The Swellex rockbolts closest to these accelerometers were selected for analysis of energy absorption versus displacement. A maximum elongation of 80 mm was estimated for the Swellex Mn24 in Test 2. The results from a series of dynamic laboratory tests on Swellex Mn24 by Voyzelle et al. (2014) was used to estimate the amount of absorbed energy by the Swellex rockbolts in Test 2. The estimated elongation was used in combination with the energy absorption curves by Voyzelle et al. (2014). This resulted in an energy absorption of around 17 kJ for the Swellex MN24 for an elongation of 80 mm.

4.2 Fibre reinforced shotcrete

At the points of the tested wall where rockbolts were installed, the relative displacement between the rockbolt and its surrounding surface support was calculated to estimate the maximum residual deflection of the surface support after the dynamic loading. The displacements of the rockbolt and surface support were obtained from the laser scanning of the test wall before and after the blast. The measured deflection of the shotcrete for each square meter of the tested wall was used to estimate the energy absorbed by the reinforced shotcrete and the weld mesh in Tests 1, 2 and 6 (cross-cut 100). The corresponding absorbed energy per square meter of the tested walls was then estimated by using the deflection-energy absorption curves by Thyni (2014) from a series of laboratory tests of reinforced shotcrete and weld mesh at the LKAB Kiirunavaara mine. Figure 9 shows the energy absorbed by the reinforced shotcrete and weld mesh per square meter of the test wall in Test 2. The range of energy absorption in the mentioned tests is:

- Test 1: 0.1 – 0.8 kJ/m2
- Test 2: 0.2 – 4.0 kJ/m2
- Test 6: (cross-cut 100): 0.3 – 0.8 kJ/m2

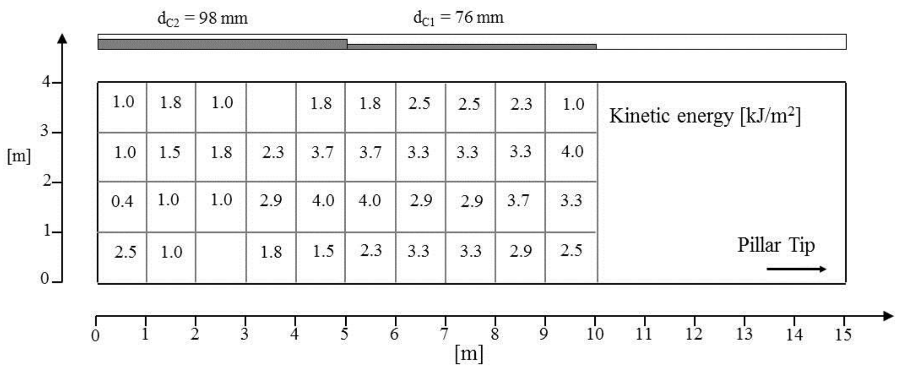


Figure 9 Energy absorbed by reinforced shotcrete and weld mesh in Test 2 (Shirzadegan et al. 2016b)

4.3 Support system

The maximum calculated kinetic energy of the ejected rock (29 kJ/m^2) in cross-cut 103, according to the classification by Kaiser et al. (1996) , falls within the range of very high level damage intensity. As the blasthole in Test 6 was drilled almost in the middle of the pillar it could be expected that the blast will result in similar damage intensity in cross-cuts 100 and 103 if the geology is the same and the same support is used. Cross-cut 103 was supported with plain shotcrete while cross-cut 100 was supported with a dynamic rock support. Therefore, Test 6 provided an opportunity to compare the damage intensity and the recorded PPV (by accelerometers) for an unsupported cross-cut and a cross-cut supported by Swellex Mn24 + fibre reinforced shotcrete + weld mesh. The generated average PPV 0.2 m behind the surface of the tested wall in cross-cut 100 where the dynamic rock support system was installed was up to 1.5 times lower than that in the unsupported cross-cut 103. The maximum transmitted kinetic energy to the fractured zone in the test wall in this supported cross-cut (100) was estimated to 10 kJ/m^2. Except minor cracks visible on the surface of the shotcrete, no sign of damage jeopardizing the supporting capacity was observed. Based on the kinetic energy measurements and site investigations after the blast in the two cross-cuts, it can be concluded that, the combination of Swellex Mn24, reinforced shotcrete and weld mesh can mitigate the consequences of a seismic event where the very high level of damage intensity by rock ejection is expected.

5 Discussion

Tests 3, 4 and 5 resulted in severe damage to the support system. The burden was completely destroyed, and the rockbolts were ejected and exposed. The numerical analysis revealed that using high amount of explosives and a burden of 2.5 m to 3.5 m has resulted in tangential tensile stresses exceeding the tensile strength of the rock and a reduction of the radial stresses close to the wall of the cross-cut.

In Test 6, the burden was increased in order to generate a wave front which was sub-planar to the surface of the tested wall. The improvement of the design was confirmed by the observed damage to the tested wall in cross-cut 103, i.e., ejection of rock slabs with a thickness of up to 80 cm. The whole 8.5 m burden was not destroyed despite the fact that this cross-cut was not supported with the dynamic rock support (only plain shotcrete was sprayed).

In Tests 1, 2, 6 and 7, despite the low level of damage to the support, more quantified data was produced compared to that in Tests 3, 4 and 5. This included, the PPV, the depth of damage, and the estimated kinetic energy transmitted to the support system. Furthermore, the tests provided calibration data for use in numerical models. The tests also provided the possibility to interpret the performance of the rock support in Tests 1, 2 and 6. This was done by linking the data obtained from these tests to the data provided from laboratory dynamic tests conducted on the rockbolts and surface supports.

In Test 6 the damage to the tested wall in cross-cut 103 was similar to that which occurs in a rockburst with high damage intensity (according to classification by Kaiser et al. 1996). As the blasthole was drilled in the middle of the pillar in this cross-cut, it could be expected that the test wall in cross-cut 100 was similarly loaded. Therefore it can be concluded that, the support system in cross-cut 100 has withstood the dynamic loading conditions. It can be concluded that, the installed support system can limit the damage and enhance the safety of the excavation where high damage intensity from a rockburst is expected. It is believed that this method will provide better opportunity to compare different support alternatives that could be used at a mine site under seismic loading conditions.

One of the goals for the large scale testing of rock support was to assess the energy demand on the tested support system for the design purposes. The tests conducted in the Kiirunavaara mine provided the opportunity to study the response of the test wall under dynamic loading and evaluate the performance of the installed rock support. It was decided to not continue with the tests after Test 7. Therefore no design data was provided from this testing series.

6 Conclusion

This paper presented a summary of the results from the development of a large-scale dynamic testing method of rock support systems by using blasting to generate the dynamic load. The major conclusions of this work are:

- The influence of gas expansion on the test results can be reduced by using high impact, low gas explosives, avoiding stemming the blasthole, and designing a reasonable burden.
- Evidence of damage showed that the new blast design (Test 6) was successful in generating sub-planar waves with tangential tensile stress levels lower than the tensile strength of the rock mass. Thus it can be concluded that no radial cracks with an extension from the blasthole to the tested wall surface was created.
- The installed support system (in Test 6) showed that it has the capacity to perform well in the mining areas where very high ejection conditions are expected.
- The large amount of data recorded during these trials will be useful for the calibration of more advanced numerical models. The numerical analyses can then be used for sensitivity analyses simulating different blast designs.

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