DEVELOPMENT OF A METHODOLOGY FOR IN-SITU DYNAMIC TESTING OF GROUND SUPPORT

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Summary

A series of seven large scale dynamic tests were conducted at LKAB Kiruna mine using explosives in the vicinity of cross-cuts to generate dynamic load on the support system. The aim was to develop an in-situ testing method for rock support, i.e., to determine the dynamic load that causes failure to the test wall and/or support system. The methodology used to design Tests 1 to 7 is discussed in this paper and the level of damage to the test wall and support system in each test is described. Comparison of results in different test designs indicated that increasing burden and number of blasthole at the same time, increases the possibilities of obtaining more planar waves and decreases the destructive effect of detonation gases.

1 Introduction

The conventional design approach of rock support essentially consists of (i) the identification of potential failure modes and (ii) a comparison of the available capacity with the driving force/demand (including dynamic components). By calculating the factor of safety or the probability of failure, the demand on the rock support can be estimated. Unfortunately, it has been concluded that it is impossible to design support systems under seismic loading conditions by using this approach, since neither the
demand on a support system nor the capacity of a support system can be satisfactorily defined (Stacey 2012).

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 2008).

Within the framework of a research program focused on deep mining problems at Luleå University of Technology, Division of Mining and Geotechnical Engineering, in-situ dynamic tests of rock support using blasting as the seismic source were conducted in the Kiirunavaara underground mine, owned and operated by Luossavaara Kiirunavaara Aktiebolag (LKAB). The main purpose of the tests was to develop an in-situ testing method for rock support and to obtain quantitative data for modelling.

Similar large scale simulations have been performed in other parts of the world in order to assess the capacity of ground support systems (Andrieux et al. 2005; Ansell 2004; Archibald et al. 2003; Espley et al. 2002; Hagan et al. 2001; Heal and Potvin 2007; Ortlepp 1992; Tannant et al. 1994; Tannant et al. 1995). 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. In the tests conducted at LKAB Kiirunavaara mine, the crucial issue for success was the design of the blast in order to generate waves which in some way imitated the characteristics of the waves from a real seismic event. Another issue was to reduce the destructive effects of expanding gases generated by the blast.

2 Test site

Adjacent pillars between the cross-cuts in the completed production block 9 on the 741 m level were chosen for the tests. All of the tests were planned to be conducted at the chosen site because of (i) No mining activity was taking place at that level, (ii) The pillars were only supported with plain shotcrete and sporadic Kiruna bolts which can be assumed to have limited supporting effect when exposed to seismic loading, (iii) Many cross-cuts with similar rock mass conditions were available for further tests and (iv) Comprehensive geological investigations had been done in the area.

The rock types in the test area have traditionally been referred to as syenite porphyries, including a nodular variety (Geijer 1910), mainly consisting of trachytes to trachyandesites (Ekström and Ekström 1997) of variable character and degree of alteration. The rock mass in the test area was very blocky and the geological strength index (GSI) values were estimated to lie mostly within the range of 40 – 50, with joint quality from good to acceptable (Andersson 2010).
3 Design: Tests 1 - 5

The blast in Test 1 was designed to mimic a seismic event with magnitude +3 (Richter scale) located 15 m from the drift. This resembles the largest seismic events that have occurred in the Kiirunavaara mine (Malmgren 2010) which caused serious damage to the rock mass and the rock support. The Peak Particle Velocity (PPV) was chosen as the quantity characterizing the seismic event. The maximum PPV was calculated using a PPV – magnitude – distance relationship by Kaiser et al. (1996). This resulted in PPVs which were in a range of 1.5 m/s to 3.5 m/s. The initial estimation of the amount of explosive, blasthole diameter, and burden for the first trial was based on experience from earlier studies in the Kiirunavaara mine (Olsson et al. 2009) which resulted in a theoretical burden of 3.5 m. Burden in Tests 2, 4 and 5 was kept constant, but the amount of used explosives in later tests was designed based on the results observed in the previous tests. Test 3 is not described in this paper since the initial condition of the burden and type of explosive in this test was different from that in other tests.

The tests were conducted either in the left or the right hand sidewall of the cross-cuts (Fig. 1). The blasthole, with an approximate length of 15 m, was drilled parallel to the cross-cut from an adjacent footwall drift. The burden varied along the tested sidewall due to the deviation of the blasthole orientation combined with an irregular profile of the tested wall surface. Therefore, only the average or effective burden is illustrated in Fig. 1. Two different charge diameters, each with a length of around 5 m, were used in Tests 1, 2 and 5 to reduce the number of tests. The first 5 m of the blasthole were not charged nor stemmed to vent the gas and reduce the gas pressure. The next 5 m + 5 m of the blasthole were charged with two different charge densities in Tests 1, 2 and 5. Only one charge density was used in Test 4. The area of the cross-cut wall in front of the higher charge diameter is denoted “high charge segment” and the area in front of the lower charge diameter is denoted “low charge segment” in this paper. The blasthole, and charge characteristics and the effective burden in each test are summarized in Table 1.

The explosive selected for the tests in the Kiirunavaara mine was a military type, NSP711, with a measured velocity of detonation (VOD) of 7931 m/s and a density of 1500 kg/m³. The reason for selecting this type of explosive was the lower amount of gas production compared to commercial explosives, high VOD and a blasthole pressure resulting in more wave energy than gas expansion, a better control over the amount of explosives, and the well-known Jones-Wilkins-Lee (JWL equation of state) parameters for numerical analysis (Helte et al. 2006). In all of the tests, the tested rock support was consisting of 100 mm steel fibre reinforced shotcrete (40 kg/m³ steel fibre), 75 mm × 75 mm weld mesh with 5.5 mm diameter, and Swellex rockbolts with a length of 3 m and 1 m spacing.
Fig. 1 Blast design in Tests 1 to 5. In this figure, $d_C$ is the diameter of the concentrated explosive material.

### Table 1 Summary of burden, blasthole and charge dimension in Tests 1 - 5

<table>
<thead>
<tr>
<th>Test</th>
<th>Average burden (m)</th>
<th>Diameter (mm)</th>
<th>Length (m)</th>
<th>Decoupling ratio (%)</th>
<th>Charge concentration (kg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.7</td>
<td>blasthole</td>
<td>115</td>
<td>15</td>
<td>...</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C1}$</td>
<td>76</td>
<td>5</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C2}$</td>
<td>45</td>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>3.9</td>
<td>blasthole</td>
<td>152</td>
<td>15</td>
<td>...</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C1}$</td>
<td>76</td>
<td>5</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C2}$</td>
<td>98</td>
<td>5</td>
<td>66</td>
</tr>
<tr>
<td>4</td>
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<td>152</td>
<td>15</td>
<td>...</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C1}$</td>
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<td>6</td>
<td>79</td>
</tr>
<tr>
<td>5</td>
<td>3.3</td>
<td>blasthole</td>
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<td>16</td>
<td>...</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C1}$</td>
<td>94</td>
<td>5</td>
<td>62</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$d_{C2}$</td>
<td>83</td>
<td>6</td>
<td>55</td>
</tr>
</tbody>
</table>

4 Field tests results: Tests 1 - 5

Post-blast observations of the tested support system in Test 1 showed that cracks with a width of up to 5 mm and a length of 2 m to 3 m were created on the surface of the reinforced shotcrete mainly within the high charge segment ($d_{C1} = 76$ mm) (Fig. 2 (a)).
No obvious damage to the rockbolts or the mesh was observed. The event magnitude of the test recorded by the mine seismic system was $M_L = 0.7$ on the local magnitude scale.

Observations in Test 2 showed that cracks with widths of up to 15 mm and 2 m to 3 m in length were formed within the high and low charge segments ($d_{C2} = 98$ mm and $d_{C1} = 76$ mm, respectively), see Fig. 2 (b). No obvious damage to the rockbolts or the mesh was observed and a local event magnitude of $M_L = 1$ for the test was recorded by the local seismic system.

Completely different results were observed in Tests 4 and 5 compared to those in Tests 1 and 2. In Tests 4 and 5 the burdens were completely destroyed (Fig. 2 (c) and Fig. 2 (d)). The ejected rock material in Test 4 was broken into rather small pieces as a result of a high charge concentration ($d_{C1} = 120$ mm) (note that only one charge segment was used in this test), while in Test 5, the burden was broken into large blocks of rock at both charge segments. The mesh and the rockbolts had totally lost their functionality in both of these tests. Failure mapping of the rockbolts in Test 4 was performed and the results indicated that most of the rockbolts were cut into pieces of 1 – 2 m of length. In 95% of the cases the face plates were detached. The local event magnitudes for Test 4 and Test 5 were $M_L = 0.8$ and $M_L = 0.9$, respectively.

![Fig. 2](image)

**Fig. 2** Level of damage in (a) Test 1, (b) Test 2, (c) Test 4 and (d) Test 5

### 5 Design: Tests 6 and 7

Tests 2 and 5 were numerically analysed by Zhang et al. (2013). The simulation of the tests resulted in the expected fractures parallel to the tested wall near the free face and a large number of radial fractures propagating from the crushed zone created around the
blasthole, forming a large cone-shaped volume (Fig. 3). 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.

Fig. 3  Numerical analysis of Tests 2 and 5 (Zhang et al. 2013)

Numerical simulations with burdens of 5 m and 8.5 m (larger than in Tests 1 – 5), and two blastholes with diameters similar to that in Test 2 were also conducted by Zhang et al. (2013) (Fig. 4). This was done to determine how to avoid the creation of radial cracks and the conical failure volume and to induce dominantly wall-parallel fractures mimicking the damage caused by a planar seismic wave. The results from the numerical analyses of both 5 m and 8.5 m burden indicated that the fractures parallel or sub-parallel to the wall surface were created without forming any cone-shaped volume. This will also affect the propagation of the detonation gases as the radial fractures were providing access to the near surface rock mass. The absence of radial fractures will therefore make the rock mass less conductive to gas transport. Therefore two blast holes with zero delay and a designed burden larger than 5 m was proposed in order to generate a sub-planar wave and reduce the negative effect of radial fracturing.

Fig. 4  Numerical analysis of Test 2 with burdens of 5 m and 8.5 m (Zhang et al. 2013)
Based on the results from the numerical analysis, it was decided to use two blastholes in Test 6. These holes (diameter 152 mm) were drilled in the middle of the pillar between cross-cuts 100 and 103, and charged with NSP711 (diameter 120 mm) (Fig. 5). No stemming material was used to allow venting and reduce the effect of detonation gases. The burden varied along the tested sidewall due to the deviation of the blasthole combined with the irregular profile of the tested panel, therefore, only the average or effective burden, $B_{ave}$, is shown in cross-section A-A in Fig. 6.

In cross-cut 100, the test wall was supported by 100 mm fibre reinforced shotcrete (40 kg/m$^3$ steel fibre), 75 mm × 75 mm weld mesh with 5.5 mm diameter and Swellex Mn24 rock bolts in a 1 m x 1 m pattern, while the wall in cross-cut 103, was only supported by the plain shotcrete (the existing support when the tests started). This provided the opportunity to compare the reaction of fully supported (Swelllex + shotcrete + mesh) and shotcrete supported surfaces under the same dynamic loading condition. The length of the supported wall was 20 m.

![Fig. 5 Location of blasthole with respect to the test wall in Test 6 [Unit: m]](image1)

![Fig. 6 Effective burden in tests 6 and 7 [Unit: m]](image2)
6 Field test results: Tests 6 and 7

6.1 Test 6: cross-cut 100
In Test 6, only the upper blasthole (BH1) detonated. No major damage to the rockbolts and mesh was observed in cross-cut 100, after the blast. The shotcrete, however, showed a few new and very fine cracks (Fig. 7). 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). However, the damaged material was kept in position by the mesh. Farther into the cross-cut, in the areas not covered with the rock support system installed for the test, the wall showed signs of ejection of material with a depth of up to 30 cm (Fig. 8).

Fig. 7 Test wall in cross-cut 100. The location of damages are highlighted

Fig. 8 Unsupported areas of the wall of the cross-cut 100
6.2 Test 6: cross-cut 103

In cross-cut 103 the wall failed to be functional and blocks of rock were ejected from the wall. Fig. 9a shows the damage associated with rock ejection during the simulated seismic event. The large pieces of rock and shotcrete lying in the middle of the cross-cut were ejected from a location 1.5 to 2 m above the floor and travelled a horizontal distance of about 2 m. Fig. 9b shows the state of the cross-cut after removal of the rock piles. The thickness of the ejected rock was measured by direct observations after removal of the rock piles from the cross-cut. This indicated a thickness of 0.2 to 0.8 m of ejected rock from the tested wall in this cross-cut.

6.3 Test 7: Cross-cut 100

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. Post blast observations indicated that the number of cracks on the surface of the shotcrete increased. The area at the end of the installed support, farthest from the footwall drift, which showed damage to the shotcrete after Test 6 (debonding over a 1 m x 1 m area) was further damaged by Test 7 (Fig. 10a). No significant damage to the rockbolts and weld mesh was observed. The parts of the wall of the cross-cut (outside the supported test wall) showed larger areas of ejection compared to Test 6 (Fig. 10b).
7 Discussion of results

In Tests 1, 2, 4, and 5 the charge concentration was increased in a step by step order to determine the critical charge density resulting in damage to the support system. However, the results indicated that this was not a successful method. The increase of charge concentration in Tests 2, 4 and 5 was decided based on the results and the level of damage observed in the previous test. The increased charge concentration in Test 4 resulted in a complete destruction of the burden. In Test 5, the charge concentration was in between that of Tests 2 and 4. Also in this test complete destruction of the burden was obtained. This can be attributed to the effect of the burden. In Tests 1 – 5 the primary aim was to obtain a burden of around 3.5 m. However, due to practical drilling issues there was a variation in burden in the range 2.8 – 3.9 m. This effect was observed in the results obtained in Tests 1 and 2 with burden in the range of 3.7 – 3.9 m and lower charge concentration resulting in minor damage to the support system compared to that in Tests 4 and 5. In Tests 4 and 5 the burden was in the range of 2.8 – 3.3 m burden and the charge concentration was higher which resulted in the destruction of the burden. One possible explanation for the complete destruction of the burden in Tests 4 and 5 was addressed by Zhang et al. (2013) who carried out numerical back analysis of Test 5. The analysis revealed that using high amount of explosives and a burden of 2.5 m to 3.5 m resulted in tangential stresses exceeding the tensile strength and a reduction of the radial stresses close to the wall of the cross-cut, i.e., a more conical wave.

The idea behind increasing the burden and number of blastholes in Test 6 (compared to that used in Tests 1 to 5) was to (i) generate waves which were sub-parallel to the surface of the cross-cut sidewalls and (ii) avoid wave-induced tangential stresses which exceeds the tensile strength of the rock mass (resulting in tangential fractures) close to the boundaries of the cross-cut. If this objective is reached no conical-shaped damaged rock mass volume will be formed thus avoiding complete destruction of the burden and the tested wall as was observed in Tests 4 and 5. The lack of tangential fractures also reduces the effect of the gas expansion to reach the tested wall. This means that the destructive effect of the gas expansion in both of the cross-cuts 100 and 103 was limited. Furthermore, the new blast design also generates a wave front which is more similar to that from a real seismic event, i.e., a sub-planar wave front.

The post blast investigation in Test 6 (and also in Test 7), revealed that the part of the test wall farthest from the footwall drift had been damaged by the dynamic loading over an area of 1 x 1 m. One explanation for this difference in behaviour between the part of the panel farthest from the footwall drift and the rest of the panel can be the fact that the damaged part of the wall is in a transition zone. The supported tested panel ends close to the damage.
8 Conclusion

The conclusions from the large scale tests reported in this paper can be summarized as follows:

- The results indicated that the burden is crucial when designing in-situ dynamic tests of rock support and that the burden should be larger than 5.0 m.

- The type of damage obtained in cross-cuts 100 and 103 indicated that the new blast design 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, the radial cracks extending to the vicinity of the tested wall surfaces were not created and the effects of the blast gases were also kept at a low level.

- By comparing the results in cross-cuts 100 and 103 it can be concluded that, part of the energy carried by the incident wave will be:
  - absorbed by the installed support (during the fracturing of the shotcrete, sliding of the rockbolt and yielding the mesh),
  - absorbed during the fracturing of the rock mass,
  - and, part of the incident wave energy will be reflected back to the surrounding rock mass.

- The results of the tests (Test 1 – 7) provided a testing method for assessment of the performance of a support system in a scale similar to that in a real mining situation. The method also provides important calibration input data for numerical analyses of the behaviour of rock support systems. The calibration data provided in Tests 6 and 7 can be used specifically to gain more insight into the question of the created highly affected zone of the test wall.

9 References


