2015

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IN WEAKNESS ZONE CONTAINING SWELLING CLAY

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ABSTRACT

Tunnelling in weakness zones containing swelling clay represent one of the most difficult conditions in hard rock tunnelling, which could result in large excavation problems and in extreme cases even tunnel collapses. To enrich the engineering experience for such ground, the case of rock fall at the twin-tube Hanekleiv road tunnel is studied in the paper. The rock fall occurred ten years after tunnel completion in the southbound tube, approximately 1.1 km from the northern entrance, in a fault zone containing swelling clay. Laboratory testing results indicate gouge material in the collapse zone was not very active on swelling and the content of swelling clay was low. Numerical simulation has been carried out with focus on several selected mechanical states, particularly the one representing the long term loading on rock support. Both the detected shotcrete cracks during tunnel excavation and the tunnel collapse have been verified.

Keywords: Tunnel, Instability, Weakness Zone, Swelling Clay, Numerical analysis

INTRODUCTION

Major weakness zones or faults with heavily crushed and altered rock mixed with gouge material represents considerable challenges for tunnelling, particularly when the gouge material contains swelling clay (Selmer-Olsen et al. 1989; Nilsen and Dahlø 1994; Blindheim et al. 2005; Richards and Nilsen 2007; Nilsen and Palmstrøm 2009). It could cause large excavation problems and in extreme cases even tunnel collapses resulting in tremendous additional costs. Instabilities in such ground have been often connected to hydraulic tunnels after water filling. Reported cases of road tunnel instability are considerably less, mostly during construction through the zones.

The rock fall at the Hanekleiv road tunnel, however, occurred 10 years after the tunnel completion (Bollingmo et al. 2007; Reynolds 2007; Nilsen 2011). In this paper, this typical case of tunnel instability in weakness zone containing swelling clay is studied.

ROCK FALL AT HANEKLEIV TUNNEL

The Hanekleiv road tunnel is located in southeast Norway. It is 1765 m long and has two tubes spacing approximately 15 m. Each tube has a theoretical excavation cross-section of about 65 m^2 (Norwegian Public Roads Administration 2004). The bedrock along the northern part of the tunnel is Silurian sandstone which underlies Permian basalt and a layer of Carboniferous shale and conglomerate. The southern half of the tunnel is in Permian syenite. Topography and geology along the tunnel alignment is shown in Figure 1.

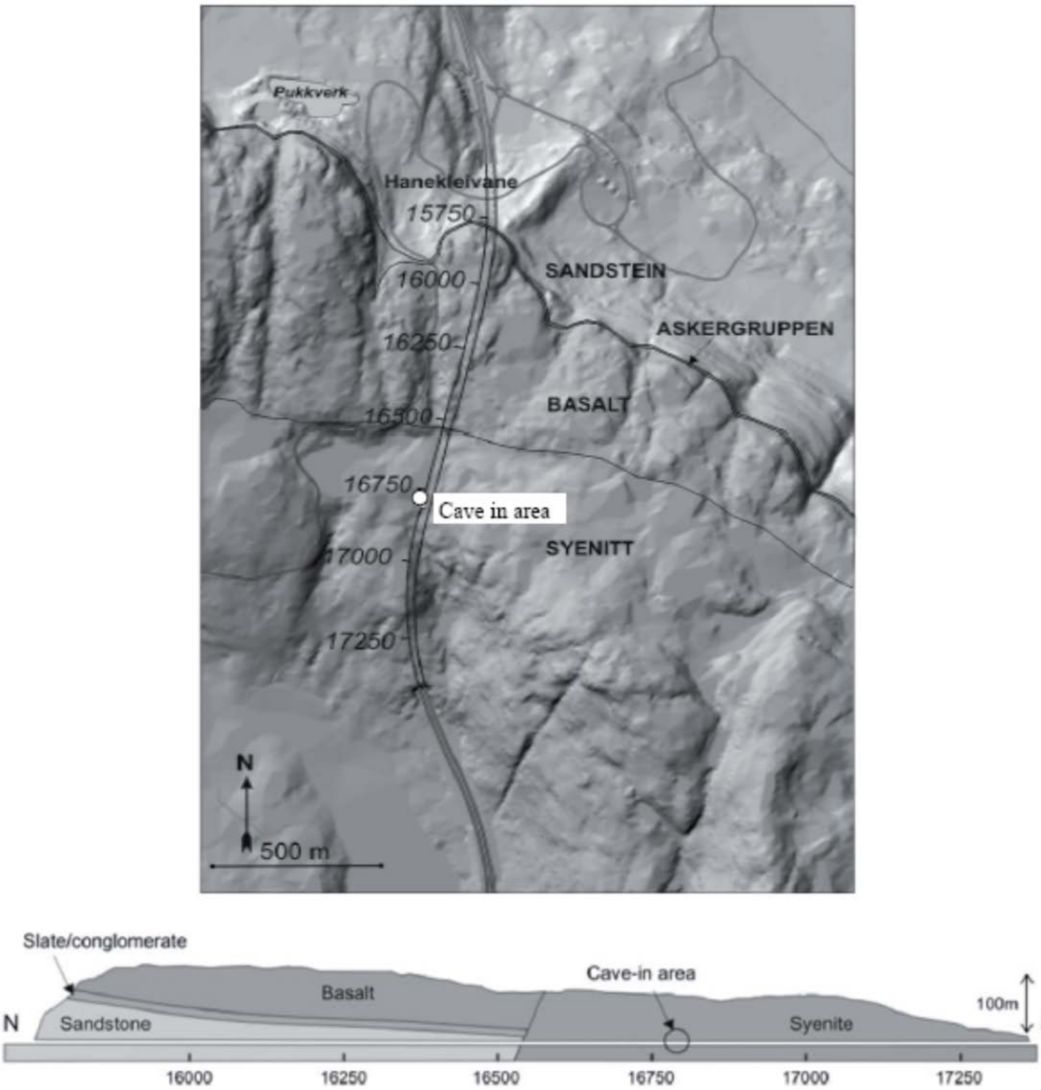

Figure 1 Topography and geology along the tunnel alignment, top: plan view, below: longitudinal cross section (based on Bollingmo et al. 2007)

The tunnel was excavated from the north. Blasting rounds was normally taken as 5 m and the rock was considered fair good. Rock supports used include mainly steel fibre reinforced shotcrete and radial rock bolts. Few minor stability problems were experienced during tunnel excavation and breakthrough was achieved with only 8 months. The northern half tunnel was expected to be the most difficult part for the drive due to the relatively flat bedding in the Silurian sandstone. A fault zone with swelling clay, however, was identified during excavation in syenite approximately 1.1 km from the northern portal of the southbound tube (Figure 1). Cracks were detected in the support of 15 cm thick fibre reinforced shotcrete during tunnel excavation at the zone. Thus 10 cm extra shotcrete was applied in the area after tunnel break through before installation of the shield for water proofing and frost protection. The principle sketch of shielding is shown in Figure 2. It may cope with small pieces of falling rock, but it is not designed to withstand major rock slides.

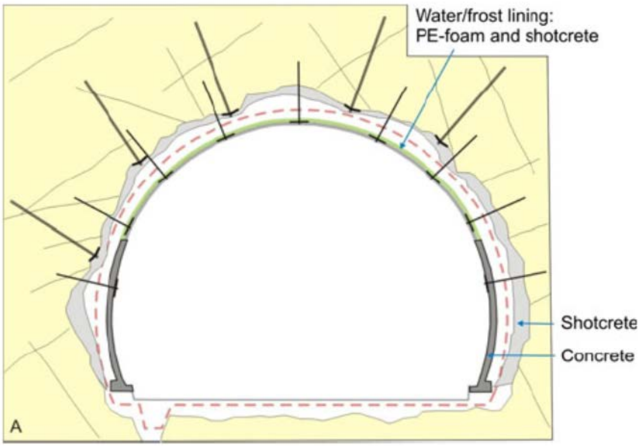

Figure 2 Principle sketch of shielding (inner water/frost lining) commonly used in Norwegian road tunnels, including the Hanekleiv tunnel (modified after Bollingmo et al. 2007)

The rock fall at the Hanekleiv tunnel occurred in December 2006, ten years after the tunnel completion (Reynolds 2007). Figure 3 shows the area where the rock fall went through the frost and water shielding. Only minor dripping was observed at a few locations and this area was generally dry (Bollingmo et al. 2007). The total volume of caved in material was estimated to 250 m^3. Although one large block of syenite weighing several tones was found at the site, most of the material was a mixture of small block, gravel, and fragments of altered syenite. The fault zone causing the cave-in intersected the tunnel at a small angle of 10 – 15^o, and was bounded by two parallel joints striking approximately 030 – 040^o, dipping 70 – 80^o to the southeast (Figure 3). The joints were filled with clay gouge with a thickness of 5 to 10 cm. Longitudinal and cross sections of the rock slide area are illustrated in Figure 4.

Figure 3 Cave-in section of the fault zone

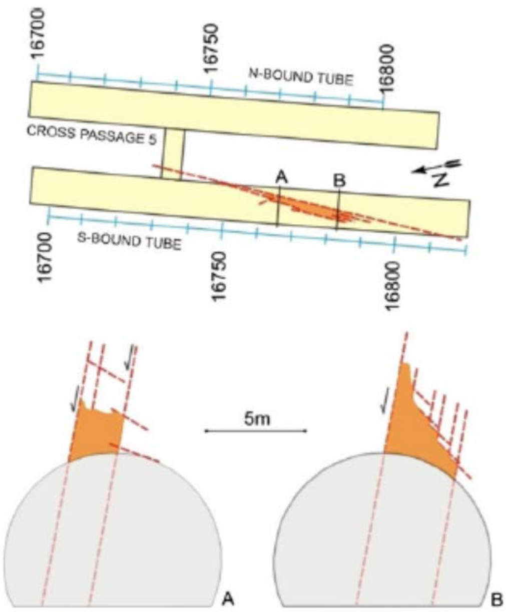

Figure 4 Sketch of longitudinal and cross sections of the rock slide area (modified after Bollingmo et al. 2007)

The rock mass quality of the fault zone was extremely poor (Q = 0.01 – 0.02). Estimated individual parameters of the Q-system are given in Table 1. The cracks that developed in the sprayed concrete and the rock fall that ultimately occurred clearly demonstrated that the applied support, consisting of rock bolts and shotcrete, was insufficient for this fault zone (Norwegian Geotechnical Institute (NGI) 2011). Based on thorough investigation it was concluded that, in addition to the effect of the swelling process, gravitational collapse due to the very low internal friction also played an important role in the development of instability in this case (Bollingmo et al. 2007; Nilsen 2011). The swelling process most likely was caused by both the water in joints and the accumulation of moisture behind the water/frost shield. The strength of the material was reduced with absorption of water during the swelling process until the collapse suddenly took place. The presumed combined effects will be focused in the numerical analysis.

Table 1 Input parameters for estimation of Q-value for the fault zone

Parameter	Fault zone
Rock quality designation (RQD)	10
Joint set number (J_n)	20
Joint roughness number (J_r)	1
Joint alteration number (J_a)	15
Joint water reduction factor (J_w)	1
Stress reduction factor (SRF)	2.5

Note: $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$

LABORATORY TESTING OF SWELLING CLAY

Clay samples were collected from the caved-in material as well as the joint fillings. Swelling mineral (e.g., smectite) was identified by the X-ray diffraction analysis. Identified minerals from clay samples include alkali feldspar (53%), plagioclase (21%), kaolinite (11%), mica (9%) and smectite (6%) (Bollingmo et al. 2007).

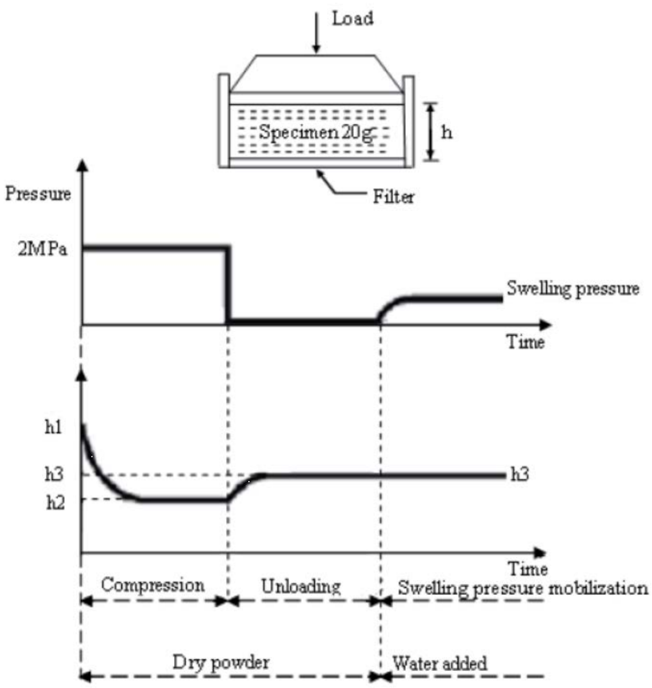

Figure 5 Principle sketch of swelling pressure test (based on Nilsen and Broch 2009)

In Norway, a laboratory test method based on measuring the swelling pressure of remoulded specimens (Brekke, 1965) has been used over decades for characterizing the swelling gouge material. In this standardized oedometer test, 20 g dried clay powder with grain size less than 20 μm segregated from the gouge material is pre-compressed at a 2 MN/m^2 to constant volume and then unloaded until constant volume is reached again. The sample thickness is kept constant as water is accessed to the sample and the mobilized pressure is measured (SINTEF Rock Engineering Group 2005). Based on this type of test (Norwegian Group for Rock Mechanics (NBG) 2000), NBG defines swelling pressure below 0.1 MPa as low, 0.1-0.3 MPa as moderate, 0.3-0.75 MPa as high and above 0.75 MPa as very high. For the collected samples, the clay fraction with grain size less than 20 μm accounted for 14% of the weight and the tested swelling pressure was 0.18 MPa (Bollingmo et al. 2007; Nilsen 2011).

The so called free swelling test is another common test to quantify the relative swelling potential, in which the clay fraction with particle size less than 20 μm is also used. 10 ml of loosely packed dry clay powder is drizzled into a 50 ml measuring cylinder filled with distilled water. The volume occupied by the clay powder after sedimentation is recorded, and the free swelling is calculated as the percentage of the original powder volume. NBG defines free swelling below 100% as low, 100-140% as moderate, 140-200% as high and above 200% as very high (Norwegian Group for Rock Mechanics (NBG) 2000). For the Hanekleiv sample the free swelling of 150% was recorded (Bollingmo et al. 2007; Nilsen 2011).

Numerical Analysis

Model description

For the numerical simulation, the finite difference code of FLAC3D (Itasca 2009) has been used. The generated model represents the rock fall area of the tunnel (see Figure 6). The model size is 117m (X) × 78m (Y) ×100m (Z), and the tunnel alignment is along Z-axis. Each boundary is fixed in its normal direction, except the top boundary where the pressure caused by gravity is applied. Simultaneous excavation of the two tubes is assumed in simulation. There is no stress measurement for the Hanekleiv tunnel. The initial vertical stress, in accordance with general Norwegian experience, is assumed to be caused by gravity as one of the principal stresses. Based on the available stress measurement in the same geological region (Hanssen 1998), the stress ratios, $k_H = 2$ and $k_h = 0.6$, are used. The minimum horizontal stress is taken as the same direction as the tunnel alignment.

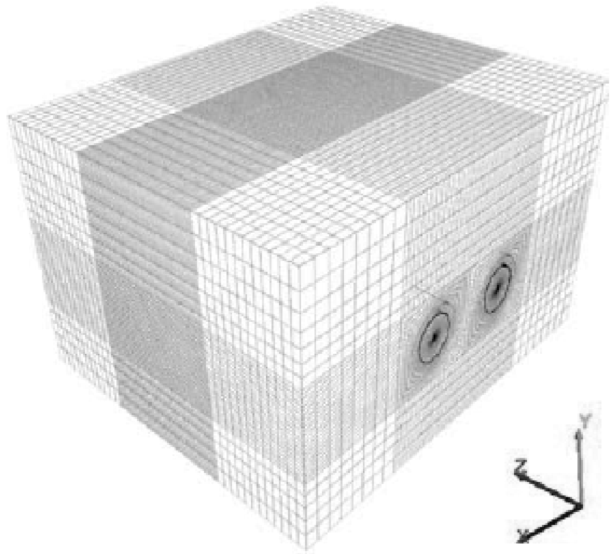

Figure 6 FLAC3D model

The fault zone and side rock are simulated with the Mohr-Coulomb model by the elasto-perfectly plastic stress-strain law. For shotcrete, the strain softening model is used in order to simulate its post failure behavior considering the cracks detected during tunnel excavation. Since the shotcrete cracks were caused by tension as demonstrated later in simulation results, the tensile strength of the shotcrete is assumed to drop to zero when failure occurs. The shotcrete is in tensile failure in simulation when the plastic tensile strain reaches 20 $\mu\varepsilon$, which is calibrated based on the grid size of the model. The rate at which the tensile strength drops is controlled by the plastic tensile strain and the linear softening law for the tensile strength (Itasca 2009). Physical and mechanical properties of the original fault zone, the weakened fault zone (close to the tunnel periphery 10 years after excavation), the side rock and applied shotcrete used in the modeling are presented in Table 2. Estimation of properties is based mainly on engineering experiences for these types of materials. Since the strength reduction developed gradually inwards, there was a variation of strength reduction in the weakened fault zone. In numerical simulation, the conservative simplification has been made that the properties of the weakest material from the caved-in rock mass are assumed for the whole weakened fault zone.

The ground water level above the caved-in area is unknown and potential effects of the ground water have been neglected in the simulation. As earlier mentioned, the rock mass in the caved-in area was relatively dry with dripping only at a few locations. Consequently, the water pressure on shotcrete is assumed to have been very low. Rock bolts are not considered in numerical simulation since very few bolts were installed and the installation pattern is unknown.

Table 2 Properties of fault zone, side rock and shotcrete

Properties	Original fault zone	Weakened fault zone	Side rock	Shotcrete
Density (kg/m^3)	2,700	2,700	2,700	2,350
Modulus of elasticity (E) (GPa)	3.1	2.0	45	20
Poisson's ratio (υ)	0.3	0.3	0.22	0.2
Bulk modulus[a] (K) (GPa)	2.6	1.7	26.8	11.1
Shear modulus[b] (G) (GPa)	1.2	0.8	18.4	8.3
Cohesion (MPa)	0.45	0.2	6	7
Friction	30°	15°	50	45°
Dilation	–	–	15°	12°
Tensile strength (MPa)	–	–	1	0–4[c]

[a] $K = E/3(1–2\ \upsilon)$

[b] $G = E/2(\ 1\ +\upsilon)$

[c] Tensile strength of shotcrete depends on the plastic tensile strain

The rock fall was a long lasting, gradual process of mobilization of swelling pressure and strength weakening of the fault zone. This process is hard to define explicitly, and in the numerical simulation three stages of mechanical states have been focused. The first stage represents tunnel exaction and detection of shotcrete cracks. The swelling pressure on the rock support of shotcrete is considered at the second stage. Combined effects of strength reduction of the fault zone and swelling are simulated at the last stage. The flow chart of modeling of the Hanekleiv case is shown in Figure 7.

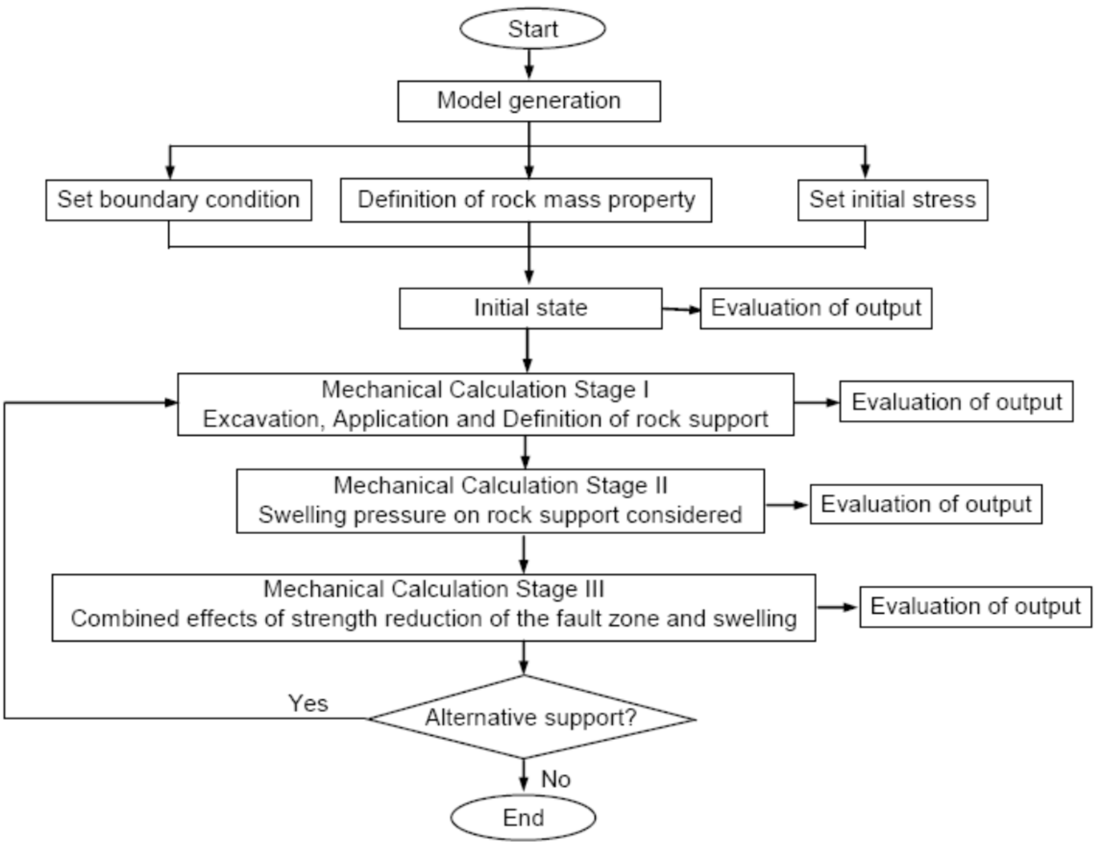

Figure 7 Flow Chart of modeling process of the Hanekleiv case

Simulation result

The tunnel excavation with 5 m blasting rounds can be realized by changing ground elements into null elements in sequence. Application of shotcrete after each excavation step is simulated by changing the corresponding layers into elements with properties of shotcrete. At the first stage, simulation results show the shotcrete close to the blasting face at the fault zone fails due to tension. The yielded zones of shotcrete at the boundary of shotcrete and rock are shown to the left in Figure 8. Yielded zones are also found at the inner surface of the shotcrete, and this verifies the detected shotcrete cracks. The simulated yielded zones of the faults (see Figure 9) are similar to the geometry of the slides. The reduced strength of the weakened fault zone is assumed for these yielded zones at the third stage.

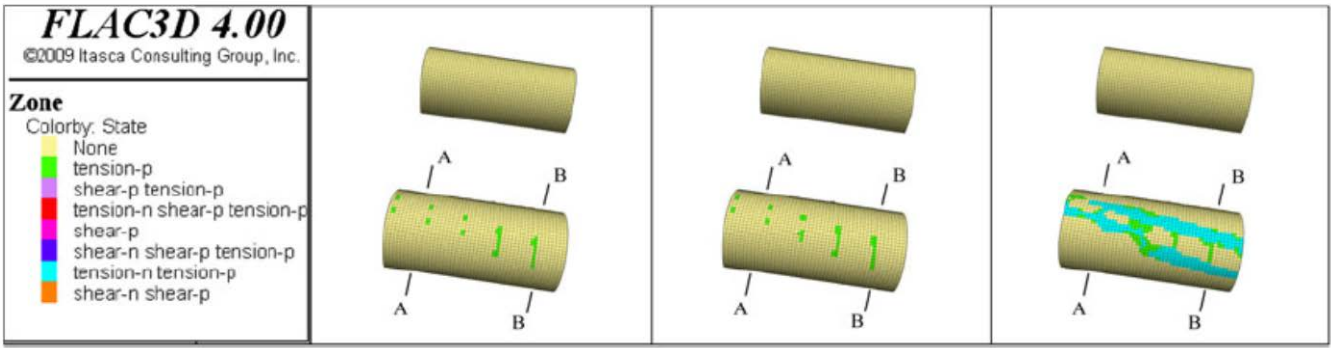

Figure 8 Yielded zones of shotcrete at three stages of the numerical simulation. (*Left* after excavation, *Middle* swelling pressure considered, *Right* Swelling pressure and weakened fault zone considered)

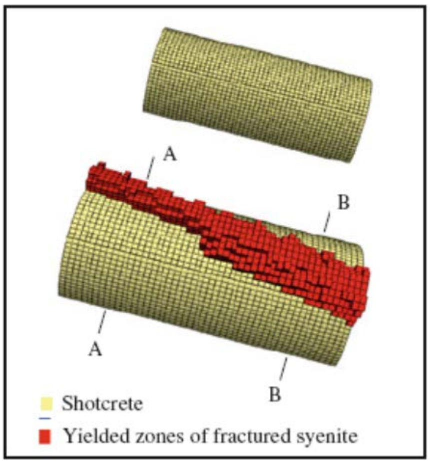

Figure 9 Yielded zones of fault based on numerical simulation

In the model extra shotcrete was applied when cracks were detected in the shotcrete. There is no detailed information on exactly where the extra shotcrete was applied. At the second stage of simulation, the extra shotcrete is however assumed to have been applied covering the whole area of the fault zone. Effects of the swelling pressure on the rock support are considered from an engineering perspective as the uniformly distributed load. Swelling pressures up to 0.18 MPa have been assumed on the shotcrete support. This is realized in the simulation by applying the corresponding loads at the boundary between the fault zone and the shotcrete (Mao et al. 2011). Yielded zones of shotcrete (for swelling pressure 0.18 MPa) are illustrated in the middle part of Figure 8. Comparison of the simulated yielded zones indicates no essential difference between the areas of failed shotcrete. This shows that the relatively low swelling pressure had only a limited influence on the loading of the shotcrete.

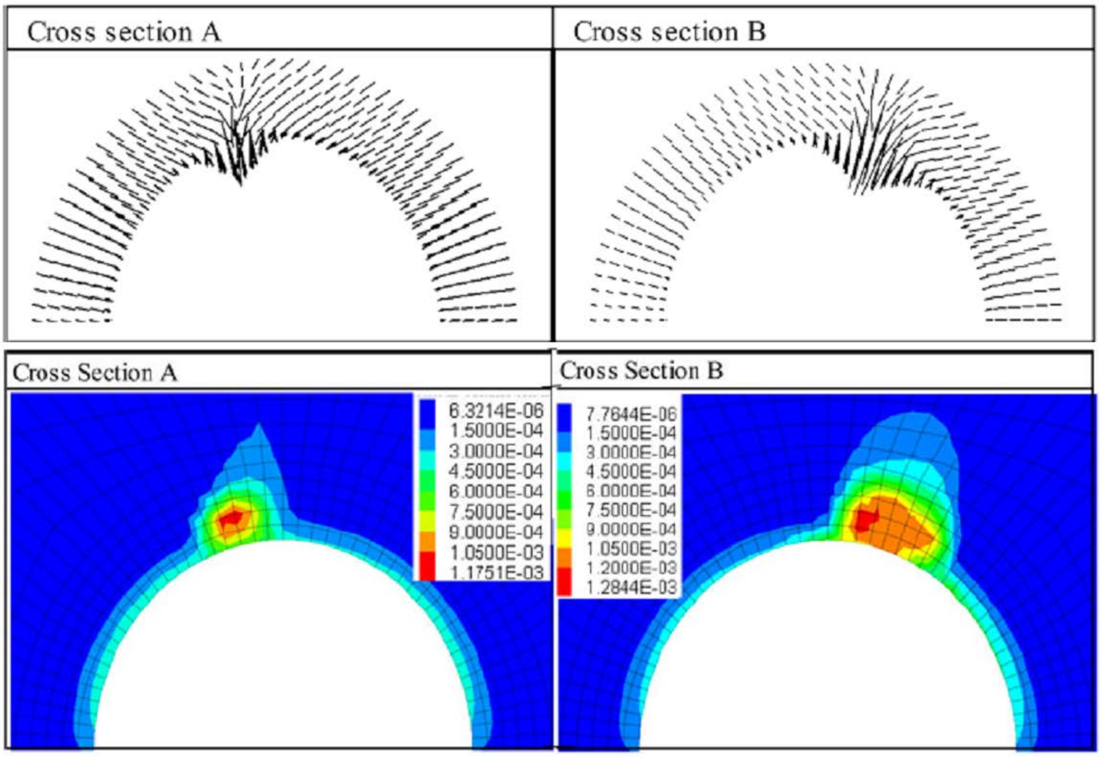

Figure 10 Displacement vector (Top) and shear strain increment contour (Bottom) at rock fall area

At the third stage, the properties of weakened fault zone are assumed for the simulated yielded zones of the fault. A previous study carried out suggests that the in situ swelling pressure is often lower than the laboratory value, in some cases only 50 % (Tyssekvam 1996). The right section of Figure 6 shows the yielded zones of shotcrete for the weakened fault zone and a swelling pressure of 0.09 MPa (50% of the laboratory tested swelling pressure). The yielded area of shotcrete due to tension has been greatly increased along the fault zone and continues to increase with more calculation steps. As a sign of calculation non-convergence, the contiguous line of active plastic zones joining two surfaces indicates breakdown of the shotcrete and tunnel collapse (Itasca 2009). This is also confirmed by the velocity vector where the movement of the fault zone is much higher than for the other parts of the tunnel periphery. The vector of displacement close to the tunnel periphery at one instantaneous state is illustrated at the top of Figure 10. The shear strain increment of the rock mass at cross sections A and B of the collapse area is shown at the bottom of Figure 10. The inclination of the contour lines is comparable to the dip of joints in the fault zone. The area within the contour line of the strain increment of 450 $\mu\varepsilon$ has almost the same width as the slide area at both locations of cross section A and B.

Discussion and Conclusion

The case of Hanekleiv rock fall discussed above illustrates the effects of weakness zones containing swelling clay on tunnel stability may take long time to develop and the cave in could occurred years after tunnel completion. For this case, comprehensive analysis from the laboratory testing to numerical simulation has been conducted after site investigation. Laboratory testing includes the X-ray diffraction analysis for identifying the mineral composition, and the swelling pressure test and free swelling test for quantifying the swelling potential. Testing results show gouge material in the collapse zone was not very active and the content of swelling clay (smectite) was quite low. In the numerical analysis, several equilibrium mechanical states rather than the whole long term process, particularly the one representing the long term loading on rock support, have been focused. Both the detected cracks during tunnel excavation and the tunnel collapse have been verified. Heavy rock support like reinforced shotcrete rib or cast-in-place concrete lining should have been considered.

At present, the weakness zones containing swelling clay can not be fully accounted in any of the commonly used rock mass classification systems due to its complexity. Regular check of signs of instability such as shotcrete cracks will be essential for stability control in practical. In situ instrumentation on strain, stress, displacement, etc., if applicable, may also provide the base for evaluation of potential instability. With further research on more cases, the knowledge on tunnelling through such ground will be enriched.

References

Blindheim O L, Grøv E, Nilsen B (2005) Nordic sub sea tunnel projects. Tunn Undergr Space Technol 20 (6): 570 -580

Bollingmo P, Nilsen B, Nordgulen Ø (2007) The cave-in at the Hanekleiv tunnel, 25 December 2006. Report from investigation panel established by the Norwegian Department of Transportation (in Norwegian), 14 February 2007

Brekke T L (1965) On the measurement of the relative potential swellability of hydrothermal montmorillonite clay from joints and faults in Precambrian and Paleozoic rocks in Norway. Int J Rock Mech Min Sci 2:155-165

Hanssen T H (1998) Rock stresses and tectonic activity. In: Proceedings of the Norwegian National Rock Mechanics Conference. Oslo, pp. 29.1-29.24

Itasca (2009) Fast lagrangian analysis of continua in 3 dimensions, version 4.0, user's manual. In: Itasca I (ed) Consulting Group, Minneapolis, Minnesota, USA

Mao D, Nilsen B, Lu M (2011) Analysis of loading effects on reinforced shotcrete ribs caused by weakness zone containing swelling clay. Tunn Undergr Space Technol 26: 472 - 480

Nilsen B, Broch E (2009) Engineering Geology of Rocks, Basic Level Compendium. Department of Geology and Mineral Resources, NTNU. In Norwegian

Nilsen B (2011) Cases of instability caused by weakness zones in Norwegian tunnels. Bull Eng Geol Environ 70:7-13

Nilsen B, Dahlø T S (1994) A study of cases of instability in hard rock tunnel. In: Proceedings of 7th IAEG Congress. Lisbon, vol. VI, Balkema, pp 4233-4242

Nilsen B, Palmstrøm A (2009) Engineering geological key factors for planning and constructing hard rock subsea tunnels. In: Proceedings of the 5th Symposium on Strait Crossings. Trondheim, pp 403-408

Norwegian Geotechnical Institute (NGI) (2011) https://www.ngi.no/en/Contentboxes-and-structures/Reference-Projects/Reference-Projects/Q-method/

Norwegian Group for Rock Mechanics (NBG) (2000) Engineering geology and rock engineering. Handbook No. 2. Oslo, Norway, p 192

Norwegian Public Roads Administration (2004) Manual of road tunnels, Handbook No. 021. Oslo, Norway, pp 31-38

Reynolds P (2007) Finding fault at Hanekleiv. Tunnels and Tunnelling International, pp 14-16

Richards D P, Nilsen B (2007) Geotechnical risk management for tunnelling beneath open water. In: Proceedings of the 33rd ITA World Tunnel Congress, Prague, pp 1585 – 1590

Selmer-Olsen R, Palmstrøm A, Strømme B (1989) Tunnel collapses in swelling clay zones. Tunn Tunn 21: 49-51

Tyssekvam I O (1996) Sampling and Characterization of swelling material in weakness zones (In Norwegian), Diploma Thesis. NTNU, Trondheim.