

Numerical analysis of spalling potential in a TBM tunnel using Phase² from Rocscience

Numerische Analyse des Potenzials der Abschaltung in einem TBM Tunnel mit Phase² von Rocscience

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Abstract

During tunneling for the Kobbelv hydropower project in Norway, heavy spalling and rock bursting led to challenges with regard to working safety, excavation and rock support measures. A numerical analysis using Phase² from Rocscience has been conducted in order to investigate the theoretical spalling potential and depth of failure in one of the TBM (Tunnel Boring Machine) tunnels. The Hoek-Brown brittle parameters ($m = 0$ and $s = 0.11$) have been used for this purpose. The potential for spalling has been found to be large and the numerical results are discussed in relation to modern spalling theory. The results are in agreement with observations made during construction, but a more detailed study is needed in order to confirm the findings with actual in-situ conditions. For future Norwegian TBM tunnels in highly stressed rocks it is believed that analyses of brittle failure potential and severity are crucial for the choice of the right TBM machine and to forecast and implement the right rock support measures.

Keywords: Rock stress, brittle failure, spalling, Hoek-Brown brittle parameters, TBM, Norway

Zusammenfassung

Während des Baus des Kobbelv Wasserkraftwerks in Norwegen stellt das Auftreten von Abschaltungen und Gebirgschlag im Bezug auf die Arbeitssicherheit, den Bau und Felssicherung eine neue Herausforderung dar. Um das theoretische Potenzial für Abschaltung und das Ausmass von Bruch in einem der TBM (Tunnel Boring Machine) Tunnel festzustellen, wurde eine numerische Modellierung mit Phase² von Rocscience durchgeführt. Hierfür wurden die Hoek-Brown brittle Parameter ($m = 0$ und $s = 0.11$) verwendet. Das Potenzial für Abschaltung stellte sich als gross heraus, die Ergebnisse der numerischen Modellierung werden in Bezug auf die moderne Abschaltungs Theorie diskutiert. Die Ergebnisse werden durch die während der Konstruktion gemachten Observierungen bestätigt, allerdings muss eine detaillierte Studie durchgeführt werden um das Ergebnis unter tatsächlichen in-situ Bedingungen zu bestätigen. Für zukünftige norwegische TBM Tunnel in Fels mit erhöhtem Spannungszustand ist die Sprödigkeitsanalyse unerlässlich um die richtige TBM Maschine und weiterführenden Massnahmen auszuwählen und umzusetzen.

Schlüsselworte: Felsspannung, Sprödbbruch, Abschaltung, Hoek-Brown brittle Parameter, TBM, Norwegen

1 Introduction

The Kobbelv hydropower scheme in north Norway was build between 1983 and 1987 and include over 30 km of tunnels, in which about half were excavated with TBMs. In the bored tunnels extensive spalling and rock bursting occurred in the roof and the invert, resulting in tedious rock support works and delays. A comprehensive investigation program was commenced to illuminate the causes for the problems and to help determine practical measures to counteract the difficulties (MYRVANG et al. 1998). The rock types at Kobbelv were mostly massive gneisses and granites. Their mechanical properties were carefully tested in the laboratory and in-situ stress measurements were undertaken in 6 tunnels by overcoring of triaxial cells. The measurements showed a very consistent pattern where high horizontal stresses were dominating.

The finite element program Phase² and the Hoek-Brown brittle parameters as described in MARTIN et al. (1999) are used to investigate the spalling potential and the theoretical depth of spalling at Kobbelv. The results are discussed in relation to the observations made during construction and modern spalling theory.

2 Spalling theory

The equations of Kirsch describe the induced tangential stresses on the periphery of a circular opening (Fig. 1). At Kobbelv the major principal stress (σ_1) is horizontal and the minor principal stress (σ_3) vertical. The intermediate principle stress (σ_2 or σ_z) is also horizontal. The author does not know the exact orientation of the horizontal stresses in relation to the tunnel axis. With the assumption that σ_1 and σ_3 are in-plane stresses and σ_2 out-of-plane (aligned in the



direction of the tunnel axis), the configuration represents a worst-case scenario with the highest ($\sigma_{\theta\max}$) and lowest ($\sigma_{\theta\min}$) possible tangential stresses induced on the contour.

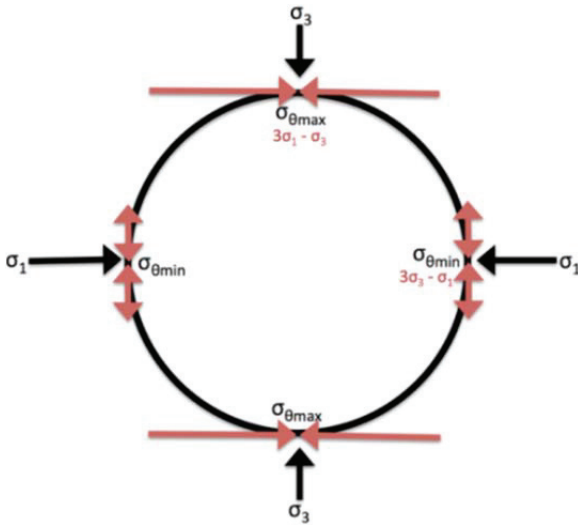


Fig. 1: Tangential stresses around a circular opening (described by Kirsch equations) induced by the major and minor principal stresses. The double-direction arrows in the walls illustrate that the induced stresses also can be tensile.

Abb. 1: Tangentielle Spannungen um eine kreisförmige Öffnung (beschrieben durch die Gleichungen von Kirsch) induziert durch die grösste und kleinste Hauptspannung. In der Tunnelwand kann auch Zugdehnung vorhanden sein.

Many researchers have studied brittle failure of rocks in recent years (e.g. MARTIN et al. 1997, DIEDERICHS et al. 2004, MARTIN & CHRISTIANSSON 2009, NOFERESTI & RAO 2010). ROJAT et al. (2009) summarize some of the findings: Damage initiation (microcracks start to form) and coalescence (merging of microcracks) happens at distinct levels in the laboratory, respectively around 0.4 and 0.8 of the UCS (σ_c) for granitic and gneissic rock types; The coalescence stage corresponds to the long-term laboratory strength of the sample; Brittle failure however typically occurs around 0.4 ± 0.1 of the UCS in a tunnel, i.e. near or slightly above the stress level required for damage initiation; The reasons for in-situ strength degradation relative to laboratory strength is complex and depends on the rock mass stress-strain history, heterogeneity and stress-rotation effects among other factors. A general agreement among researchers seems to be that the damage initiation threshold from laboratory tests can be taken as a lower bound for the rock mass strength.

MARTIN et al. (1999) showed that numerical modelling of brittle failure using the traditional Hoek-Brown parameters underpredicts the depth of failure around a tunnel. In order to account for the fact that brittle failure processes is dominated by cohesion loss they propose to use the Hoek-Brown brittle parameters (Eq. 1) for such predictions. These parameters are thought to reflect the damage initiation threshold.

$$m = 0 \text{ and } s = 0.11 \quad \Leftrightarrow \quad \sigma_1 - \sigma_3 = \frac{1}{3} \sigma_c \quad (\text{Eq. 1})$$

MARTIN et al. (1999) also propose an empirical criterion for predicting the depth of failure around near circular tunnels (Eq. 2).

$$\frac{R_f}{a} = 0.49 (\pm 0.1) + 1.25 \frac{\sigma_{\theta\max}}{\sigma_c} \quad (\text{Eq. 2})$$

Where “ R_f ” is the failure radius (tunnel radius + depth of failure) and “ a ” is the tunnel radius.

3 Numerical model

Both the plane-strain and the axisymmetric analysis option of Phase² have been used to assess the problem in 2 and 3 dimensions respectively. All analyses are elastic. The orientation of the tunnel in relation to the horizontal principal stresses is not known. Two plane-strain analyses are therefore executed (σ_1 and σ_2 alternate as in-plane stress together with σ_3) in order to model the best and worst case scenario. In the axisymmetric analysis σ_1 is used as the radial stress and σ_2 as the axial stress. The assumption for this constant radial in-plane stress around the tunnel will not give a correct picture of the stress magnitudes, but it will shed some light on the 3-dimensional nature of the stress-flow/stress-rotation at the tunnel face. The two models are shown in Fig. 2 a) and b). To achieve equilibrium in model a) before excavation an initial vertical load of 5 MPa (equal to the constant vertical in-situ stress) has been applied. The axisymmetric analysis requires that the axis $X = 0$ in front of the tunnel face is restrained in the X-direction (ROSCIENCE 2011) to allow the model to rotate around this axis (the axis of symmetry). The tunnel diameter is 6.25 m, which is the largest tunnel diameter used at Kobbelv.

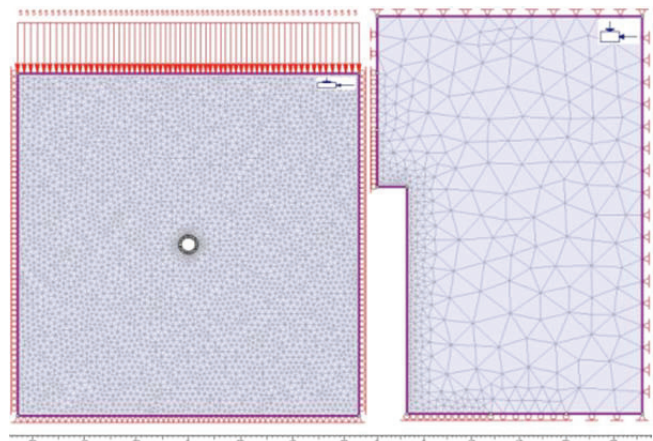


Fig. 2: Model setup and boundary conditions for a) plain strain (left) and b) axisymmetric analysis.

Abb. 2: Modellaufbau und Randbedingungen für a) plain strain (links) und b) axisymmetric analysis

The input parameters used in the models are summarized in Tab. 1. Results are presented in chapter 4.

Tab. 1: Rock mass parameters and stress state from MYRVANG et al. (1998). Tangential stresses calculated from Kirsch equations. Brittle parameters after MARTIN et al. (1999).

Tab. 1: Gebirgsparameter und Spannungszustand von MYRVANG et al. (1998). Tangential Spannungen berechnet nach den Kirsch Gleichungen. Sprödbruch Parameter nach MARTIN et al. (1999).

Rock mass param.	Stress state plane-strain / axisymmetric	Plain-strain	Axisymmetric
	σ_1 / radial (MPa)	27 / 15	27
	σ_2 / axial (MPa)	15 / 27	15
	σ_3 (MPa)	5	
	$\sigma_{\theta max}$ (MPa)	76 / 40	
	$\sigma_{\theta min}$ (MPa)	-12 / 0	
Tunnel radius (m)		3.125	
Rock type		Granite	
σ_c (MPa)		89	
$\sigma_c * 1/3$ (MPa)		29.67	
$\sigma_{tensile}$ (MPa)		9.5	
Young's Modulus (MPa)		18500	
Poisson's ratio		0.13	
Unit Weight (MN/m ³)		0.027	
mb		1*10 ⁻⁵	
s		0.11	

4 Results

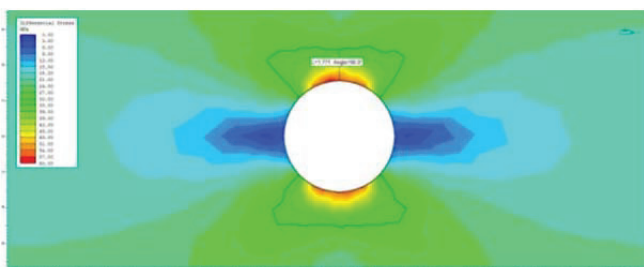


Fig. 3: Differential stress ($\sigma_1 - \sigma_3$) plot. $\sigma_1 = 27$ MPa $\sigma_2 = 15$ MPa (out of plane) $\sigma_3 = 5$ MPa. Brown iso-line represents $\sigma_c * 1/3 = 29.67$ MPa. Depth of spalling measured to ca. 1.8 m.

Abb. 3: Tiefe der Abschalung ca. 1.8 m.

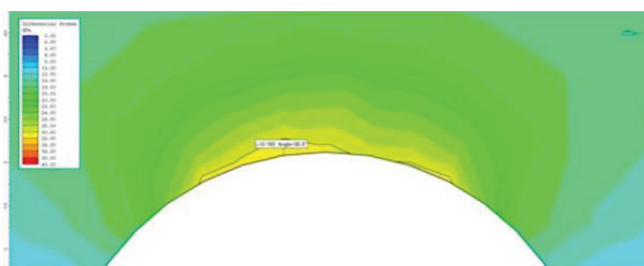


Fig. 4: Differential stress ($\sigma_1 - \sigma_3$) plot. $\sigma_1 = 15$ MPa $\sigma_2 = 27$ MPa (out of plane) $\sigma_3 = 5$ MPa. Brown iso-line represents $\sigma_c * 1/3 = 29.67$ MPa. Depth of spalling measured to ca. 0.2 m.

Abb. 4: Tiefe der Abschalung ca. 0.2 m.

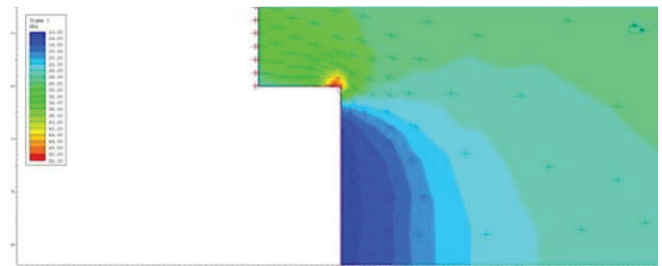


Fig. 5: Stress flow (induced by the radial stress) around the tunnel face. $\sigma_{1(radial)} = 27$ MPa $\sigma_{2(axial)} = 15$ MPa. Obvious stress concentration (ca. 50 MPa) in the transition between the face and the crown&invert/walls.

Abb. 5: Spannungszustand an der Ortsbrust.

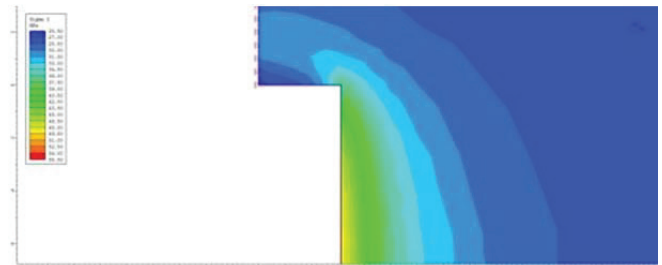


Fig. 6: Plot of Sigma Z (σ_z). Stress configuration as in Fig. 5. Due to the axisymmetric nature of the problem Sigma Z is the induced circumferential stress around the excavation (ROCSCIENCE 2011). It can be observed that the stress is large in the transition between the face and the crown&invert/walls and that it reaches full magnitude at short distance behind the face.

Abb. 6: Spannungszustand um den Tunnel rum.

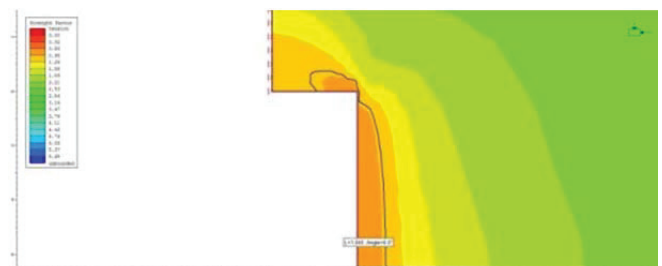


Fig. 7: SF (Strength Factor) plot. Stress configuration as in Fig. 5. Black iso-line represents SF = 1. Depth of spalling measured to ca. 1 m about 2 r behind the face. Most of the spalling develops in front and immediately behind the face.

Abb. 7: Tiefe der Abschalung ca. 1 m.

5 Discussion and conclusion

MYRVANG et al. (1998) reported that the laboratory parameters showed relatively low values compared to granitic rocks normally found in Norway. This was believed to be a result of the coarse grained texture of the rock. Uncertainty in the presented results comes from, but is not limited to, lack of detailed information about tunnel orientation in relation to the measured in-situ stress and lack of information about damage initiation and coalescence thresholds from laboratory tests. No true 3D modelling has been performed



and stress magnitudes from the axisymmetric analysis must be considered with care.

Simply by comparing the tangential stresses in Tab. 1 with a typical damage initiation threshold from the literature (89 MPa (UCS) * 0.4 \approx 36 MPa) the potential for spalling behavior in the crown and invert of the tunnel can be seen. The worst and the best-case configurations of principal stresses result in a SF (Strength Factor) for the rock mass of 0.5 (36/76) and 0.9 (36/40) respectively. The corresponding depth of failure using Eq. 2 is 1.7 m (interval 1.4 to 2.1 m) and 0.2 m (interval 0 (-0.2) to 0.5 m). These values correspond well with the results from the plane-strain analysis, respectively 1.8 m (Fig. 3) and 0.2 m (Fig. 4). There is unfortunately no detailed data on the depth of failure at Kobbelv, but Fig. 8 and Fig. 9 gives an impression of the prevailing conditions during construction. Both larger and smaller overbreak can be seen. A more detailed study needs to be undertaken in order to confirm the results of this paper with the actual in-situ conditions.

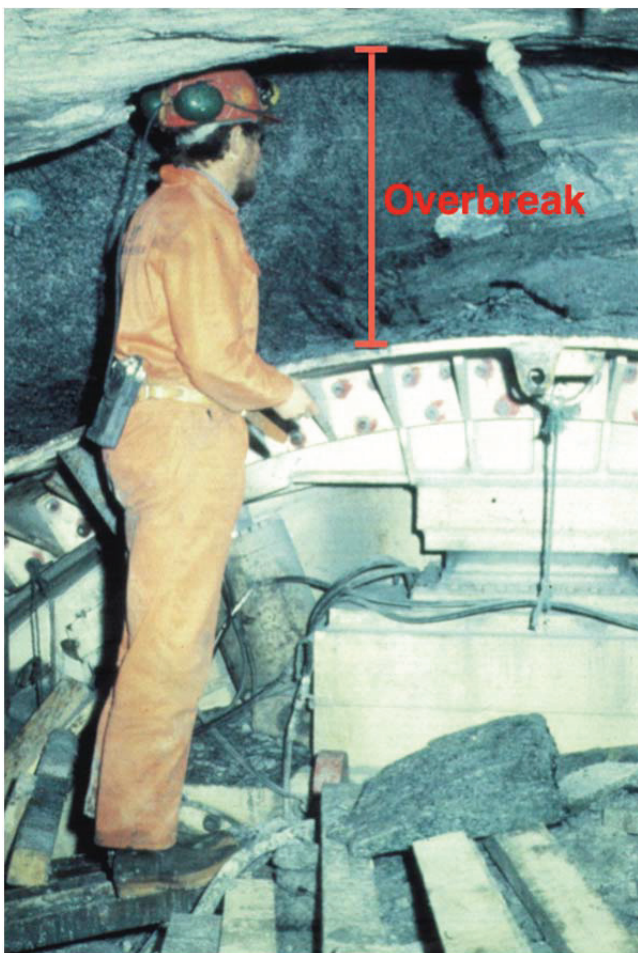


Fig. 8: Overbreak due to spalling at Kobbelv. Modified from MYRVANG et al. (1998).

Abb. 8: Abschaltung in dem Kobbelv Tunnel. Modifiziert nach MYRVANG et al. (1998).

Loading path and stress rotation due to an advancing tunnel face has been shown to influence spalling potential around underground openings (e.g. DIEDERICHS et al. 2004). The rock mass may already spall at the face or the damage initiation threshold might be reached, leaving behind a weaker

rock mass that is more prone to spalling when the tunnel advances and the tangential stresses increase just behind the face. Fig. 5 shows a stress concentration of 50 MPa in the transition between the face and crown&invert/walls. Fig. 7 shows a SF plot. The results show that spalling (and definitely forming of microcracks) may occur already at the face. Since the stress regime in reality is not radial ($\sigma_1 \neq \sigma_3$) at Kobbelv the face/crown&invert or the face/wall transition will be loaded more than its counterpart depending on the prevalent stress configuration. Gripper problems due to overbreak in the walls was reported at Kobbelv. This is indicative of substantial loading and failure at the face/wall transition, since the tangential stresses in the walls are low behind the face.



Fig. 9: Overbreak due to what was described as spalling and crushing at Kobbelv. Modified from MYRVANG et al. (1998).

Abb. 9: : Abschaltung in dem Kobbelv Tunnel. Modifiziert nach MYRVANG et al. (1998).

Fig. 6 shows how the stresses increase rapidly just behind the face. The SF plot in Fig. 7 shows that the depth of failure behind the face is ca. 1 m. As described above the axisymmetric model will not give fully reliable results. From Tab. 1 it can be seen that the tangential stress in the walls may be as low as -12 MPa. This exceeds the tensile strength of the rock, which was measured to 9.5 MPa in the laboratory. The tensile strength of the rock mass is likely to be much lower than the laboratory value. At Kobbelv horizontal tensile cracks was actually observed in the springline in each of the walls in one of the tunnels. These cracks typically started to develop 20-30 m behind the face and extended for several hundred meters. They did not cause any stability problems (MYRVANG et al. 1998). As the rock cover increased it was reported that the spalling intensity decreased and finally ceased. This was believed to be a result of progressively higher vertical stress and a resulting lower tangential stress concentration.

For future Norwegian TBM tunnels in highly stressed rocks the author believes that analyses of brittle failure potential and severity are crucial for the choice of the right TBM machine and to forecast and implement the right rock support measures.

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