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# Numerical analysis of stresses and displacements for the Tafjord slide, Norway

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**Abstract** Numerical modelling has been used for analyzing stresses and displacements for the very steep and more than 1,000 m high Heggura rock slope near Tafjord, Norway where a disastrous 3 million $m^3$ rock slide occurred in 1934. It is shown that very anisotropic stresses exist near the slope surface and displacements of the remaining slope as result of the 1934 slide have been calculated to up to 210 mm. Such considerable displacements are believed to have a significant impact on the present and future stability of the Heggura slope.

**Keywords** Rock slope stability · Numerical analysis · Stresses · Displacements · Norway

**Résumé** Une modélisation numérique a été réalisée pour analyser les contraintes et les déplacements relatifs au versant rocheuxde Heggura, de plus de 1000 m de hauteur, près de Tafjord en Norvège. Il s'agit d'un endroit où un glissement rocheux désastreux de 3 millions de $m^3$ a eu lieu en 1934. Le calcul montre qu'un champ de contraintes très anisotrope existe sous la surface du versant et que les déplacements du versant qui ont suivi le glissement rocheux s'élèvent à 210 mm. On pense que de tels déplacements ont des conséquences sur la stabilité actuelle et future du ver ant de Heggura.

**Mots clés** Stabilité de versant rocheux · Analyse numérique · Contraintes · Déplacements · Norvège

## Introduction

In 1934, a disastrous rock slide occurred at Heggura near the village of Tafjord on the west coast of Norway. The origin of the slide was high up in the steep mountain at the north-eastern shore of the fjord, see Figs. 1 and 2. The slide included a rock mass with an approximate volume of three million cubic metres and caused a huge tsunami when it ended up in the fjord (Bjerrum and Jørstad 1968; Blikra et al. 2004; Braathen et al. 2004). The bedrock in the area consists of mica gneiss with distinct foliation dipping steeply towards south. It is believed that the slide occurred as a result of the formation of a steep tension joint that came in contact with the slip surface along the foliation.

In general terms, the stability of rock slopes is controlled by several factors, such as the slope topography, the orientation of discontinuity planes, the shear strength of discontinuities and intact rock, groundwater pressure, the in situ stress conditions and seismic activity. Geotechnical numerical models can be used to analyze the stress and displacement patterns along the slope topography and therefore are of great significance for a full understanding of the slope stability situation at Heggura.

Analysis of rock slope stability typically involves a three-step procedure (Nilsen and Palmstrøm 2000):

1. Identification of potential failure mode (slide geometry),
2. Quantification of input parameters and,
3. Calculations. For the Tafjord case, step (1) is well documented by former studies (see references above), and step (2), which often represents the main uncertainty for analysis, is greatly assisted by data collected in a nearby road tunnel.

The road tunnel referred to is the 5,360 m long Heggura road tunnel, which was constructed in 1980–1982 (Fig. 1). During tunnel excavation, rock stress measurements as well as laboratory testing of the rock samples were performed (Broch and Sørheim 1984). Having such factual data of the area is of great advantage when numerically analyzing the stress and displacement regimes of the Heggura slope and is believed significantly to increase the reliability of numerical analysis on the slope.

It is generally accepted that the in situ rock stresses have considerable influence on the rock deformation. As the Heggura slope consists of mica gneiss with the foliation dipping towards the fjord, it is natural to assume that the in situ stresses must have played a vital role in

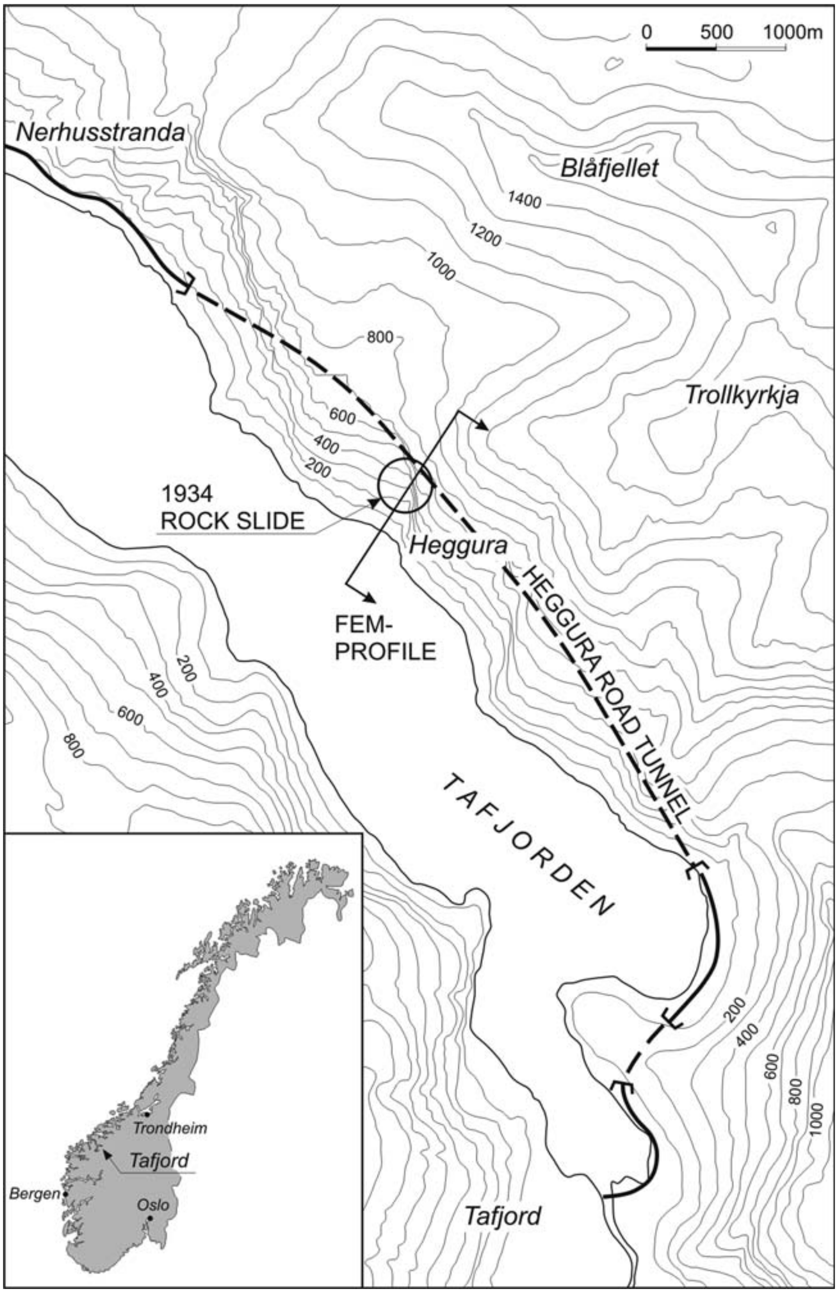


**Fig. 1** Map with 100 m contour interval showing the locations of the 1934 Tafjord rock slide, the Heggura road tunnel and the Phase$^2$ FEM-profile

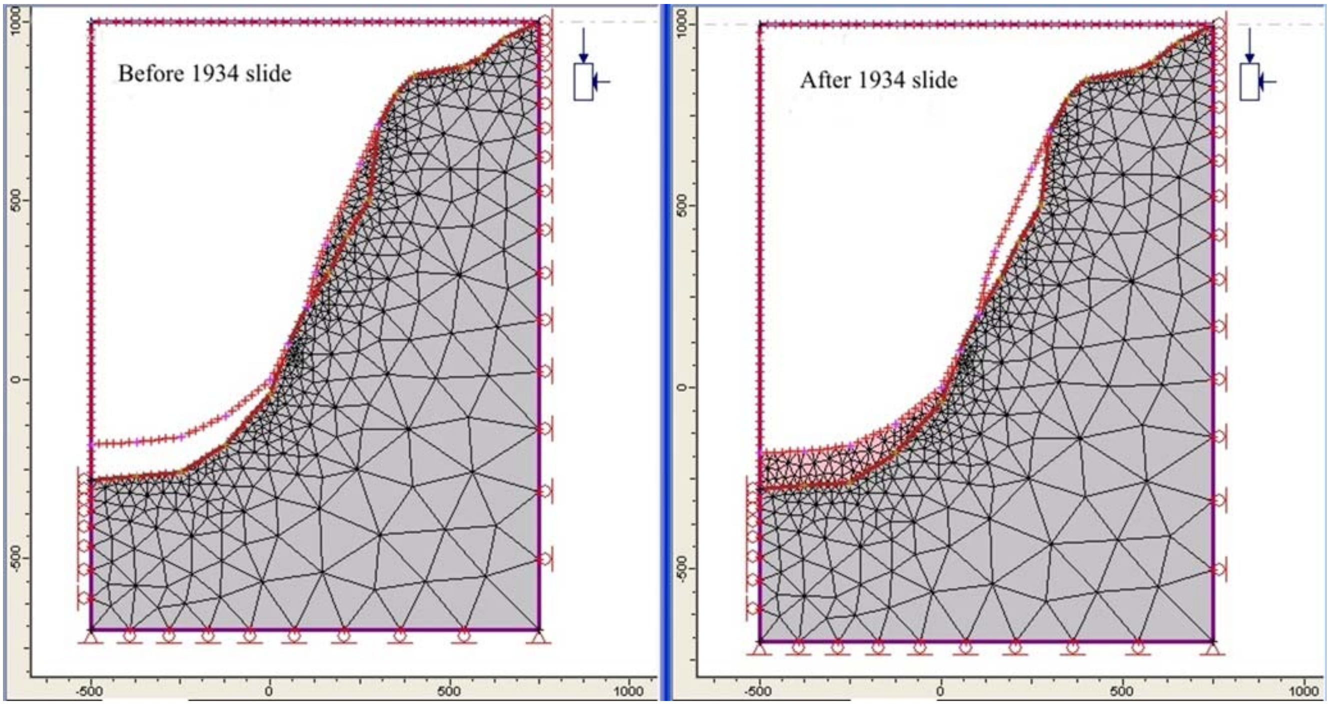


**Fig. 2** Heggura slope topography and Phase$^2$ FEM-mesh representing situation before (*left*) and after (*right*) the 1934 slide, respectively. *Horizontal* and *vertical lines* are in metres

weakening the rock mass and in forming exfoliation joints as a result of large deformations. This paper discusses the numerical analysis and the stress and displacement regime of the Heggura slope. The stress and displacement distribution along the slope topography before and after the slide are discussed, as well as the effect of the 1934 slide on the present stability situation. For the analysis, the Phase$^2$ 2D finite element numerical modelling tool has been used (Rockscience 2001).

## Input data and slope model for the analysis

The rock stress measurements and laboratory testing results for the mica gneiss from the Heggura tunnel project (Broch and Sørheim 1984) and the Hoek–Brown failure criterion (Hoek et al. 2002) have been used to estimate the required input data for the numerical analysis of the Heggura slope geometry. The available and estimated input data for the simulation are given in Table 1.

A two dimensional finite element model of the Heggura slope, representing two stages of events (before and after sliding), has been established for the analysis. The finite element model, including the 3 million m$^3$ slide volume, is shown in Fig. 2. An elastic rock material model with rock mass properties and field stress as given in Table 1 has been used for the simulation. The horizontal to vertical stress ratio is calculated by using the measured stresses from the tunnel project. After creating the numerical model and assigning the stress field and rock mass material properties from Table 1, the Phase$^2$ 2D finite element program was run for simulation of the modelled slope geometry. The results obtained from the simulation, giving the details on the stresses and displacement distribution patterns, are discussed in the following sections.

## Simulation results of the stresses and displacements

Based on Phase$^2$ computation and simulation, the distribution patterns of the major and minor principal stresses (sigma 1 and sigma 3, respectively) before and after the 1934 slide have been found to be as shown in Figs. 3 and 4. The squares in the figures represent fixed points along a reference line.

As shown by Figs. 3 and 4, there is a significant topographic effect on the stress distribution in the slope. Also, it is worthy of note that there are considerable changes in the magnitude of in situ stresses (both major and minor principle stresses) as result of the unloading of the huge rock mass represented by the 1934 major slide. Of particular interest for the future stability is the significant reduction of stress at the top of the slope.

**Table 1** Measured, laboratory tested and estimated input data for FEM-analysis

| Description | Symbol | Unit | Value | Remarks |
|---|---|---|---|---|
| Measured in situ | | | | |
| Magnitudes of rock stresses in the Heggura Tunnel | $\sigma_1$ | MPa | 24.8 | By 3D overcoring |
| | $\sigma_2$ | MPa | 9.3 | By 3D overcoring |
| | $\sigma_3$ | MPa | 6.6 | By 3D overcoring |
| Orientation of major principle stress | Direction | Degree | N222E | |
| | Inclination | Degree | 49SW | |
| Laboratory tested | | | | |
| Uniaxial Compressive Strength | $\sigma_{ci}$ | MPa | 67 | Intact rock |
| Young's Modulus (laboratory) | E | MPa | 10,000 | Intact rock |
| Poisson's ratio | $v$ | | 0.21 | Intact rock |
| Estimated/assumed | | | | |
| Horizontal by vertical stress ratio | $\sigma_h/\sigma_v$ | | 0.49 | By resolving $\sigma_1$ and $\sigma_3$ |
| Young's Modulus (rock mass) | $E_{rm}$ | MPa | 5,000 | 50% of lab result (Nilsen and Palmstrøm 2000) |
| Rock mass unit weight | $\gamma$ | $MN/m^3$ | 0.027 | |
| Estimated for rock mass based on Hoek-Brown Failure Criterion | | | | |
| Geological Strength Index | GSI | | 70 | For blocky rock mass |
| Constant (s) | $s$ | | 0.036 | $s = exp\left(\frac{GSI-100}{9-3D}\right)$ |
| Constant (a) | $a$ | | 0.50 | $a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$ |
| Material Constants (m) | $m_{b(peak)}$ | | 2.53 | $m_b = m_i \times \exp\left(\frac{GSI-100}{28-14D}\right)$ |

Figure 5 shows the overall displacement patterns of the slope resulting from the 1934 slide. The simulation results indicate that the maximum total and horizontal displacements amount to about 210 mm and 60 mm respectively as a result of unloading of the rock mass due to this major slide.

## Interpretation and discussion of the results

To further explore the extent of anisotropic stress conditions in the slope before and after the 1934 slide, the Phase$^2$ user defined input and output analysis on the

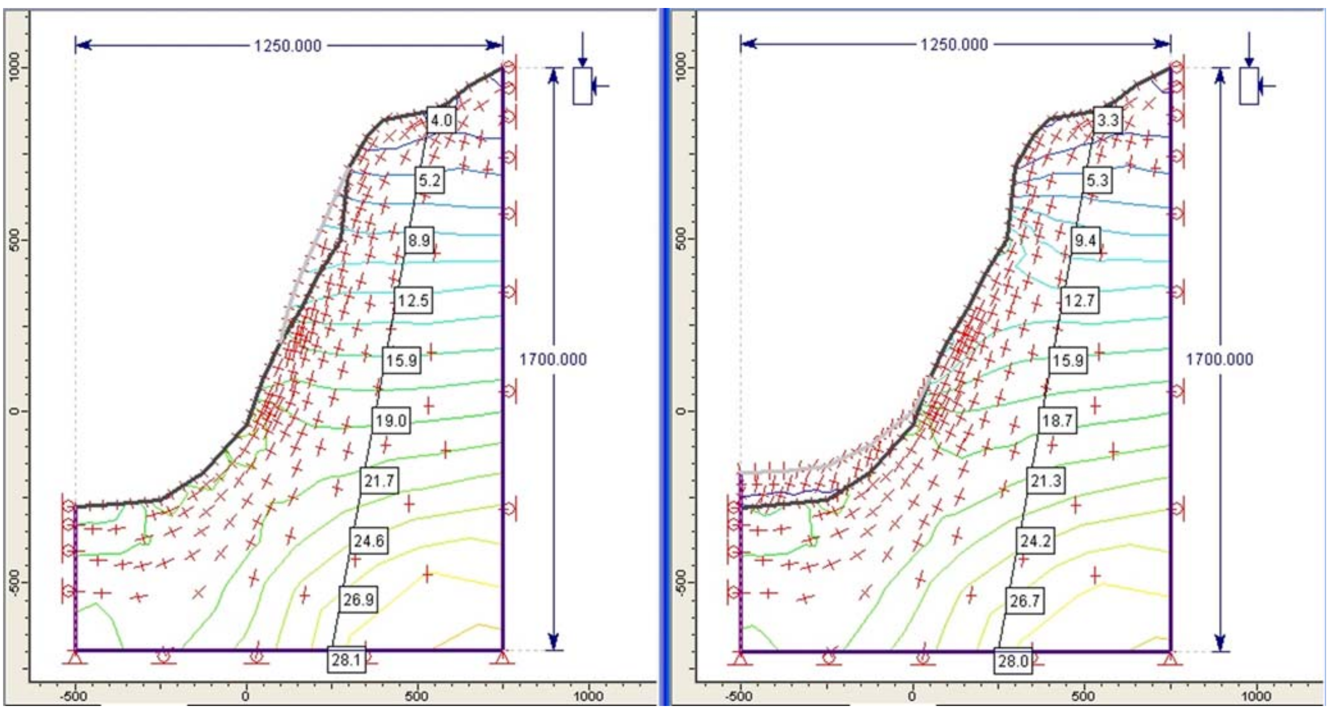


**Fig. 3** Major principle stress (Sigma 1) distribution in MPa in the slope before (*left*) and after (*right*) the 1934-slide, respectively. Lines with squares are reference lines with identical locations. *Horizontal* and *vertical lines* are in metres

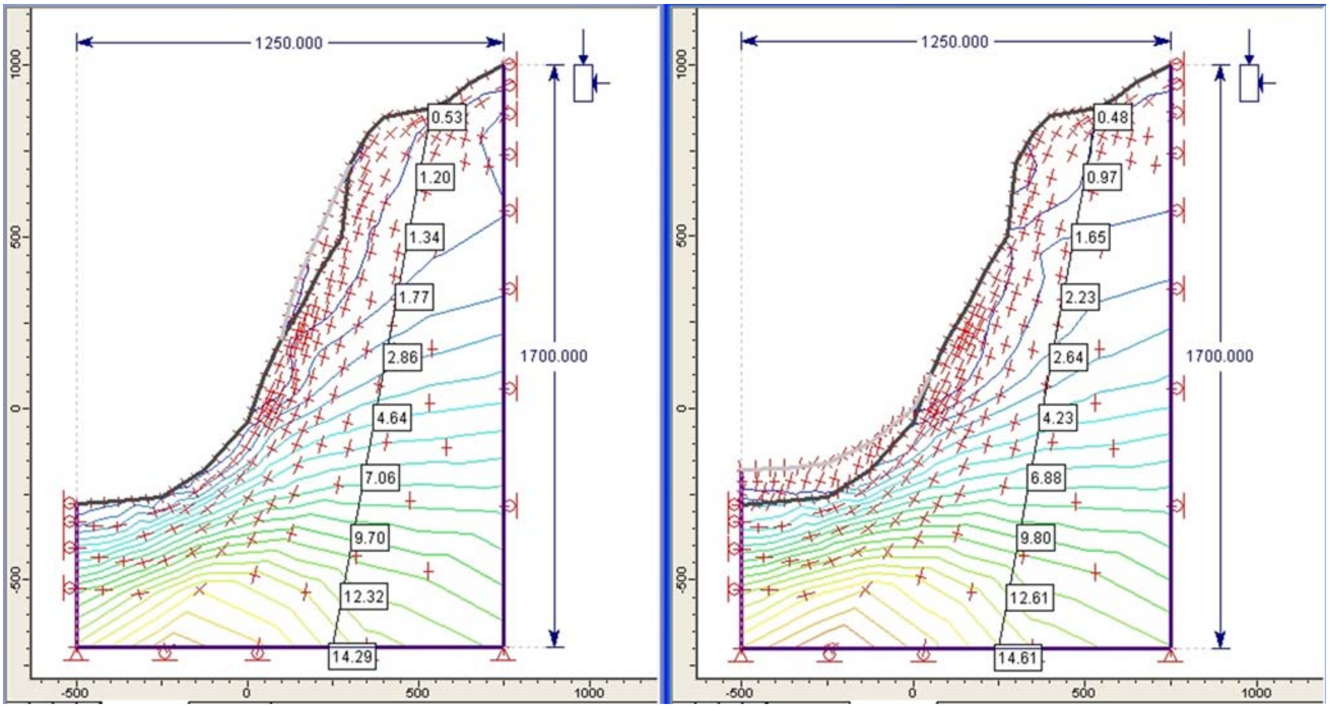


**Fig. 4** Minor principle stress (Sigma 3) distribution in MPa in the slope before (*left*) and after (*right*) the 1934-slide, respectively. *Lines* with squares as explained for Fig. 3. *Horizontal* and *vertical lines* are in metres

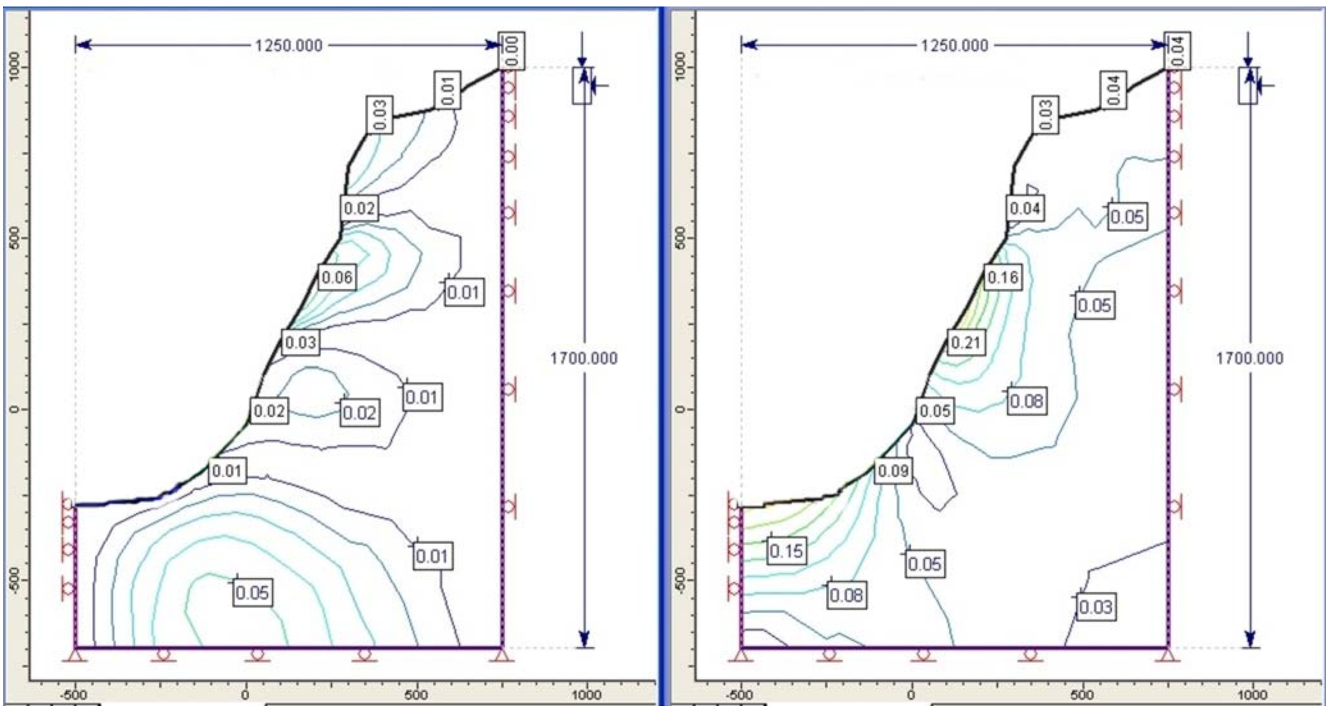


**Fig. 5** Horizontal (*left*) and total (*right*) displacements in centimetres of the Heggura slope as a result of the 1934-slide, respectively. Maximum displacements occur in the slide scar (note that the contour intervals are different for the two plots). *Horizontal* and *vertical lines* are in metres

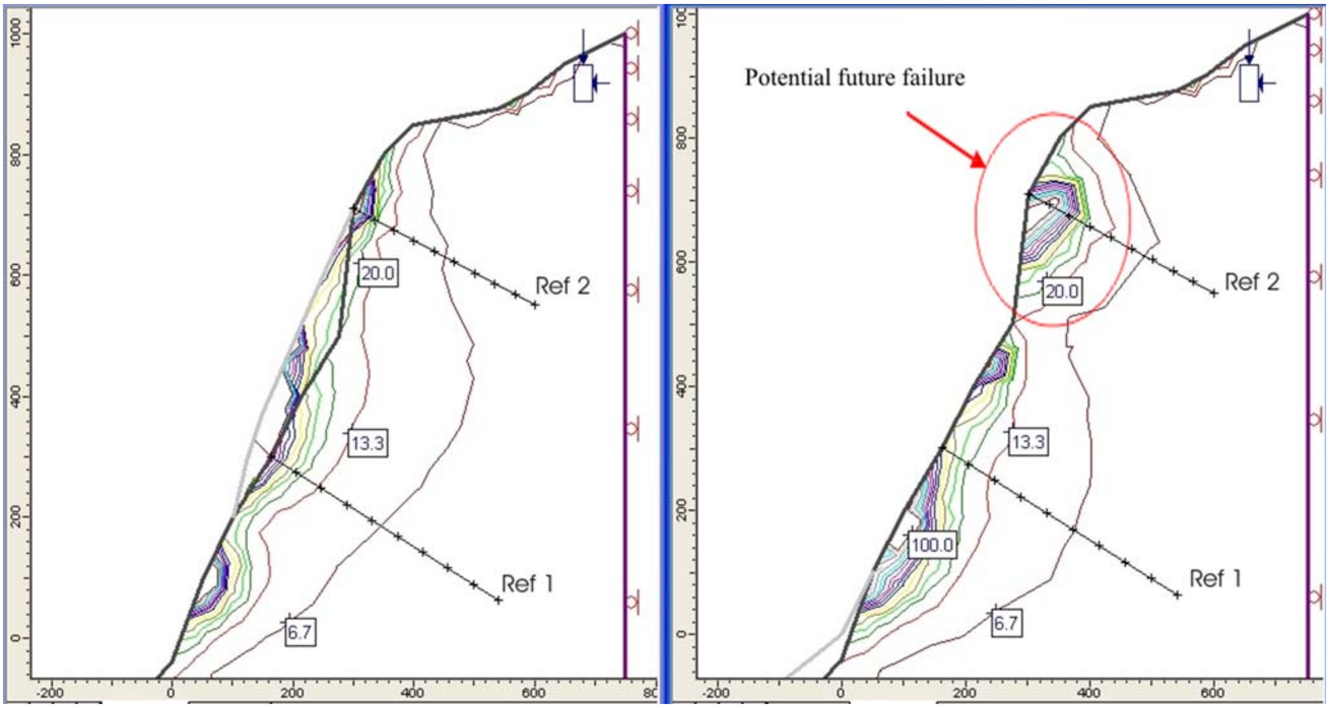


**Fig. 6** Ratio between major and minor principle stresses before (*left*) and after (*right*) the 1934-slide (spacing between contour lines: approximately 6.7 m). The respective reference lines have identical locations. *Horizontal* and *vertical lines* are in metres

ratio between major and minor principle stresses has been undertaken. The results of this simulation are shown in Fig. 6.

In Fig. 7, results of a further in-depth analysis of the extent of anisotropic stress condition are shown. The results refer to two reference lines which have been drawn perpendicular to the slope face as shown in Fig. 6; No. 1 from the foot of the 1934 slide block and No. 2 from the starting point of the tension joint of that slide.

The simulation results for the slope prior to sliding (Fig. 6 left and Fig. 7) clearly indicate that the stress ratio between the major and minor principle stresses is very high near the surface. Thus, a high degree of stress anisotropy existed along the slope geometry before the 1934 slide. This stress anisotropy probably caused considerable displacement and as a result of the displacement, formation of tension joints and reduction of the shear strength most likely took place and ultimately led to the 1934 major rock slide.

The unloading caused by the 1934 slide has further increased the stress anisotropy, particularly near the top of the slope (Fig. 6 right and Fig. 7) and as shown in Fig. 5, the displacements are of considerable dimen-

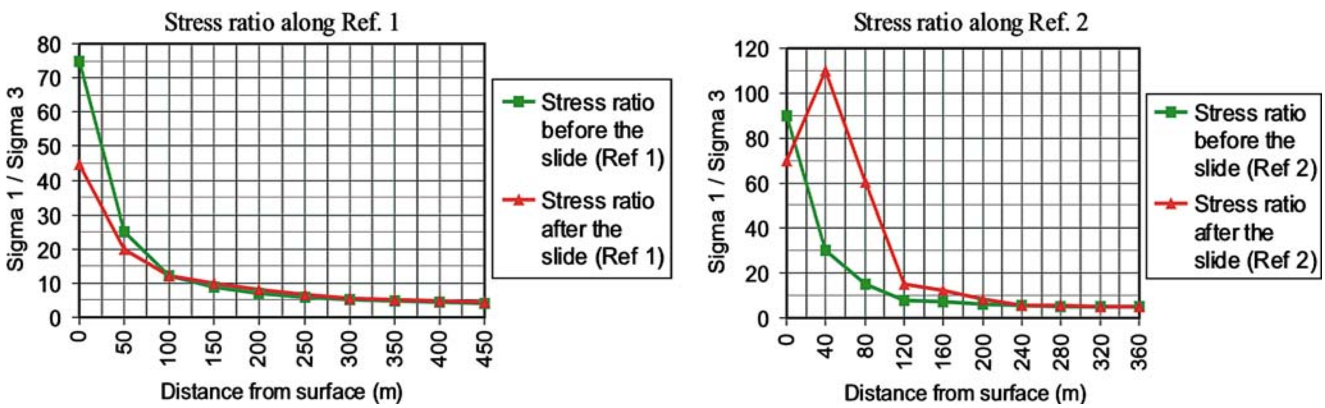


**Fig. 7** Stress anisotropy (ratio between major and minor principle stress) along the two reference lines shown in Fig. 6

sions. This highly anisotropic stress condition and the large displacements are believed to have great influence on the stability of the "new" slope geometry and may cause the development of new tensional joints with considerable opening. Such tensional joints may greatly increase the risk of new failures of the Heggura slope. According to Braathen et al. (2004), tension joints with considerable opening have recently been detected near the top of the Heggura slope.

## Conclusion

The Phase$^2$ numerical analysis of the Heggura slope has demonstrated that if reliable, good quality input data are available, valuable analysis of stresses and displacements as well as evaluation of possible slope failure can be achieved. In particular, the simulation results obtained from such analysis may provide valuable input for predicting the potential progressive development of tensional joints which may ultimately lead to future rock slope failure.

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## References

Bjerrum L, Jørstad FA (1968) Stability of rock slopes in Norway. Norwegian Geotechnical Institute Publication 79:1–11

Blikra LH, Longva O, Braathen A, Anda E, Dehls J, Stalsberg K (2004) Rock-slope failures in Norwegian fjord areas: Examples, spatial distribution and temporal pattern. In: Evans SG, Scarascia Mugnozza G, Strom A, Hermanns, RL (eds) Massive Rock Slope Failure: new models for hazard assessment, NATO Science Series, Kluwer, Dordrecht (in press)

Braathen A, Blikra LH, Berg SS, Karlsen F (2004) Rock Slope Failures of Norway; type, geometry, deformation mechanism and stability. Norwegian J Geol (NGT) 84: 67–88

Broch E, Sørheim S (1984) Experience from the Planning, Construction and Supporting of a Road Tunnel Subjected to Heavy Rockbursting. Rock Mech Rock Eng 17:15–35

Hoek E, Torres CC, Corkum B (2002) Hoek-Brown Failure Criterion. Proc North American Rock Mechanics Society Meeting, Toronto

Nilsen B, Palmstrøm A (2000) Engineering Geology and Rock Engineering. Norwegian Group for Rock Mechanics (NBG), Handbook No.2, pp 161–170

Rocscience (2001) PHASE$^2$ Manual, http://www.rocscience.com/products/Phase2.asp, 163p