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# Static and dynamic stability analyses of the 800m high Åknes rock slope, western Norway

Thesis for the degree philosophiae doctor

Trondheim, July 2008

Norwegian University of Science and Technology Faculty of Engineering Science and Technology Department of Geology and Mineral Resources Engineering



#### NTNU

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## Abstract

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Catastrophic rock slope failures have caused destructive tsunamis in Norwegian fjords. At the Åknes rock slope the tsunami generating potential is large due to the potential large volume involved in a possible catastrophic failure. In a worst case scenario, a volume of 35 - 40 million m<sup>3</sup> has been estimated for the about  $650,000m^2$  unstable slope. Widening of the upper tension fracture has been monitored since 1986, and rock slides on the western flank occurred in 1850 – 1900, 1940 and about 1960. Numerous rock slide deposits have been observed on the bottom of the fjord by which Åknes is located. An extensive investigation and monitoring campaign started in 2004. Åknes is in 2008 among the most investigated rock slopes in the world. An early warning system has been implemented.

Information on structural geology and geophysical methods were used to understand the structural geometry, to build a geological model and to constrain the area and the depth of the unstable rock mass. The unstable area was found to be constrained by a 800m long back scarp/tension fracture to the north, a gently dipping fault to the east, a daylighting sliding surface to the south and a steeply dipping fault to the west. Sliding surfaces were suggested to be sub-parallel to the slope surface and located along foliation parallel mica-rich layers of the gneisses. The sliding surfaces were further interpreted to daylight at different levels. The unstable area was sub-divided into four sub-domains, experiencing extension in the upper part and compression in the lower part. Maximum depth of the instability was estimated to be 65 - 70m in the western part. However, recent measurements in the upper borehole indicate movements down to 120m.

Barton-Bandis shear strength parameters intended for numerical stability computations using the Universal Distinct Elemen Code (UDEC – Version 3.10) were derived for the Åknes rock slope by field mapping and laboratory testing. Back-calculations of a 100,000m<sup>3</sup> rock slide which occurred on the western flank of Åknes about 1960 (1960-slide) were performed for evaluation of the data set. The limit equilibrium analysis showed that the Joint Roughness Coefficient has the greatest effect on the calculated safety factor of the slide. Probabilistic computations showed that the JRC stood out as the most important contributor to the total uncertainty over the whole set of variables, and that the computed failure probability of the 1960-slide was very high. This implies that the assumed input variables and the Barton-Bandis shear strength criterion are reasonable for the slide. The dataset was thus used for stability analysis of the Åknes rock slope.

The slope was divided into sub-areas (blocks) based on displacements measured at the slope surface. Discontinuous deformation analysis (DDA) used in the backward mode showed that 3-4 blocks in the upper half may be considered as potential sub-areas that may fail catastrophically. The lower half was divided into 2-3 blocks, but more limited data introduce more uncertainty into block definition in the lower part. 2D stability analyses were performed using UDEC. By varying fracture geometry, fracture friction and ground water conditions within reasonable limits based on site specific data a number of possible models were compared. The conclusions show that models which were unstable to great depths were in closer agreement with the Barton-Bandis shear strength parameters than models that were

unstable to smaller depths. The length (depth) of the outcropping fracture, along which shear displacements are shown to occur, plays an important role. A (shallow) slide at 30m, at which depth displacements have been measured, will reduce the stability at greater depths. Increased ground water pressure was demonstrated to be less critical for very deep slope instability compared to instability at shallower depth.

The seismic stability was analysed by using UDEC. The dynamic input was based on earthquakes with return periods of 100 and 1000 years, and in most models the input shear wave was a harmonic function (sine wave). Models with depths of the sliding surface up to 200m and with ground water conditions derived from site investigations were analysed, as well as models with ground water conditions assumed from possible future draining of the slope. Friction angles somewhat higher than the friction angles that were required for static stability were used in the dynamic analyses. The analyses indicate that an earthquake with a return period of 1000 years is likely to trigger sliding to great depth in the slope with the present ground water conditions and that the slope will remain stable if it is drained. The analyses also indicate that sliding is not likely to be triggered by an earthquake with a return period of 100 years with the present ground water conditions, although this conclusion is somewhat dependent of the depth of the instability.

## Preface and acknowledgements

The work presented in this thesis started in 2004 and is part of ongoing projects funded by the Research Council of Norway through the International Centre for Geohazards (ICG), Norwegian Geotechnical Institute (NGI), Geological Survey of Norway (NGU), Norwegian University of Science and Technology (NTNU), National Fund for Natural Damage Assistance, Møre & Romsdal County and the Aaknes/Tafjord project. I wish to thank all these organizations for their support.

I am particularly thankful to NGI represented by its managing director Suzanne Lacasse and its division director Frode Sandersen for letting me use the major part of my NGI working hours on this thesis, for the major part of the financing including financing of a stay at University of California, Berkeley.

I want to thank my main supervisor Professor Bjørn Nilsen for his kind co-operation and advice and for reading all the manuscripts and providing useful comments. Also, many thanks to Bjørn Nilsen for engaging his master students in the Åknes-project. Their work, in particular the work by Nicole Ragvin, have been of help.

Thank you to all my co-authors and special thanks to Professor Herbert H. Einstein who has been involved in all the manuscripts for which I am the main author. His comments and guidance have been of great help.

Finally, I want to thank my wife Tori for her support and enthusiasm during my PhD studies.

# **Modification**

The following modification has been made with respect to the thesis manuscript which was submitted for assessment: Paper IV has been replaced with the published version in Rock Mechanics Rock Engineering (DOI 10.1007/s00603-008-0005-1).

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## Note on contributions

#### General

Investigations on the Åknes rock slope have been conducted by a large number of people and organizations. I have used data made available to me through other peoples' efforts. The information below does not address all details regarding who did what the project. I have tried to address contributions that appear most important in the various papers. For all the papers I have had discussions with the co-authors in varying degrees on what to do and how to do it, i.e. planning and performance.

**Paper I: Evaluation of movement data and ground conditions for the Åknes rock slide.** Authors: Kveldsvik, V., Eiken, T., Ganerød, G. V., Grøneng, G., Ragvin, N.

I wrote the paper and did all the data analyses except analyses on fracture orientations which were performed by Nicole Ragvin under my guidance. Fig. 5 was modified from a figure made by Nicole Ragvin and Fig. 15 was made by Nicole Ragvin. Trond Eiken reviewed the text on photogrammetry. Herbert Einstein, Bjørn Nilsen and Lars Harald Blikra reviewed the whole paper. The core logging data were result of work carried out by Guri Venvik Ganerød, Guro Grøneng, Nicole Ragvin and myself. I analysed the data myself. The field work was carried out by myself assisted by Nicole Ragvin. Some data on fracture orientations were also provided by others.

# Paper II: Alternative approaches for analyses of a 100,000 m3 rock slide based on Barton–Bandis shear strength criterion.

Authors: Kveldsvik, V., Einstein, H. H., Nadim, F. and Nilsen, B.

I wrote the paper and did all the analyses assisted by Nicole Ragvin who performed data analyses on the Barton-Bandis parameters under my guidance. I did the field work assisted by Nicole Ragvin and, to a lesser extent, Guri Venvik Ganerød. Tilt testing for deriving the basic friction angle was carried out by Nicole Ragvin. The paper was reviewed by the co-authors and partly by Lars Harald Blikra.

#### Paper III: Geological model of the Åknes rock slide, western Norway.

Authors: Ganerød, G. V., Grøneng, G., Rønning, J. S., Dalsegg, E., Elvebakk, H., Tønnesen, J. F., Kveldsvik, V., Eiken, T., Blikra, L. H. and Braathen, A.

I participated in planning of the paper, planning of the field work and performing it. I reviewed the various draft versions of the paper and provided comments and suggestions to the main author, Guri Venvik Ganerød.

# Paper IV: Numerical analysis of the 650,000m2 Åknes rock slope based on measured displacements and geotechnical data.

Authors: Kveldsvik, V., Einstein, H. H., Nilsen, B. and Blikra, L. H.

I wrote the paper and did all the analyses. Most of the data on fracture orientations (Figs. 11 – 14) were collected during a field campaign led by Guri Venvik Ganerød, supplemented by data collected by myself. The paper was reviewed by Herbert Einstein, Bjørn Nilsen and partly Lars Harald Blikra.

#### Paper V: Dynamic analysis of the 800m high Åknes rock slope using UDEC.

Authors: Kveldsvik, V., Kaynia, A. M., Nadim, F., Bhasin, R., Nilsen, B. and Einstein, H. H.

I wrote the paper except for parts of the introduction that was written by Rajinder Bhasin. I did all the analyses except for the computations of Newmark displacements (Table 2 – Newmark) which were performed by Farrokh Nadim. The paper was reviewed by Amir Kaynia, Farrokh Nadim, Bjørn Nilsen, Herbert Einstein and partly, Rajinder Bhasin. The frequency and duration of the sine waves used as dynamic input were derived by Amir Kaynia and Farrokh Nadim. The "real" representation of a time history was derived by Amir Kaynia.

# Introduction

## Background

#### Large rock slides

Landslides due to massive rock slope failures represent a major geological hazard in many parts of the world and have been responsible for some of the most destructive natural disasters in recent history. Landslide volumes cover at least five orders of magnitude between  $10^5$  and  $10^{10}$  m3 (Evans et al., 2006). Secondary processes associated with massive rock slope failures include landslide-generated waves (tsunamis) and displaced water effects and those associated with landslide dams (e.g. Blikra et al., 2005, Blikra et al., 2006, Fritz et al., 2001, Müller-Salzburg, 1987, Govi et al., 2002). Landslide generated waves may reach large run-up altitudes. On 8 July 1958 an 8.3M earthquake triggered a rock slide of an estimated volume of 30 million m<sup>3</sup> into Lituya Bay, Alaska. The wave ran up to an altitude of incredible 524m, the highest wave run-up in recorded history, on a spur ridge in direct prolongation of the slide axis (Fritz et al., 2001).

In Norway, large rock slides represent one of the most serious natural hazards, as exemplified by the Tafjord disaster of 1934 when 3,000,000m<sup>3</sup> rock mass and scree material dropped into the fjord. The tsunami ran up to a maximum altitude of 62m, and several inhabited villages along the fjord were destroyed (Fig. 1) and 41 people were killed. In the 20<sup>th</sup> century 175 people lost their lives in three such events in a region in northern West Norway (Tafjord 1934 and Loen 1905 and 1936). Generally, the more destructive historical rock slides in Norway were destructive due their generation of tsunamis (Bjerrum and Jørstad, 1968, Jørstad, 1968).



**Fig. 1.** The damage caused by the 1934 tsunami in the village Fjøra in Tafjord is shown by photographs before (A) and after (B) the event. Photo from Astor Furseth.

NGU's (Norwegian Geological Survey) landslide database covers more than 3000 events, and it includes numerous large rock slides. Recently, rock slide deposits have been mapped in the counties Møre & Romsdal, Sogn & Fjordane and Troms (Fig. 2). Many of these have been visited in the field in order to document special features, while some have undergone detailed investigations in the form of geological mapping, georadar profiling, refraction-seismic profiling and excavations for dating of the deposits. Several fjords have been covered by swath bathymetry, and most of the fjords affected by large rock slides are covered by reflection seismic profiles (Blikra et. al 2006). Rock slide deposits in Storfjorden, along which the Åknes study area is located, are shown in Fig. 3.



**Fig. 2.** Historical, documented rock slides in two counties in western Norway (Sogn & Fjordane and Møre & Romsdal) and Troms county in northern Norway, NGU – Geohazard database. Altogether 31 of these recorded events have caused tsunamis. From Blikra et al., 2006.



Fig. 3. Rock slide deposits in Storfjorden, Møre & Romsdal. Modified after Blikra et al. (2005).

#### Geological control and triggering mechanisms

Initial failure may be preceded by observable slope deformation manifested in growth and widening of tension cracks, increased rock fall activity and increasing disaggregation of the initial failure mass on the slope (Evans et al., 2006). Slope deformations and dislocations have close relationships with pre-collapse creep (Varga, 2006), i.e. time-dependent deformation of material under constant load (Wyllie and Mah, 2004). Ter-Stepanian (1966) identified different types of depth creep in rock slopes, including planar depth creep on long slopes with strata dipping parallel to the slope and rocks having different rheological characteristics.

A rock slide in a natural slope can, excluding human actions, be triggered by earthquake, water pressure and its fluctuations, erosion, frost wedging, permafrost thaw, and for older events; stress release and removal of lateral support when glacier ice melts down. The geometry of the rock slide may often be controlled by predominant rock structures such as weak layers, unfavourably orientated fractures and foliation. The shear strength of the final failure surface may have been reduced over time before the catastrophic failure as a result of

weathering, breakdown of fracture asperities (abrasion) and progressive failure in intact rock bridges (fracture propagation) (Eberhardt, et al., 2004a) and these processes may be observed as pre-failure slope deformation. Pre-failure deformations have been measured and documented for many rock slides, e.g. the disastrous 270 million m<sup>3</sup> Vaiont landslide (Petley and Petley, 2006), which was triggered by the combined effects of a rising reservoir and increases in piezometric levels as a result of rainfall (Hendron and Patton, 1987). Pre-failure deformations were also observed prior to the Tjelle, Tafjord and Loen events in Norway (Jørstad, 1968, Bjerrum and Jørstad, 1968). Thus, monitoring of possible slope deformation is essential for the evaluation of the stability of large rock slopes and for hazard assessments. Before a creeping slope fails catastrophically, it will undergo an acceleration phase, which can be used to predict time to failure (Voight, 1989, Petley et al., 2002, Kilburn and Petley, 2003, Crosta and Agliardi, 2003, Fukuzono, 1985). Kilburn and Petley (2003) showed that a linear trend in inverse rates of horizontal slope movement versus time was apparent at least 30 days before the catastrophic collapse of Mt. Toc into the Vaiont reservoar, which could have been used to forecast the collapse.

#### Methods for stability analysis of rock slopes

Modes of possible slope instability in a fractured rock slope are often identified by plotting fracture and slope face orientations on a stereonet, i.e. stereographic analysis. In the simplest case of a possible plane failure this would appear in a stereonet as a fracture (or concentration of fracture orientations) striking parallel to the slope face and dipping out of the slope face. By stereographic analysis of a specific rock slope with various fracture and slope face orientations, some possible modes of slope instability can be identified, i.e. plane failure, wedge failure, toppling failure and circular failure, the latter implying that fractures are randomly orientated. Once the mode of slope instability has been identified on the stereonet, the same diagram can also be used to examine the direction in which a block will slide, and by adding friction cones, give an indication of the stability conditions, which is known as kinetic analysis (Wyllie and Mah, 2004). The stereographic and kinetic techniques are a useful starting point of rock slope stability analysis, and would often be followed by detailed stability analyses.

Limit equilibrium methods are still frequently used methods in surface rock engineering, also for major rock slope instabilities, although in many cases, major rock slope instabilities often involve complex internal deformation and fracturing that are not allowed for in rigid block assumption adopted in most limit equilibrium back-analyses (Stead et al. 2006). After the Vaiont landslide, many different two-dimensional limit equilibrium analyses using methods of slices were performed. The friction angles required for stability back-calculated from these analyses range from 17.5° to 28°, whilst strength test data on the clay material along the failure surface show friction angles ranging from 5° to 16° with an average value around 12° (Hendron and Patton, 1985 and 1987, Sitar et al. 2005). Sitar et al. (2005) concluded that, since the slope had been at least marginally stable for quite some time prior to the failure, there were factors controlling the stability that were not accounted for in these twodimensional limit equilibrium stability analyses as the available friction angles along the failure surface were lower than the computed friction angles required for stability. Hendron and Patton (1985) suggested that the discrepancy between the required and available friction could be due to three-dimensional effects introduced as a result of high friction along the lateral margins of the slide and, internally, between slices within the slide mass. In an analysis of a rock slope with internal dilatation Martin and Kaiser (1984) showed that if the mode of failure requires the existence or creation of internal shears to accommodate large internal slide mass distortion, then these internal displacements are required to allow motion along the basal

slip surface, and that traditional limit equilibrium methods often underestimate the factor of safety when used on slopes with this failure mechanism. As a result, back-analysis may overestimate the available shear resistance on the basal slip surface as the contribution from the internal shear resistance is not accounted for.

Continuum codes (e.g. finite element and finite difference) are often used for numerical modelling of soil slopes, and they are also used for rock slopes. Continuum codes assume material is continuous throughout the body and discontinuities (fractures) are represented implicitly as interfaces between continuum bodies, with the intention that the behaviour of the continuum model is substantially equivalent to the real fractured rock mass being represented. Typical continuum-based models may have less than ten non-intersecting discontinuities (Wyllie and Mah, 2004). Early numerical analysis of rock slopes were predominantly undertaken using finite element codes, and continuum codes remain in routine use in engineering landslide investigations and are most appropriate in the analysis of slopes involving weak rock/soils or rock masses where failure is controlled by the deformation of the intact material or through a restricted number of discrete discontinuities such as a bedding plane or fault (Stead et al. 2006). Continuum codes are also applicable if the rock mass is highly fractured and it can be assumed that it behaves like a continuum.

In recognition of the controlling influence that fracturing has on complex slope deformation, discontinuum discrete element codes are being used increasingly for numerical modelling. In discrete element codes discontinuities are represented explicitly, that is, the discontinuities have a specific orientation and location. Two principal methods are in use, the distinct element (Hart, 1993) and discontinuous deformation analyses - DDA (Shi and Goodman, 1989) of which the former is most commonly used in engineering practice (Stead et al., 2006). Some recent examples of discrete element analyses are those by Seagalini and Giani (2004) who used the Universial Distinct Element Code - UDEC (Cundall, 1980) to analyse the evolutionary mechanisms of a 30 million cubic metres rock slide (the Randa rock slide), Bhasin and Kaynia (2004) who used UDEC to estimate the potential failure volume of the rock mass of a 700m high slope under static and dynamic forces and Sitar et al. (2005) who used DDA to analyze two typical examples of slope failure and demonstrated that accurate representation of the discontinuity geometry is essential for the identification of the kinetically correct failure modes. Sitar et al. (2005) also demonstrated for the Vaiont landslide that the required friction angle increased with the number of slices (vertical fractures from the slope surface to the failure surface) in their DDA models until there were enough fractures concentrated around the slope break that most of the kinetic constraints were removed. Progressive failure of intact rock and fracture propagation which may be an important failure mechanism in fractured rock slopes, have been modelled with the hybrid finite-/discrete element code ELFEN (Eberhardt et al., 2004a, Eberhardt et al., 2004b and Stead et al., 2006).

Fig. 4 illustrates three levels of landslide analysis, including the methods described above.



Fig. 4. Flowchart illustrating three levels of landslide analysis and the modes of translation/rotational failure they apply to. From Stead et al., 2006.

#### Selection of study area

Originally, it was planned to study two to three sites in Norway where large rock slides are feared to occur in the future. The initial site inspections in 2004 included four of the sites which are described in Braathen et al. (2004), including the Åknes rock slope (Fig. 5 and 6). Shortly after the site inspections it was decided to focus on the Åknes rock slope for basically four reasons:

- 1. The Åknes rock slope poses perhaps the largest threat of the potential study areas as several communities are located within the possible tsunami hazard zone and the fjord is one of Norway's most visited tourist attractions. Thus, studying the Åknes rock slope appeared more important than studying other sites.
- 2. Funding for investigations with the primary goal of implementing an early warning system was made available by the Norwegian government. The Åknes- Tafjord project was established with its permanent staff and economical resources to conduct investigations carried out by a large number of organizations. It thus became clear that more data would be made available from different sources than at any other possible study area.
- 3. Despite of the quite steep terrain and large area a substantial part of the slope is accessible on foot, and quite suitable for engineering geological field work.
- 4. Data on slope displacements existed from 1986 onwards.



Fig. 5. Map of Storfjorden showing the location of the Åknes rock slope.



**Fig. 6.** Overview of the Åknes rock slope. The white lines indicate the contour of the unstable area as derived through various investigations (slightly modified after Derron et al. 2005). The length of the "top scarp"/upper crack is about 800m. The length along the dip direction of the slope inside the assumed unstable area is about 1100m. U, M, L: Upper, middle and lower borehole sites.

#### **Objectives**

The objectives of this thesis were to:

Contribute to the understanding and knowledge of the Åknes rock slope with emphasis on:

- Geology
- Boundary of the unstable area
- Displacements
- Stability
- Response to earthquakes

Contribute to the implementation of the early warning system at the Åknes rock slope including planning of future monitoring and investigation.

Contribute to numerical modelling of natural rock slopes by using existing codes to:

- Demonstrate a novel application of the discontinuous displacement analysis (DDA)
- Demonstrate an overall method/approach for evaluating the static and dynamic stability of a natural rock slope by using data from the Åknes site
- Demonstrate how to collect and analyse input data for numerical modelling
- Investigate the applicability of the Barton-Bandis shear strength criterion in large rock slopes

#### Methods

Data from field mapping, core logging and a number of data collection techniques on displacements and sub-surface conditions have been used of in this study. The various data collection, data analysis and computation methods are described in the papers. The overall approach for evaluating the stability of the Åknes rock slope is illustrated in Fig. 7. The arrows going both ways between "Geometry" and "Numerical modelling – DDA" illustrate that data on geometry (assumed sub-block boundaries) were used in the DDA modelling and results from the DDA modelling were used to refine the sub-block boundaries.



**Fig. 7.** Flow chart for stability analysis of the Åknes rock slope. Abbreviations: DDA – Discontinuous Deformation Analysis, UDEC – Universal Distinct Element Code.

# Brief presentation of the papers

#### Paper I

Kveldsvik, V., Eiken, T., Ganerød, G. V., Grøneng, G., Ragvin, N. (2006). Evaluation of movement data and ground conditions for the Åknes rock slide. In: Proceedings of International Symposium on Stability of Rock Slopes in Open Pit Mine and Civil Engineering, p. 279 – 299. The South African Institute of Mining and Metallurgy (SAIMM).

The tsunami generating large rock slides of Norway and the tsunami generating potential of a possible catastrophic failure at Åknes are introduced. Most of the Åknes investigations that had been performed until 2006 are summarized. The monitoring data showing widening of the upper crack and photogrammetric data covering a larger portion of the slope are analysed. The

conclusions are that the displacement rates have been quite steady during the period of the monitoring and that the upper (northern) western part has moved more than the rest of the slope. Further, the ground conditions are analysed, mainly based on field mapping and core logging. It is concluded that the instability is controlled mainly by the slope sub-parallel foliation, and that the weaker biotitic gneiss may also play a role. The instability appears to be restricted to maximum depths of about 60m, which is 10 - 20m below the measured ground water table.

### Paper II

Kveldsvik, V., Einstein, H. H., Nadim, F. and Nilsen, B. (2007). Alternative approaches for analyses of a 100,000 m<sup>3</sup> rock slide based on Barton-Bandis shear strength criterion. Landslides, http://ejournals.ebsco.com/direct.asp?ArticleID=414B898789CF80825C7D, DOI 10.1007/s10346-007-0096-x.

The purpose is to evaluate a data set, collected through field work and laboratory tests, with respect to further use in stability analyses of the Åknes rock slope. Data collection and analysis focus on input parameters for the Barton-Bandis shear strength criterion, which are used for back-analyses of a 100,000m<sup>3</sup> rock slide that has occurred on the western flank of Åknes. Limit equilibrium analyses (deterministic and probabilistic) and numerical modelling (UDEC) are used for the stability analyses. The results show that the use of the Barton-Bandis shear strength criterion with the collected input parameters, are very reasonable for the rock slide having occurred, and that the Joint Roughness Coefficient (JRC) has the greatest effect on the computed stability of the rock slide. JRC also stands out as the most important contributor to the total uncertainty over the whole set of variables.

#### Paper III

Ganerød, G. V., Grøneng, G., Rønning, J. S., Dalsegg, E., Elvebakk, H., Tønnesen, J. F., Kveldsvik, V., Eiken, T., Blikra, L. H. and Braathen, A. (2008). Geological model of the Åknes rock slide, western Norway. Engineering Geology, DOI 10.1016/j.enggeo.2008.01.018. In print.

The focus is on structural geology and the usage of geophysical methods to interpret and understand the structural geometry of the Åknes rock slope. A geological model of the site is suggested. The unstable area is sub-divided into four sub-domains, experiencing extension in the upper part and compression in the lower part. Sliding surfaces are suggested to be subparallel to the slope surface and located along foliation parallel mica-rich layers of the gneisses. They are further suggested to daylight at different elevations.

#### Paper IV

Kveldsvik, V., Einstein, H. H., Nilsen, B. and Blikra, L. H. (2008). Numerical analysis of the 650,000m<sup>2</sup> Åknes rock slope based on measured displacements and geotechnical data. Rock Mechanics Rock Engineering. Accepted April 2008.

Displacements from different periods, measured by different techniques, are used to subdivide the unstable area into sub-blocks (sub-areas) using discontinuous deformation analysis (DDA) in a backward mode. The initial block boundaries are based on the geological model proposed in Paper III. It is shown that the upper (northern) half of the unstable slope can be sub-divided into three to four blocks that may fail catastrophically and that the lower half can be divided into two to three blocks of which one is shown to have been moving insignificantly from 2004 to 2006. The block models derived from DDA computations are somewhat different than the block model (sub-domain model) proposed in Paper III. Stability analyses using UDEC are performed on a cross section through the slope with a geometry including two outcropping fractures (possible daylighting of the sliding surfaces); one near the middle of the assumed unstable area and one at the lowest part (the toe). Fracture friction angles, possible maximum depth of instability and the location of the ground water table are varied within assumed reasonable limits based on site specific data. The conclusions show that models that are unstable to great depths (up to 200m) are in closer agreement with the shear strength parameters derived in Paper II than models that are unstable to smaller depths. A (shallow) slide at 30m, at which depth displacements have been measured, is demonstrated to be less critical for very deep slope instability compared to instability at shallower depth.

#### Paper V

Kveldsvik, V., Kaynia, A. M., Nadim, F., Bhasin, R., Nilsen, B. and Einstein, H. H. (2008). Dynamic analysis of the 800m high Åknes rock slope using UDEC. Int. J. of Rock Mechanics and Mining Science. Submitted April 2008.

The seismic stability of the Åknes rock slope is analysed using UDEC. The dynamic input is based on earthquakes with return periods of 100 and 1000 years. In most models the input shear wave is a harmonic function (sine wave) with frequency and number of cycles of motions to represent equivalent number of cycles with maximum acceleration of the earthquake motions expected in this region. Models a with maximum possible depth of sliding equal to 200m and with ground water conditions that are derived from the site investigations are analysed, as well as models with ground water conditions assumed from possible future draining of the slope. Friction angles somewhat higher than the friction angles that are required for static stability are used in the dynamic analyses. The analyses indicate that an earthquake with a return period of 1000 years is likely to trigger sliding to great depth in the slope at the present ground water conditions and that the slope will remain stable if draining is implemented. The analyses also indicate that sliding is not likely to be triggered by an earthquake with a return period of 100 years at the present ground water conditions.

# Discussion

#### Friction angles of fractures

In Paper IV the static (quasi-static) modelling was performed by applying gravity in the model in question and run the model until the equilibrium criterion of UDEC was reached, or until it was clear that equilibrium would not occur. The results for a model with fixed geometry, possible maximum depth of sliding and ground water table were the friction angles required for equilibrium to occur (limiting friction angles) and the depth of instability for friction angles somewhat lower than the limiting friction angles. The friction angles of the foliation fractures were in most models made dependent on the estimated effective normal stress in order to incorporate the non-linearity of the Barton-Bandis shear strength criterion, implying that the friction angles decreased with increasing depth in the models (Fig. 8). This method implied that possible anomalously low friction angles that could be anticipated from the diamond drilled boreholes, i.e. the observed sections of core loss / crushed core, were not considered in the modelling. The observed core loss / crushed core sections may have been caused partly or entirely by the ongoing creep (Paper I). Numerical modelling is thus used to investigate where instability occurs not considering anomalously weaknesses in the rock mass that may have existed before the creep started and not considering the possible degradation of

shear strength caused by creep. Considering anomalously weak layers / low friction angles in the numerical models would pre-define the depth of instability, and the modelling could then for example focus on analysing fracture geometry that could correlate with an anomalously low friction at say 50m. Estimating the anomalously low friction angle would involve large uncertainties due to the following reasons:

- Data on friction angles for poor rock mass quality of Åknes are very limited. Grøneng et al. (2008) succeeded in collecting three samples of gouge from the assumed toe zone of the unstable area suitable for tri-axial testing. Their tests yielded friction angles of 18°, 23° and 35°.
- 2. Core drilling has been carried out at three locations (Fig. 6). The observed assumed low-friction sections in the boreholes combined with the limited exposure of the assumed toe zone form a very limited basis for estimating friction angles along the whole surface on which creep take place. The surface may consist of a combination of poor quality low friction material and higher friction rock-to-rock contact sections.

The large uncertainties imply that selection of friction angle would be rather incidental. Thus, using the Barton-Bandis parameters for estimating friction angles at various depths is believed to be more consistent and to provide more insight of the possible behaviour of the slope through the numerical modelling.



Fig. 8. Active friction angle vs. depth at different JRC for a fracture with dip 35° at depth to ground water equal to 40m. From Paper IV: Fig. 15.

One of the conclusions in Paper IV was that the shear strength parameters derived in Paper II would predict instability down to great depths. The limiting friction angles of foliation fractures corresponded to JRC of 6-8 for models which became unstable to depths 70m to 200m at friction angles somewhat lower than the limiting friction angles. Models that became unstable at depths less than 70m showed limiting friction angles corresponding to JRC smaller than 6. The mean JRC of foliation fractures in Paper II was 7.8 while the back-calculated limiting JRC was 6 when all the other parameters were assigned mean values and dry conditions were assumed. As described earlier, back analysis using limit equilibrium methods

may overestimate the available shear resistance on the basal slip surface as the contribution from the internal shear resistance is not accounted for. If internal shear resistance contributed to the total shear resistance for the 1960-slide this implies that a JRC of 6 for the slip surface is too high. Assuming a limiting JRC of 6 is too high for the 1960-slide, and that the limiting JRC derived from 1960-slide is representative for the Åknes rock slope, imply that models that were unstable to more shallow depths may be more appropriate than models that were unstable to greater depths.

However, assuming the available friction is not fully mobilised (Paper V) would lead to an increase in the estimated friction angles for the Åknes rock slope. The back-calculated limiting JRC would increase from 6 if water pressure was assumed at the slip surface of the 1960-slide and water may have triggered the slide as discussed in Paper II.

It is assumed that internal shears probably are important in the complex Åknes rock slope (Paper III), which may not be fully accounted for in the simplified fracture geometry applied in the numerical models (Paper IV and V). This assumption implies that less of the total shear resistance is available on the basal slip surface than derived in the numerical modelling. A more accurate representation of the real and more complex fracture geometry of the Åknes rock slope in the numerical models could probably have resulted in lower limiting friction angles for the foliation fractures as the contributions from internal shears would have increased.

The conclusion based on the above is that the uncertainties in limiting friction angles go both in favour of higher and smaller values.

#### Depth of instability and location of the toe zone

One of the upper boreholes (Fig. 6 and 9) has been instrumented in the interval 83 - 133mm. The deepest displacement has been measured at 120m (Paper V) which corresponds to altitude ~540m above sea level (m.a.s.l.). The assumed daylighting sliding surface in the middle of the unstable area (Fig. 9) is located at altitude ~530m. The horizontal distance between the upper borehole and the assumed daylighting surface is ~190m. The three measuring points in the lowermost part of Block 8 showing dip of the displacement vectors from 23° to 31° are located at elevations ~570m.a.s.l. to ~660m.a.s.l. It should be noted that the altitude is the least accurate in the measurements. Consequently, the dip should not be taken too literally. A trend is although clear: the dip of the displacement vectors are generally considerably steeper in the uppermost part of the slope than near the assumed daylighting sliding surface in the middle of the slope, indicating a steeper dip of the sliding surface in the uppermost part.

From the altitudes it can be concluded that the displacement measured in the upper borehole at altitude ~540m.a.s.l. cannot take place at the sliding surface daylighting at altitude ~530m.a.s.l., ~190m south of the upper borehole, unless the friction angle of this sliding surface at 120m depth is anomalously low (which will be shown later). If the movement at ~540m.a.s.l. is connected to a sliding surface daylighting in the toe zone, the average dip of that surface between the upper borehole and the toe zone is 28°. This is reasonable in view of a model with slope sub-parallel sliding surfaces daylighting due to folding that has caused more gentle dips of the foliation. Assuming the absence of anomalously low friction at 120m depth and the existence of anomalously low friction at moderate depth, the assumed dayligthing sliding surface in the middle of the slope must be connected to a movement taking place at moderate depth, and the relatively gentle dips of the displacement vectors in the

lowermost part of Block 8 may be connected to the assumed daylighting sliding surface. An attempt to illustrate the situation is shown in Fig. 10 with the steepest dip of displacements in the upper part caused by a steep but not vertical fracture, slope parallel displacement dip caused by a slope parallel fracture and more gentle displacement dip caused by the daylighting of sliding surfaces of dip 20°. It should be noted that the dip of the lowest displacement vector in the upper layer of the sliding blocks would have been smaller if relatively more of the total displacement had occurred on the shallow sliding surface compared to the deep sliding surface.



**Fig. 9.** Initial block model and annual average slope displacements 2004-2006. Red: Total station, blue: GPS, green: extensometer. Red and blue numbers: dip of the displacement vector from the first measurement to the last measurement. The last measurements were performed during autumn 2007 for GPS and during autumn 2006 for the total station. Scale: cm for displacements, m for block model. The red letters show the locations of the boreholes (Upper, Middle, Lower). The model measures 1100m × 1140m. Black broken line: modified block boundary after DDA computation (Fig. 33, Paper IV). Green broken line: profile for numerical modelling shown in Fig. 11. This figure is a modified version of Fig. 6 in Paper IV.



**Fig. 10.** Conceptual model of a slope dipping 35° and sliding taking place at two depths. The red lines show how the monitoring points have displaced during the DDA computation. The red dots show the position of the monitoring points before block movements occurred in the model.

As indicated above, sliding can take place at 120m depth / altitude  $\sim$ 540m.a.s.l. at the upper borehole on a sliding surface daylighting at altitude ~530m.a.s.l. The model showing this is illustrated in Fig. 11, and the location of the profile is shown in Fig. 9. The model is similar to the upper half of most models analysed in Paper IV regarding geometry of the slope surface, the location of the upper tension fracture, the location of the daylighting fracture and the location of the ground water table, i.e. 40 - 50m below the slope surface. It should be noted that kinematic constraints have been reduced compared to the models analysed in Paper IV in that two vertical fractures have been added just to the left of the lower break point. The limiting friction angle for the non-vertical fractures in the model equalled 25° when the friction angle of the vertical fractures were set to 49.5°, i.e. the same friction angle that was used for the vertical fractures in most models analysed in Paper IV. A limiting friction angle of 25° is equal to the residual friction angle calculated for the foliation parallel fractures in Paper II. Thus, the model shown in Fig. 11 corresponds to movements taking place at 120m at the upper borehole at a surface daylighting in the middle of the slope with the friction at the foliation parallel fractures degraded to the residual friction angle. The model also implies that movement at 120m depth by the upper borehole is possible without movements taking place in the lower half of the slope where movement measurements have not been performed, i.e. Block 10 and the western part of Block 11 (Fig. 9).



**Fig. 11.** UDEC model of the upper half of the Åknes rock slope. The arrows show computed displacement vectors in the model. The blue broken line shows the ground water table.

In Paper III, possible daylighting of the sliding surface below sea level is discussed. If the assumed toe zone is not correctly located in Fig. 9, other daylighting above sea level is also possible. The location of the toe zone may be questioned based on data from the lower boreholes. There is a section of very poor rock at about 40m depth in both the boreholes at the lower location (Fig. 12), corresponding to altitude ~200m.a.s.l. The assumed toe zone downslope of the lower borehole site, a horizontal distance of 40 - 60m, is located at altitude 210 - 220m.a.s.l. If the poor rock at ~200m.a.s.l. is connected to a sliding surface daylighting in the toe zone, the sliding occurs uphill at an angle of  $10 - 20^{\circ}$ , meaning that very low friction angle is required. It is thus possible that the toe zone may be located more to the south (downslope) in the eastern part of the slope than indicated on Fig. 9, and that the assumed toe zone (Fig. 9) may represent a less deep sliding surface. Assuming the toe zone is located at altitude 100m.a.s.l., i.e. where most springs are being observed (Paper III and IV), then the possible sliding surface at 40m depth in the lower boreholes has an average dip of  $28^{\circ}$  between the lower borehole location and the toe zone, i.e. a more reasonable dip in view of the geological model.



Fig. 12. Lower boreholes L1 and L2. Cumulative length of core loss / crushed core.

#### Internal shears

A modified version of the block geometry shown in Fig. 11 was used for demonstrating an effect of internal shears as discussed previously: One vertical fracture was added 2m to the left of the break point at 150m (along the horizontal axis) for the purpose of reducing the kinematic constraints around this breakpoint. Then the model was computed with friction angle of the vertical fractures equal to 49.5° and 25° respectively. With a friction angle of 49.5° the limiting friction angle of the non-vertical fractures equalled 26°, i.e. an increase of 1° compared to the model shown in Fig. 11 without the extra fracture near the breakpoint at 150m. The limiting friction angle of the non-vertical fractures increased to 30° when the friction angle of the vertical fractures in this model. These results indicate that varying the number of vertical fractures, their location and their friction angle in the larger models analysed in Paper IV might have given some difference in limiting friction angles (i.e. limiting JRC) of the foliation parallel fractures in these models.

#### Upward movements

Upward movements in the lower part of the slope are discussed in Paper III and Paper IV based on displacement data from autumn 2004 to autumn 2006. GPS measurements performed autumn 2007 (Eiken, 2008) show that a systematic trend of upward movements does not exist (Fig. 13). It is concluded that atmospheric conditions probably cause the slight changes in altitude. In all, the updated GPS measurements show that the lower part of Block 11 (Fig. 9) still moves negligibly.



Fig. 13. Changes in altitude from GPS measurements in the lower part of the slope.

#### Ground water conditions

As discussed in Paper III the ground water conditions may be quite complex. In the numerical models (Paper IV and V), this potential complexity is not considered, as the water pressure is assumed to increase linearly with depth below the measured ground water table (or the model assumes dry conditions) and there is a considerable difference in the limiting friction angles dependent on whether ground water is included in the model or not. Estimation of anomalous water pressure at different depths would be guesswork since concrete data on sub-surface water pressures do no exist. Thus, the approach used in the numerical modelling is believed to be more consistent and to provide more insight of the possible behaviour of the slope.

# **Main conclusions**

The main conclusions of this thesis are:

The boundaries of the unstable part of the slope can be identified at the surface and the area totals about  $650,000m^2$ .

The upper half of the slope is sub-divided into 4-5 blocks (sub-areas) of which 3-4 blocks may be considered as blocks that may fail catastrophically. The lower half is sub-divided into 2-3 blocks, of which a considerable part moved negligibly in the period 2004 - 2006.

In the western part, based on geophysical data, the maximum depth of the unstable area is estimated to be 65 - 70m, i.e. about 20m below the ground water table. Larger maximum depth is shown by borehole deformation measurements and also indicated by numerical modelling. The maximum depth is yet to be derived.

The average displacement rates per year across the upper tension fracture have been quite steady since the monitoring of the fracture started in 1986 (Paper I). The upper western part has moved more than the rest of the unstable area, and it moved at a higher rate from 1961 to

1983, than it has done later (Paper IV). This might suggest that upper western part now is in the secondary (or steady) phase of the three phases of idealized creep behaviour for continuum materials and cohesive soils as observed in laboratory tests: primary (strain hardening), secondary (steady) and tertiary (accelerating/deceleration).

The displacement rates initially measured in the upper borehole between depths 32m and 82m are much smaller than the displacement rates measured at the slope surface in the upper western half of the slope, showing that movements take place above 32m and/or below 82m. Movements at depths 87m, 97m, 107m and 120m have later been identified after moving the measuring column to depth 83 - 133m.

The major part of the core loss / crushed core is observed in the upper 50 - 60m of the boreholes, and the fracture frequency is also highest at these depths, indicating that movements take place in the upper 50 - 60m of the subsurface. Poor rock mass quality is also observed at greater depths, indicating possible deeper sliding surfaces.

The instability is controlled by the foliation which is sub-parallel with the slope, as derived from borehole logging, geophysical surveys and field mapping. The daylighting of the sliding surface is caused by large scale folding as estimated from geophysical and field mapping data.

Back-calculations of a 100,000m<sup>3</sup> rock slide that occurred on the western flank of Åknes in 1960 or 1961 have shown that the use of Barton-Bandis (BB) shear strength criterion with the BB input parameters collected inside the unstable area of Åknes is reasonable for the rock slide having occurred. Sensitivity studies in the deterministic calculations and probabilistic calculations showed that the Joint Roughness Coefficient (JRC) had the greatest effect on the computed stability of the rock slide and that JRC contributed mostly to the total uncertainty over the whole set of variables.

The conclusions from numerical modelling of cross sections are as follows: Static analyses:

- Models which are unstable to great depths (up to 200m) are in closer agreement with the friction angles derived from the Barton-Bandis parameters than models which are unstable to more moderate depths.
- Stability decreases with depth.
- The limiting friction angles (i.e. the friction angles required for stability) increase with increasing inclination of the daylighting fracture.
- A shallow slide reduces the stability at greater depths.
- A rise of the ground water table is shown to be less critical for very deep slope instability than for less deep instability.

Dynamic analyses:

- Models with the present assumed ground water conditions require friction angles considerably larger than the limiting friction angles required for static stability to withstand an earthquake with a return period of 1000 years. This indicates that an earthquake of return period 1000 years is likely to trigger sliding to great depth in the slope.
- Models with the present assumed ground water conditions require friction angles slightly larger than the limiting friction angles required for static stability to withstand an earthquake with a return period of 100 years. This indicates that an earthquake of return period 100 years is not likely to trigger sliding to great depth in the slope.

- For withstanding the same dynamic impact, the model in which the depth of instability was restricted to a maximum of 110m required larger friction angles compared to the friction angles required for static stability than the corresponding model with a maximum depth of instability equal to 200m. This is interpreted as a shallow instability has less capacity to withstand an earthquake than a deep instability.
- If the lower daylighting fracture is inclined less than the assumed maximum dip of 20°, this will have a positive effect on the slope's capacity to withstand earthquakes.
- Computations of dry models and models with the ground water pressure reduced by 50 % from the assumed present conditions show that the Åknes rock slope will have very good chance of withstanding an earthquake with return period of 1000 years if draining is implemented.

# Suggestions for future work

#### Investigation and early warning system

#### Displacements

The very limited monitoring of displacements in the lower western part of the Åknes rock slope, i.e. radar measurements from across the fjord, is believed to be a weak part of the monitoring system. 3D measurements of several points in the south-western part should definitively be included in the early warning system.

A micro-seismic network is today operating in the upper area of the slope. It has been proven to successfully capture the noise triggered by the slips in the upper part of the slope. A similar network may be necessary to supplement the direct measurements of movements in the southwestern part.

The measuring column in the upper borehole has been moved once, now covering about a 100m long interval. The measuring column in the upper borehole should be moved to even greater depth as soon as the results from the present measuring interval (83 - 133m) are found to be reliable. In general, the measuring intervals in all three boreholes should be changed until all possible depths of movements have been identified (with the exception of various lengths of the upper sections of the boreholes which are lined by steel casing, making them unsuited for measurements).

#### **Ground water**

Despite of detailed investigations being performed since 2004, data on water pressure at depth do not exist. Boreholes at three locations are available to conduct such measurements. It is recommended that piezometers should be installed in isolated sections at various depths in the boreholes at all three locations. The location of the piezometers should be planned based on results from the quite extensive borehole investigations which have been performed already. Although results from water pressure measurements are believed to be important for obtaining a better understanding of the unstable Åknes rock slope, such results would still only give a rough indication of the ground water conditions. The area is large and local variations in ground water conditions must be expected.

Obtaining a better understanding of the ground water conditions is very important with respect to possible draining of the slope. A first evaluation of draining, including a cost estimate, has been carried out (Moen, 2007). Based on this evaluation draining seems feasible.

It is recommended that detailed planning of drainage should start right ahead, and, if supplementary investigations on ground water conditions indicate that drainage will stabilize the slope, little time will be lost with respect to implementing drainage.

#### Numerical modelling

Once more detailed data on displacements and water pressure in the three boreholes become available, supplementary 2D numerical modelling with somewhat fewer uncertainties compared to the modelling performed in this study can be carried out. Hypotheses and models that arise from an extended data basis may be tested through numerical modelling.

3D numerical modelling of the Åknes rock slope is under preparation. Although major uncertainties in the 3D geometry exist, this modelling is believed to have the potential to increase the understanding of long term deformation and creep behaviour of the slope.

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# Paper I: Evaluation of movement data and ground conditions for the Åknes rock slide

Kveldsvik, V., Eiken, T., Ganerød, G. V., Grøneng, G., Ragvin, N. (2006). In: Proceedings of International Symposium on Stability of Rock Slopes in Open Pit Mine and Civil Engineering, p. 279 – 299. The South African Institute of Mining and Metallurgy (SAIMM).
# EVALUATION OF MOVEMENT DATA AND GROUND CONDITIONS FOR THE ÅKNES ROCK SLIDE

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### ABSTRACT

Catastrophic rock slope failures have caused destructive tsunamis in Norwegian fjords. At the Åknes rock slope the tsunami generating potential is large due to the potential large volume involved in a possible catastrophic failure. Widening of the upper crack has been recorded since 1986, and in recent years, a quite extensive investigation and monitorin campaign has been conducted. Data from some of these investigations are presented and analysed with respect to a preliminary evaluation of the stability of the slope.

# **1 INTRODUCTION**

Large rock slides represent one of the most serious natural hazards in Norway, as exemplified by the Tafjord disaster of 1934 when 2 - 3 million m<sup>3</sup> rock mass and scree material dropped into the fjord (Jørstad, F. 1968). The tsunami generated by the slide reached a maximum of 62 m above sea level, and several villages were destroyed. 41 people were killed by the tsunami. In the 20<sup>th</sup> century 175 people lost their lives in three such events in the region of northern West Norway (Tafjord 1934 and Loen 1905 and 1936, Figure 2).



Figure 1. Index map showing the location of the area in Figure 2.



Figure 2. Locations of historical rock slides in the county of Møre og Romsdal. Skafjellet (year 1731), Tjellefjellet (1756), Loen (1905 and 1936) and Tafjord (1934) generated destructive tsunamis killing 224 people. At Åknes, a tsunami generating slide is feared.

Data available on the historical rock slides and rock slides in general since deglaciation of Norway used to be sparse. In the 90's a systematic study on rock slides and their hazard started by the Geological Survey of Norway, and later also by the International Centre for Geohazards. These studies and investigations in fjords and onshore have shown numerous rock slide deposits, in addition to a series of large-scale unstable mountains slopes along valleys and fjords. (Blikra, L. H. et. al. 2004, 2005a and 2005b, Braathen, A. et. al. 2004 ). Some of these unstable rock slopes present a threat to people, buildings and infrastructure.

The most detailed investigations have been conducted at the Åknes rock slope (Figure 2). The first investigations started in the late 80's (NGI 1987 and 1989) after local authorities had been informed that a well known crack in the rock slope was widening. The first reports were followed up by installation of some bolts for monitoring movements over the crack (NGI 1987 and 1996). The Åknes/Tafjord project was initiated in 2004 aiming at investigations, monitoring and early warning of the unstable slope at Åknes, and also of some slopes along Tafjorden. The responsible for the project is the municipalities of Stranda and Norddal, with the Geological Survey of Norway as the geo-scientific coordinator. The investigations have till now been focusing on detailed lidar survey, geological field investigations, geophysical surveys, core drilling, and measurements of movements. The investigations also include initial studies of the tsunami generating potential of a 35 mill. m<sup>3</sup> rock slide at Åknes. In this study is the maximum water surface elevation estimated to 90m, and maximum run-up heights are estimated to more than 100m across the fjord. Maximum run-up heights are roughly estimated to 25 - 35m in Hellesylt (Figure 2) and 2 - 40m in the other settlements along the fjord. The tsunami will strike Hellesylt five minutes after its generation (NGI 2005). It is preliminary estimated that 600 to 1200 people may stay in the tsunami hazard zones as an avearage over a year (Åknes/Tafjord project, unpublished). During the tourist season the number of people at risk can be several thousands.

The Åknes landslide area indicated in Figure 3 is estimated to approximately  $800,000m^2$ . The slope is dipping towards SSE with dip angle of  $35 - 40^\circ$ . Just below sea level the slope flattens to about 20°. Single open cracks and areas with several open cracks, indicating movements, are found many places in the slope. Three historical rock slides are known in the Åknes rock slope, all of them from the western flank. The approximate dating of these slides is as follows: 1850 - 1900, 1940 and 1960.

Figure 4 shows details of the western part of the upper crack. The upper western flank is separated from the back wall by about 20 - 30m. To the east of the upper western flank the minimum horizontal crack width of the upper crack is typically around 1m.

The present paper aims primarily on describing data on displacements and ground conditions as a basis for future stability analyses of the Åknes rock slope.

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Figure 3. Overview of the Åknes rock slope. The white dotted line indicates the contour of the landslide (slightly modified after Derron, M.-H., et.al. 2005). The length of the "top scarp"/upper crack is about 700m measured from west to east. Numbers show approximate dates of known rock slides.

Figure 4. Details of the upper crack. The minimum horizontal crack width at the right hand side of the photo and further towards east (to the right) is typically around 1m

# 2 **DISPLACEMENTS**

# 2.1 Across the upper crack

The first three extensometers for automatic reading were installed in 1993. Each extensometer is fixed in solid rock at both sides of the upper crack and it measures the distance between these fixed points. Measurement takes place once a day through 40 readings in a short period of time of which the mean value is stored in a computer as the recorded value. The extensometer monitoring programme was extended with two more extensometers in 2004. The locations of the extensometers are shown in Figure 5. The monitoring results are shown in Figure 6 and 7. Figure 8 shows the results of manual measurements which started in 1986. The manual measurements were carried out by measuring manually the distance between fixed bolts on each side of the upper crack with a measuring rod.



Figure 5. Ortophoto of the Åknes Landslide area. The extensometers along the upper crack are marked with circles and numbers (Nos.1 – 5). A detail of Ext 5 is shown. Locations of core borings are marked with triangles. (U = upper, one boring, M = middle, two borings and L = lower, one boring).

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Figure 6. Displacements at the upper crack. Extensometer readings from 1993-08-28 to 2005-11-25. Ext 3b is a replacement of Ext 3a which was destroyed. Ext 3b is aligned more parallel to the slope movement than Ext 3a, which means that Ext 3b picks up a larger portion of the movement.



Figure 7. Displacements at the upper crack. Extensometer readings from 2004-11-25 to 2005-11-25.

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Figure 8. Displacements at the upper crack. Manual measurements from 1986-08-26 to 2004-06-22.

With reference to Figure 5 Mp 1 is placed 10m east of Ext 1, Mp 3 is placed a few metres east of Ext 1, and Mp 2, Mp 4a and Mp 4b are placed a few metres west of Ext 4.

Figure 9 sums up the displacement in terms of mean displacement per year for all the measuring locations at the upper crack. The recorded values have been adjusted to an assumed direction of slope movement. This adjustment should not be regarded as a "correct" adjustment, but is believed to give a better comparison of the results than the measured values since some of the measuring directions are quite oblique to the assumed direction of movement, meaning that they only pick up a fragment of the real displacement. The mean values of the adjusted values for all measuring locations are 21.7mm/year, in the westernmost part 25mm/year and in the eastern part 17.8mm/year. Ext 5 in particular draws down the mean value of the eastern area. Ext 5 is located about 30m west of the point where upper crack dies out as a clearly visible open crack. From Figures 6 - 8 it is clear that the displacements at the upper crack go on with an overall steady pace. Some periods with faster movements and some periods with slower movements can be identified, but there is no general tendency of acceleration or deceleration. It may be noted that some places, near the upper crack, narrow cracks subparallel to the upper crack, exist. Up to now, these cracks have not been monitored, but their existence shows that the total displacement in the upper part of the slide area is somewhat larger than shown in Figures 6 - 9.

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Figure 9. Displacements at the upper crack in terms of mean values per year. The assumed direction of movement is N157°/35° (dip direction/dip angle), which is sub-parallel to the slope itself. Ext 3a was in operation up to 2003-01-22. Ext 3b was put into operation 2004-11-25 at almost the same location as 3a, but aligned more parallel to the assumed direction of movement.

### 2.2 At the slope surface

### 2.2.1 Measuring methods

Several methods are used for measuring movements in the slope: GPS, total station, radar and photogrammetry. This paper presents some main results of the photogrammetry. The photogrammetry covers a period of 43 years.

# 2.2.2 The photogrammetric method

Photogrammetric studies have been conducted for the periods 1961 – 1983 and 1983 – 2004. Aerial photographs of the scale 1:15000 of 1961 and 1983 were used to make elevation models (Digital Terrain Model) and orthophotos of pixel size 20cm by use of software from ZI-Imaging. For 2004 an orthophoto produced by FUGRO was used. On the orthophotos points that appeared identical have been located, i.e. mainly rock blocks. The coordinates of the apparently identical points have been used to calculate possible displacement vectors. For the 1961 and 1983 aerial photographs the coordinate system was identical. The 2004 orthophoto refers to a different coordinate system, which made it necessary to establish a transformation between apparently identical points in the 1983 and 2004 orhophotos. This transformation routine was established by use of points in the border area of the area covered by the orthophotos. At these points the displacements were presumed equal to or close to zero.

Since points at the slope surface are compared, the photogrammetric method does not distinguish between movements that take place just below the slope surface, e.g. solifluction, and surface movements that are a caused by movements at deeper levels in the slope. The accuracy of the method is estimated to be 0.5m.

# 2.2.3 Results

93 points were analysed in the period 1961 - 1983, of which 62 points showed displacement larger than 0.5m. 122 points were analysed in the period 1983 - 2004, of which 73 points showed displacement larger than 0.5m.

Table 1 sums up the results for the two periods based on points that showed displacement larger than 0.5m.

Table 1. Displacements derived from photogrammetric studies. Values are given as cm/year, that is the total displacement over the whole period divided by the number of years.

Period		Mean	Median	Maximum	Variation coefficient
					(%)
1961 - 1983	Magnitude	6.4 cm/year	4.6 cm/year	17.0 cm/year	63
1983 - 2004	Magnitude	5.9 cm/year	5.8 cm/year	13.6 cm/year	46
1961 – 1983	Dip direction	N202°	N189°		33
1983 - 2004	Dip direction	N192°	N188°		45

Table 1 indicates that the displacement rates on average have been quite stable from 1961 to 2004. It should be noted that displacement measured over the upper crack (Figure 9) are smaller or near the accuracy of the photogrammetric method. In other words; the numbers in Table 1 are almost entirely derived from points that have moved more than upper crack has widened. The results of the photogrammetric studies reflect mostly (but not only) displacements that have taken place in the upper western part of the landslide area, with the largest displacements taking place in the western flank. The other measuring methods (GPS, total station and radar) all demonstrate that the largest movements take place in the western flank, and that the movements are in the order of 10cm per year.

# 2.3 Evaluation of stability based on displacement rates

Catastrophic failure of creeping slopes is associated with an acceleration phase before the catastrophic failure (e.g. Petley, D. N. et. al. 2002, Kilburn, C. R. J. and Petley, D. N. 2003, Crosta, G. B. and Agliardi, F. 2003). According to idealized creep behaviour the tertiary, accelerating, creep phase, is preceded by a primary, or strain hardening phase, and a secondary steady phase. Use of this idealized creep behaviour on the Åknes rock slope indicates that the rock slope in general must be in the steady (secondary) phase, or perhaps in the primary phase. This implies that there is still some time before a possible catastrophic collapse of the slope. If, or when, the slope will start accelerating is the big question. An extended and improved monitoring programme will be implemented at Åknes for the purpose of an early warning system, and treshold values for different parts of the slope have to be established.

# **3 GROUND CONDITONS**

# 3.1 Rock types

The general picture from mapping the rock outcrops in the area is that three gneiss variants exist, namely granitic gneiss (pink with dark minerals), dioritic gneiss (light

### The South African Institute of Mining and Metallurgy International Symposium on Stability of Rock Slopes Vidar Kveldsvik, Trond Eiken, Guri V Ganerød, Guro Grøneng and Nicole Ragvin

grey with dark minerals) and biotitic gneiss (dark). The granitic gneiss may appear both as massive rock and as quite dense jointed along the foliation. The dioritic gneiss appears as massive, and the biotitic gneiss appears as weak layers with dense jointing along the foliation. A quite typical picture of the rock outcrops is illustrated in Figure 10 which shows general massive rock, and layers with dense jointing along the foliation. The foliation joint spacing may be less than 10cm in some places. The granitic gneiss is dominating in three of the four diamond drilled boreholes; the exception is Borehole L1 (Figure 5) where the dioritic gneiss makes up the largest portion of the rock core. A summary of the rock type distribution is given in Table 2. It should be noted that the rock type classification in Table 2 has been simplified with respect to the original core log where the rock type is described by meter: One single meter rock core may in some cases include three sections of rock, for instance two sections with granitic gneiss separated by a section of biotitic gneiss. In such cases the rock type has been classified as granitic gneiss.



Figure 10. Rock outcrops. Left: dense jointing along the foliation. Right: dense jointing along the foliation with more massive rock above.

Borehole	Length	Inclination	Granitic	Dioritic	Biotitic	BG/GG
No.	(m)		gneiss	gneiss	gneiss	and
			(GG)	(DG)	(BG)	BG/DG <sup>3)</sup>
			(%)	(%)	(%)	
U1	162	Vertical	49.4	22.8	19.8	6.8
M1 <sup>1)</sup>	149	60° <sup>2)</sup>	57.7	2.7	29.5	8.7
M2 <sup>1)</sup>	151	Vertical	43.0	13.9	27.8	13.2
L1	150	Vertical	16.2	42.6	33.8	5.4

Table 2. Dis	tribution o	of rock type	es in the	boreholes

<sup>1)</sup> M1 and M2 are spaced apart only a few metres.

<sup>2)</sup> Drilled nearly perpendicular to the slope.

3) Biotitic gneiss in combination with granitic gneiss or dioritic gneiss.

In addition to the rock types listed in Table 2, there are about 1 - 2 % dioritic gneiss in combination with granitic gneiss in the four boreholes. The biotite content in the three rock types has been estimated by visual judgement during logging. The mean biotite content based on all estimates are as follows: 35 % for granitic gneiss, 41 % for dioritic gneiss and 62 % for biotitic gneiss. Probably, this variation of the weak mineral biotite, can explain some of the variation in uniaxial compressive strength (UCS) for the three rock types which have the following mean values: 162MPa for granitic gneiss (23 tests), 134MPa for dioritic gneiss (9 tests) and 113MPa for biotitic gneiss (16 tests).

# 3.2 Discontinuities

### 3.2.1 Data collection

Mapping of discontinuities has been carried out by the following methods: measurement of the orientation and spacing of discontinuities in the rock outcrops, core logging and by structural analysis of the Digital Elevation Model (DEM). The structural analysis by DEM is performed by Derron, M.-H., et. al. (2005) for the upper and middle part of the assumed landslide area (the lower part is covered with vegetation), with most data from the upper part.

The field mapping has focused on the typical conditions at each location, which particularly for the foliation and foliation parallel joints means that some data are left out; namely the quite large variation that exist at some locations due to small scale folding.

The diamond drilled boreholes have mainly intersected the foliation. Joints that are not parallel to the foliation are generally quite steep as registered by the field mapping, and these joints have been intersected only to a small extent, due to steep inclination of the boreholes. Since the attempt to orientate the cores during drilling was unsuccessful, orientation data could only be measured in the three vertical boreholes in form of dip angles of the foliation. The foliation dip angles were measured metre by metre in the three boreholes by the following procedure: If it was concluded by visual inspection that the foliation was overall consistent over the metre, one measurement was done. If variations were detected by the visual inspection two measurements were done trying to capture the minimum and maximum values.

# 3.2.2 Orientation

All orientation data are given as dip direction and dip angle in degrees unless otherwise is assigned.

Derron, M.-H., et.al. (2005) identified three joint sets in the upper part of the landslide area by structural analysis of the DEM (Figure 11). The mean orientations of the joint sets are: J1-N180/45 (foliation parallel joints), J2-260/70 and J3-050/50. Derron, M.-H., et. al. (2005) compared this to field mapping of joints along and near the upper crack and found a fairly good agreement (Figure 12).





Figure 11. A) Orthophoto of the upper part of the Åknes landslide. The white line is the open upper crack. B) Detection of the cells of the DEM which have orientations that correspond to the joint sets J1 (foliation joints, white), J2 (black) and J3 (grey) respectively. From Derron, M.-H., et.al. (2005).

Figure 12. Measurements of the joint orientations of the upper part of the Åknes landslide. Left: DEM analysis. Right: Field measurements (courtesy of Braathen, A.). Lower hemisphere stereographic projections. From Derron, M.-H., et.al. (2005).

246 orientations have been measured by field mapping in the whole landslide area, distributed as 142 foliation parallel joints and 104 joints that are not parallel to the foliation (Figure 13). Figure 13 compared with Figure 12 shows that the joint orientations are much more scattered when measurements from the whole landslide area are included. Figure 13 shows also that the joints that are not parallel with the foliation are generally sub-vertical.



Figure 13. Structural measurements in the landslide area. Top: Pole plot of all the joints. Bottom: Contour plot.

Only foliation parallel joints have been plotted in Figure 14. It is clear from the plot that the global mean vector of the foliation joints (N155°/23°) is nearly parallel with the slope orientation (N157°/34°).



Figure 14. Foliation joints. Top: Contour plot with the global mean vector and the slope orientation. Bottom: Rosette plot.

Foliation joints from different areas are shown in Figure 15. The figure shows that the foliation generally dips quite parallel to the slope in Zones 1 - 3 whereas the foliation dips more easterly and non-parallel to the slope in the upper part along the upper crack (Zone 4).



Figure 15. Pole plots showing foliation joints in different areas of the slope. The mean great circle calculated from the pole plot is shown as the dotted line. The slope is shown as the solid line.

Table 3 shows the dip angles of the foliation from the field mapping and the core logging. For the calculations reported in Table 3, the borehole data have been treated as follows: Where minimum and maximum values have been measured over 1m core (see Section 3.2.1), three values have been recorded: the minimum value, the maximum value and the mean of the two values. Where only one measurement has been taken, three values have also been recorded such that the each metre of the core has been represented consistently by three values: the measured value and the measured value  $\pm 2^{\circ}$ .

Dataset	Mean	Median	Variation	Minimum	Maximum
	(°)	(°)	coefficient	(°)	(°)
			(%)		
Borehole U1	27.5	28	28.8	4	53
Borehole M2	33.5	33	27.2	8	57
Borehole L1	34.2	34	32.5	3	70
U1+M2+L1	31.4	31	31.3	3	70
Field mapping, all data	39.9	37	38.8	10	89
(142 measurements)					
Field mapping, selected	35.6	35	32.6	10	70
data (114					
measurements) <sup>1)</sup>					

Table 3. Dip angle of the foliation

<sup>1)</sup>One series of the field mapping consist only of data collected along the upper crack between 30m west of Ext 1 and some tens of metres east of Ext 3, and these data are not included in the "selected data". The reason is that the foliation dips steeper in this area than generally elsewhere in the slope, such that the "selected data" appear more comparable with the borehole data.

Table 3 shows that there is a considerable difference between minimum and maximum dip angles of the foliation for all five data sets. The mean values of the five data sets are quite similar, and they suggest that the dip angle of the foliation is generally of the same magnitude as the dip angle of the slope  $(35 - 40^\circ)$ .

# 3.2.3 Jointing in boreholes

The core logging included counting of number of natural joints, length of core loss and length of crushed core, as described below.

Fractures/breakage caused obviously by the drilling itself was not included in the joint count. For the foliation parallel fractures/breakage in the rock cores, it was to some extent difficult to distinguish between a natural joint and a fracture/breakage that was caused by the drilling. Some foliation parallel joints included in the joint count may therefore represent weaknesses along the foliation broken apart by the drilling, rather than natural joints. It should be noted that the majority of joints intersected by the boreholes (and counted during the core logging) are parallel to the foliation. This is certainly due to the drilling direction which favours intersection of the foliation rather than the more vertically inclined joints, but probably also for the reason that the frequency of the foliation parallel joints is higher than for other joints.

Core loss is sections of the borehole where the length of collected material is less than the drilling length. Core loss is assumed to represent weak material (e.g. fine grained material) or even voids (e.g. intersection of an open crack) in the rock mass. The term "crushed core" is used for sections where the collected material appears as rock fragments and/or fines. Crushed core is assumed to represent poor rock mass quality (dense jointing and/or low strength of the intact rock).

The jointing and core loss / crushed core in the boreholes are summarized in Figure 16.



Figure 16. Cumulative number of joints and cumulative length of core loss / crushed core for Boreholes U1 (vertical), L1 (vertical), M2 (vertical) and M1 (60°).

Figure 16 shows that the major part of the core loss / crushed core has occurred from 0m to 50 - 60m depth, and that the joint frequency decreases at about 50m in all four boreholes. It is reasonable to assume that some, perhaps the major part, of the poor rock mass quality leading to core loss / core crushing during drilling is associated with ongoing movements in the slope. It follows from this assumption, that ongoing movements in the slope may be restricted to depths of about 60m.

During drilling loss of water was such a problem in the upper part that all the holes except for Borehole L1 had to be lined. Steel casing was used down to 40m in Borehole U1, 20m in M1 and 30m in M2. L1 was only lined through a few metres of soil. These experience show that the rock mass at these shallow depths is very permeable, which may be interpreted as a broken and disturbed rock mass.

Figure 17 shows the rock type distribution, joint frequency and core loss / crushed core for the full lengths of the four boreholes. Figure 18 shows the same information, but restricted to the upper 60m of all four boreholes. Figure 17 does not reveal any specific trends with respect to core loss / crushed core and joint frequency versus rock type. Figure 18, however, shows that the biotitic gneiss and biotitic gneiss in combination with granitic or dioritic gneiss has more core loss / crushed core than the other rock types, and also; it is more jointed. Figure 19 shows that the dioritic gneiss has most core loss / crushed core when one looks at the total without considering the distribution in the various boreholes. However, this is caused only by the large portion of core loss / crushed core of dioritic gneiss in Borehole U1.



Figure 17. Rock type distribution, joint frequency and core loss / crushed core for the full lengths of Boreholes U1, M1, M2 and L1. Core loss / crushed core is given as percentage of the total length of the rock type in the borehole.



Figure 18. Rock type distribution, joint frequency and core loss / crushed core for the upper 60m of Boreholes U1, M1, M2 and L1. Core loss / crushed core is given as percentage of the total length of the rock type in the upper 60m of the borehole.



### Borehole U1, M1, M2 and L1 combined: 0 - 60m

Granittic gneiss (GG) Diorittic gneiss (DG) Biotittic gneiss (BG) BG/GG and BG/DG Figure 19. Rock type distribution, joint frequency and core loss / crushed core for the upper 60m of Boreholes U1, M1, M2 and L1 combined.

# 3.3 Ground water

The depth to the ground water in the four boreholes has been measured a few times during the autumn 2005 (Figure 20). In this period the depth is around 50 - 60m in Borehole U1, and around 40 - 45m in Boreholes M2 and L1. Continuous monitoring of the ground water level in Borehole M2 started in December 2005, and in the period December 2005 – February 2006 the depth to the ground water has fluctuated between 38m and 40m. The period of measurement is too short to draw firm conclusions about the ground water, but pretty large depths to the ground water in the slope are certainly indicated.





# **3.4** Evaluation of ground conditions with respect to stability

The instability of the Åknes rock slope appears to be caused mainly by unfavourable orientation of the foliation in relation to the orientation of the slope. The presence of gneiss rich in biotite may play an important role due the relative weakness of this rock type. The instability may be restricted to depths of about 60m below the ground. It is suggested that shear movements along the foliation take place at several levels in the rock mass from depths of about 60m and upwards. The possible maximum depth of about 60m may be governed both by the presence of ground water at these depths as well as the slope inclination of  $35 - 40^\circ$ . A possible lower failure plane is indicated in Figure 21.



Figure 21. Profile trough the central part of the landslide area. Possible lowest level of shear movements are indicated (modified after geophysical survey conducted by the Geological Survey of Norway).

# 4 CONCLUSIONS

The Åknes rock slope is located in a fjord system where several rock slides have occurred since deglaciation. Three slide events from the western flank are known to have occurred in historical times, the latest event occurred around 1960. These facts combined with the documented ongoing movements define the Åknes rock slope as a hazardous object. Because of the possible large volume involved in a possible catastrophic failure, the tsunami generating potential is large, meaning that people and infrastructure are at risk. The rather steady displacements rates that have been recorded over the years, indicate that the slope is in a secondary, or steady, creep phase, which means that one would expect an accelerating phase prior to a possible catastrophic failure.

The instability of the Åknes rock slope appears to be caused mainly by unfavourable orientation of the foliation compared to the orientation of the slope. The presence of gneiss rich in biotite may play an important role due the relative weakness of this rock type. The instability may be restricted to maximum depths of about 60m below the ground. Displacements along the foliation may take place at several levels above about 60m. Monitoring of displacements in boreholes is needed to verify or reject this hypothesis.

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# Paper II: Alternative approaches for analyses of a 100,000 m<sup>3</sup> rock slide based on Barton-Bandis shear strength criterion

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# Paper III: Geological model of the Åknes rock slide, western Norway

Ganerød, G. V., Grøneng, G., Rønning, J. S., Dalsegg, E., Elvebakk, H., Tønnesen, J. F., Kveldsvik, V., Eiken, T., Blikra, L. H. and Braathen, A. (2008). Engineering Geology, DOI 10.1016/j.enggeo.2008.01.018. In print.

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# Geological model of the Åknes rockslide, western Norway

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#### ABSTRACT

Åknes is known as the most hazardous rockslide area in Norway at present, and is among the most investigated rockslides in the world, representing an exceptional natural laboratory. This study focuses on structural geology and the usage of geophysical methods to interpret and understand the structural geometry of the rockslide area. The interpretations are further used to build a geological model of the site. This is a large rockslide with an estimated volume of 35-40 million m<sup>3</sup> [Derron, M.H., Blikra, L.H., Jaboyedoff, M. (2005). High resolution digital elevation model analysis for landslide hazard assessment (Åkerneset, Norway). In Senneset, K., Flaate, K. & Larsen, J.O. (eds.): Landslide and avalanches ICFL 2005 Norway, Taylor & Francis Group, London.], defined by a back scarp, a basal shear zone at about 50 m depth and an interpreted toe zone where the sliding surface daylights the surface. The rockslide is divided into four sub-domains, experiencing extension in the upper part and compression in the lower part. Structural mapping of the area indicates that the foliation of the gneiss plays an important role in the development of this rockslide. The upper boundary zone of the rockslide is seen as a back scarp that is controlled by, and parallel to, the pre-existing, steep foliation planes. Where the foliation is not favourably orientated in regard to the extensional trend, the back scarp follows a pre-existing fracture set or forms a relay structure. The foliation in the lower part, dipping 30° to 35° to S–SSE, seems to control the development of the basal sliding surface with its subordinate low angle trusts surfaces, which daylights at different levels. The sliding surfaces are sub-parallel to the topographic slope and are located along mica-rich layers in the foliation.

Geophysical surveys using Ground Penetrating Radar (GPR), refraction seismic and 2D resistivity profiling, give a coherent understanding of undulating sliding surfaces in the subsurface. The geophysical surveys map the subsurface in great detail to a depth ranging from 30–40 m for GPR to approximately 125 m for refraction seismic and 2D resistivity profiling. This gives a good control on the depth and lateral extent of the basal sliding surface, and its subordinate low angle thrusts. Drill cores and borehole logging add important information with regard to geological understanding of the subsurface. Fracture frequency, fault rock occurrences, geophysical properties and groundwater conditions both in outcrops and/or drill cores constrain the geometrical and kinematic model of Åknes rockslide.

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#### 1. Introduction

Unstable rock slopes pose a threat to the inhabitants along Norwegian fjords, where prehistoric and historic rock avalanches have created tsunamis, some causing severe casualties (Blikra et al., 2005a). The site presented, Åknes, is located in western Norway (Fig. 1). This is a large rockslide with an estimated volume of 35–40 million m<sup>3</sup> (Derron et al., 2005), defined by a back scarp, a basal shear zone at 50 m depth 60 and a toe zone where the basal sliding surface daylights the surface. 61 Continuous creep of the rock mass and the fact that Åknes is situated 62 above the fjord and in the vicinity of several communities as well as one 63 of Norway's most visited tourist attractions (the Geirangerfjord, listed on 64 the UNESCO's World heritage list), have triggered a comprehensive 65 investigation program. The overall aim of the project is firstly, to assess 66 the likelihood that the rockslide will accelerate into a rock avalanche and 67 secondly, to establish an early warning system with direct monitoring of 68 deformation (translation and rotation), so that the local communities are 69 able to evacuate in time. 70

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**Fig. 1.** A) Location of the rockslide site of Åknes in western Norway. This site is found 150–900 m above sea level in a SSE facing steep mountain slope. The main concern for this area is that a rock avalanche will reach the fjord at the foot of the slope, and trigger a tsunami in the fjord system. B) Map that locates domains and sub-domains (1, 2, 3 and 4) and key structures described in the text. C) Schematic profile (located in b) that outline domains, sub-domains and key structures.

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Recent regional studies of the area are summarized in Braathen 71 72 et al. (2004), Blikra et al. (2005a,b), Hermanns et al. (2006), Henderson 73 et al. (2006) and Roth et al. (2006). Henderson et al. (2006) suggest that 74 existing structures in the bedrock, such as foliation, faults and fracture zones, are controlling the development of the rockslides that occur in 75the fjord system. Where the foliation and slope angle coincide and 76 structural weaknesses are favourably oriented, the rockslide hazard is 77 considered greater (Henderson et al. 2006). Historical data from Åknes 78 79reveals three moderately sized rockslide occurrences within a rather 80 short time interval: in the years 1850-1900, 1940 and 1962 (Kveldsvik 81 et al., 2006). Other recent studies from the Åknes area are presented by 82 Derron et al. (2005), who give an estimate of size of the rockslide, and Kveldsvik et al. (2006) who present a brief summary of the in-83 84 vestigations and the progress of the project, and analyses of the 100,000 m<sup>3</sup> rockslide that occurred in 1962 (Kveldsvik et al., 2007). 85

Despite the number of regional and local studies, a detailed struc-86 tural understanding of the rockslide area is missing. A key issue has 87 been to locate the basal sliding surface of the rockslide, since this is 88 a prerequisite for precise volume estimation. Location of the basal 89 sliding surface will also lead to a better understanding of the sliding 90 mechanism(s) of the unstable area. The ongoing survey of the area is 91 comprehensive and includes borehole logging and monitoring, which 9293 will help constrain the location of the basal sliding surface or sliding zone more precisely and yield additional guantitative data regarding 94 spatial and temporal sliding velocities. The aim of this study has been 95to describe the rockslide area at Åknes by means of detailed structural 96 surface mapping, supported by subsurface data from 2D resistivity, 9798 Ground Penetrating Radar (GPR) and refraction seismic profiling, core drilling and geophysical logging of boreholes. Together, these data give 99 a detailed 3D geological understanding of the area, in which the depth 100 to - and the geometry of the basal slide surfaces can be identified and 101 102described. A secondary goal of this study has been to propose a 103 geological model of the Åknes rockslide for further numerical mo-104delling, including groundwater and slide dynamics.

Results from the study show that there are structural limits to the 105rockslide area, consisting of the extensional back scarp zone at the top, 106 a steeply dipping, NNW-SSE trending strike slip fault as the western 107 boundary zone, a gentle dipping NNE-SSW trending pre-existing fault 108 as the eastern boundary zone and a compressional toe zone at the 109bottom. The rockslide area is divided into four sub-domains (1 to 4), 110 two mapped on the surface (2 and 4) and two mapped in the sub-111 surface by geophysics (1 and 3, Fig. 1). The geophysical surveys indicate 112 that the sub-domains are bound by the basal sliding surface with its 113 four subordinate, low angle thrusts that stack the bedrock lobes upon 114 one another, forming an imbricated thrust fan. The overall geometry is 115 116 that of extension in the upper part and compression in the lower part of 117 the slope. An outline of the rockslide is given in Fig. 1B and C.

#### 118 **2. Geological setting of Åknes Site**

The Åknes site is a southward facing slope, with an average dip 119 120angle of 30-35°, with a topography that stretches from sea level to an 121 elevation of 1300 m over a distance of 1500 m (Fig. 2). We propose subdividing the rockslide area into five zones, based upon different 122structural signatures. The unstable area is estimated to be 800 m 123across-slope and 1000 m down-slope, with an upper boundary, the 124125Back Scarp Zone, located 800-900 m above sea level, and a lower boundary, the Toe Zone, at 150 m above sea level. The western margin 126is a steep NNE-SSE trending strike slip fault, called the Western 127 Boundary Zone, forming a narrow, deep crevasse in the mountainside 128 (Fig. 2). On the east side, the rockslide area is bound by a pre-existing 129fault dipping gently (35-45°) to the west, called the Eastern Boundary 130Zone. The fifth part of the rockslide is named the Central Zone. 131

The Åknes rockslide is located in the Western Gneiss Region. The
 bedrock of the area is dominated by gneisses of Proterozoic age, which
 was altered and reworked during the Caledonian orogeny (Tveten

et al., 1998). The gneisses have a magmatic origin and are described in 135 the geological map sheet as undifferentiated gneisses that are locally 136 migmatitic in composition, varying from quartz-dioritic to granitic 137 (Tveten et al., 1998). Within certain areas the gneiss has a distinct 138 metamorphic penetrative foliation (S<sub>1</sub>, dominantly 080/30) that is 139 folded around gently ESE-plunging axes (Tveten et al., 1998; Braathen 140 et al., 2004). The bedrock at the study sites alters from a white to light 141 pink, medium grained granitic gneiss to a dark grey biotite bearing 142 granodioritic gneiss, and further to a subordinate white to light grey, 143 hornblende to biotite bearing, medium grained dioritic gneiss. There 144 are also laminae, and up to 20 cm thick layers, of biotite schist within 145 the gneiss. All lithologies occur in layers parallel to the metamorphic 146 foliation. 147

3.	Methods	
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#### 3.1. Structural mapping

Structural mapping is conducted on outcrops in the field, where 150 fracture properties such as orientation, strike and dip with right hand 151 rule (RHR) measurement (Davis and Reynolds, 1996), length/persis- 152 tence and frequency is collected. The fracture frequency is measured 153 along a ruler in *x* direction parallel to foliation, *y* direction perpen- 154 dicular to foliation, and if possible in a third direction (*z*) to estimate 155 the block size. The foliation in gneissic rocks commonly represents a 156 weakness in the bedrock and will therefore fracture along it, creating a 157 higher fracture frequency in that direction as demonstrated in Fig. 3. 158 This will give a dominance of fractures parallel to the foliation while 159 other fracture orientations most likely are underrepresented in com- 160 parison. The data is later analysed per locality, for example as lower 161 hemisphere stereo plots (Wulff net) of fracture orientation.

A total of seven drill holes were drilled and cored at three sites in 164 the rockslide area. Three holes are vertical to 150 m depth; one is 165 inclined by 60° and goes down 150 m, while the remaining three are 166 vertical and 200 m deep. All cores have been logged (Ganerød et al., 167 2007) for fractures (discontinuities) per metre giving a fracture fre- 168 quency per metre by depth (Nilsen and Palmstrøm, 2000). The frac- 169 tures are classified as foliation parallel or not, and the dip angle 170 between the core axis and the fracture is measured. Since the drill core 171 is not orientated, strike and dip was not measured. The drill hole 172 presented here is from the lowest drilling site and goes down to 150 m 173 depth (Fig. 2).

#### 3.3. Surface geophysical mapping

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Geophysical methods such as Ground Penetrating Radar (GPR), 176 refraction seismic and 2D resistivity profiling have been used to map 177 the subsurface. Several lines were measured in the rockslide area, of 178 which parts will be presented here. The 2D resistivity and GPR data 179 were collected along the same profiles, while seismic acquisition was 180 limited to three profiles (Rønning et al., 2006). 181

#### 3.3.1. GeoRadar

The GPR survey at the rockslide consists of seven profiles of alto- 183 gether 5300 m. One profile has NE–SW strike, four profiles are slope 184 parallel with E–W strike, and two are oriented down-slope with N–S 185 strike. The GPR profile presented has a total length of 250 m with a 186 NNW–SSE strike. 187

GeoRadar is an electro magnetic method that is used to detect 188 structures in the subsurface (Reynolds, 1997). With an antenna electro 189 magnetic pulses are sent into the subsurface, which are reflected off 190 surfaces with different dielectric properties and received by a receiver 191 antenna at the surface. The time of the propagating wave is recorded. 192

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**Fig. 3.** Example of how structural data such as fracture frequency is collected with the help of scan lines. Fracture frequency is collected in x direction parallel to foliation, in y direction perpendicular to foliation, and if possible in z direction to estimate the block size of the area. Fracture frequency collected in y direction gives commonly the highest value.

This measurement is repeated with a fixed interval, giving a conti-193nuous image of the subsurface structures. The penetration depth and 194195 the resolution of the method depend on the signal frequency and the material property, such as electrical conductivity and dielectricity. 196 197 High antenna frequency (MHz) will give good data resolution but shallow penetration depths and visa versa. The GPR profiles were 198 acquired using a frequency of 50 MHz and a shot point interval of 1 m, 199with a GPR system called Sensors & Software Pulse EKKO 100. This 200 gives reasonable resolution of the structures in the subsurface, but 201limits the penetration depths to about 30, and not more than 50 m. 202 The velocity analysis (processed CMP-gathers) performed in the area 203gave an average velocity to deep structures of 0.11 m per nano-204 seconds (m/ns), which were used for depth conversion. 205

#### 206 3.3.2. Refraction seismics

The seismic survey consists of one slope parallel profile with E–W strike and two down-slope profiles with ~N–S strike, totalling 1440 m. The seismic profile presented has a total length of 420 m with a NNW– SSE strike. For the seismic profiles the geophone spacing was 10 m, with a total of 24 14 Hz vertical geophones depending on operator. Shot point interval was 30 to 300 m with 100 to 600 g of dynamite as energizer for each shot (Rønning et al., 2006).

The seismic method is based on the recording of first arrival times 214215of P-wave travel time of waves in the subsurface. The wave propagates 216with the elasticity of the material, and the range of the seismic P-wave velocity calculated from the travel time of the wave, commonly range 217from 200 m/s up to above 6000 m/s. In fractured bedrock the seismic 218 velocity is reduced dependent on fracture frequency, texture and 219filling (Reynolds, 1997). The refraction seismic is a method that is 220developed principally for mapping of horizontal layers, and is depen-221

dent upon there being an increase in velocity with depth. If a layer has 222 lower velocity than the above laying layer, the seismic wave will not be 223 refracted in the right manner, but continue in depth and gives rise to 224 the phenomenon called a *hidden layer*. This layer is difficult to detect 225 and may be interpreted as part of the above laying layer (Reynolds, 226 1997). 227

#### 3.3.3. 2D resistivity

The 2D resistivity survey consists of eight profiles, one oriented 229 NE–SW, five slope parallel with E–W strike, and two down-slope 230 profiles with N–S strike, totalling about 10,000 m. Here, a 420 m long 231 section out of an 1800 m long down-slope profile is presented. 232

The resistivity method measures apparent resistivity (with unit 233  $\Omega$ m) in the subsurface, which is a weighted average of all resistivity 234 values within the measured volume (Dahlin, 1993; Reynolds, 1997). 235 Measured apparent resistivities with different electrode configura- 236 tions are converted into a true 2D resistivity profile through inversion 237 (Loke, 2001). The 2D resistivity profiles were acquired according to the 238 Lund-system (Dahlin, 1993). Acquisition was collected with both 239 Wenner and Dipol/Dipol configurations, with an electrode spacing of 240 10 m for the shallow and 20 m for the deeper parts of the profiles. In a 241 few short profiles the electrode interval was reduced to 5 and 10 m 242 (Rønning et al., 2006). The depth penetration of the profile is appro-243 ximately 130 m, with reliable data coverage to approximately 70 m 244 depth. Slightly resistive material of 3000 to 10,000  $\Omega$ m, shown in blue 245 colour in the profile, may indicate material such as fractured and 246 water saturated bedrock (clay filled fractures commonly show re- 247 sistivity response lower than 1000  $\Omega$ m). 10,000 to 35,000  $\Omega$ m, shown 248 in green colours in the profile, indicate moderately resistive material, 249 for example fractured and unsaturated bedrock or less fractured but 250 water saturated bedrock. Highly resistive material, 35,000 to 150,000  $\Omega$ m 251 indicated with orange to red colours in the profile, may consist of 252 "unfractured" bedrock and dry, unconsolidated material. 253

#### 3.4. Drill hole logging

All drill holes have been logged by geophysical methods such as 255 water conductivity, water temperature, natural gamma ray, resistivity of 256 the bedrock, and seismic P- and S-wave velocity. Here, only resistivity 257 and P-wave data are shown from the lowest drilling site, which goes 258 down to 150 m depth, are presented since the other logging results 259 concur these logs. All geophysical logging were performed using 260 Robertsson Geologging equipment (http://www.geologging.com). 261

#### 3.4.1. Resistivity logging in drill holes

Resistivity is the inverse of electrical conductivity, and is thus 263 easily derived from the measured conductivity value. The resistivity 264 probe consists of one electrical current electrode and two potential 265 electrodes; making it possible to measure in two configurations; long 266 and short normal. For the long normal configuration (LN) the distance 267 between the current electrode and the potential electrode is 64 in. 268 (1.60 m), while for the short normal (SN) the distance is 16 in. (0.40 m). 269 The penetration in the drill hole wall is commonly 1/5 of the electrode 270 distance, and the resolution of the data collected depends on the 271 contrast in conductivity. With such short interval in general, the probe 272 can in detail map the apparent resistivity of the surrounding bedrock, 273 often equivalent to the specific resistivity. The resistivity is affected by 274 bedrock porosity, conductivity of the pore water, the shape of the pore 275

**Fig. 2.** Relief map of the Åknes site with stereographic presentation of the fracture distribution compiled from all localities studied. The picture (lower left) outlines the inferred area involved in the rockslide (photo: M.H. Derron). The stereo plots show great circles (lower hemisphere equal area) of fractures mapped at different localities, where c. N–S striking fractures are blue and fractures parallel to the foliation are red. Structural symbols outline tectonic boundaries along the margin and inside the rockslide area. The western boundary zone is a NNW–SSE striking and sub-vertical, strike-slip fault, whereas the eastern boundary zone is a pro-existing fault with hallow dip to the west. The upper boundary is seen as a pronounced back scarp. Low angle sliding surfaces daylights at two levels in the slope, marked with a red hatched line (sub-domain 2 and 4), while two sliding surfaces are mapped by geophysics, marked with red dashed lines (sub-domain 1 and 3). The foliation is folded and changes within the rockslide, strike 092/dip 44 (RHR) at the western boundary zone, 080/ in sub-domains 2, and the toe zone differs from 066/20 to 093/32 (*n*=137).

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and possible conductive minerals in the bedrock. For example, an 276277 increase in fracture frequency gives increased porosity, which reduces the resistivity (increased electrical conductivity). During data acqui-278279sition the probe is descended down the drill hole to log the electrical conductivity in the drill hole, calculated from the known current 280and the potential difference between electrodes (Rønning et al., 2006). 281 The maximum measurable resistivity with the logging equipment is 282 10,000  $\Omega$ m. If the true resistivity in the drill hole exceeds 10,000  $\Omega$ m, 283284 this is seen as constant maximum resistivity on the log. Due to the 285high conductivity in water the resistivity decreases in saturated fracture zones. It also decreases in clay-rich zones. As water and clay are 286extensively found in fracture and weakness zones, the aim for the 287resistivity analysis in drill holes is consequently to detect the low 288289resistivity zones.

#### 290 3.4.2. Sonic logging of P-wave in drill hole

The purpose of acoustic logging of drill holes is to determine the seismic velocity of the formation (bedrock). Seismic velocity is given in the unit metre per second (m/s). The probe consists of one transmitter and two receivers with an internal distance of 30.4 cm. The probe can automatically calculate the seismic P-wave slowness (inverse of velocity) from picks of first incoming P-waves for each 296 centimetre in the drill hole. For each 20 cm, the full waveform train is 297 transferred to the data logger, and from this, both P- and S-wave 298 velocity can be calculated. Water- or air filled fractures in rock will 299 increase the travel times and thus decrease the velocities, since the 300 P-wave velocity in air and water are 330 m/s and 1450–1530 m/s 301 respectively (Reynolds, 1997). By detecting the low velocity zones in 302 the drill hole, one can discover weaknesses in the bedrock that may 303 be relevant for defining the stability of the rock.

#### 4. Results

#### 4.1. Structural surface mapping

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The rockslide reveals gneissic bedrock that is folded. There are 307 significant variations in the orientation of the foliation, from very 308 steep and E–W striking in the upper part, near the back scarp, to E–W 309 striking and sub-horizontal in the lower parts (Fig. 2). This variation in 310 foliation occurs over a few tens of metres. In the boreholes the average 311 dip of the foliation is 31.7°, steepening slightly from top to bottom of 312 the slope (27° to 34°) (Kveldsvik et al., 2006).



**Fig. 4.** The foliation of the bedrock controls the development of the back scarp. Where the orientation of the foliation is favourable for extensional fracturing (i.e. when sub-vertical or dipping down slope), the back fracture follows the foliation. In contrast, where the foliation is not favourable for reactivation, the back fracture reuses pre-existing fracture sets that commonly have an E–W strike, and is steeply dipping. Locally, the back scarp zone splits into segments that form relay structures (B). A) Map showing the back scarp zone and the foliation along this zone experiencing extension. B) Example of site where the foliation is not favourable and the back (extensional) fracture has formed a relay structure between two larger extensional fractures, the latter following the foliation. C) Example of back fracture that is controlled by the foliation, which is sub-vertical and undulating due to mesoscopic folding. Both localities are monitored by extensometers, recording the horizontal and vertical movements along the back fracture.

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314 Structural bedrock mapping on exposures leaves a large part 315 unmapped due to cover of vegetation and/or scree. Three distinct fracture sets are mapped within the rockslide; steeply dipping 316 317 fractures with approximately N-S strike and E-W strike, and a third fracture set parallel to the foliation. The dominance and intensity of the 318 different fracture sets vary between localities. The N-S fracture set is 319 present at all localities; its strike varies from NNW to NNE-NE (Fig. 2). 320 In contrast, the E-W oriented fractures are not present at all localities, 321 322 but when present, are prominent. An example is the back scarp, which 323 partly follows E-W fracture(s) (Figs. 2 and 4). The trends of these steeply dipping fractures follow the main trends of lineaments in the 324region (Gabrielsen et al., 2002; Henderson et al., 2006), of which the 325most pronounced lineaments coincide with major fjords. Chronologi-326 cal data of these structures have not been assessed. 327

Both outcrop and drill core studies indicate an increase in fracture 328 frequency in and near biotite rich layers of the gneiss, and with the 329 lowest fracture frequency in the fairly homogenous granitic gneiss 330 (Ganerød et al., 2007). The fracture frequency in outcrops varies from 2 331 to 8 fractures per metre (f/m) in scan lines when measured parallel to 332 the foliation (see Section 4.1 for scan line description). Perpendicular to 333 the foliation, values as high as 23 f/m can be found. However, common 334 values are in the ranges of 8 to 12 f/m. In the drill cores, the fracture 335 336 frequency varies significantly, from 1 f/m in undisturbed rock to 50 f/m. The latter case is associated with fault rocks, such as breccias and 337 gouge, which appears in discrete zones. In general it is difficult to 338 distinguish between shear and extension fractures due to the lack of 339 markers. However, when there is evidence of separation perpendicular 340 341 to the fracture surface, the structures are called extension fractures.

Outside the rockslide, the fracture frequency is generally around 342 5 f/m. The highest fracture frequency occur perpendicular to the 343 foliation, reaching 9 f/m, while the lowest fracture frequency is pa-344345rallel to the foliation, with 2 f/m (Table 1). Fracture continuity of the 346different fracture sets has been estimated and varies within the different zones of the rockslide. For fractures outside the rockslide, the 347 continuity parallel to foliation is 5 to 10 m, and for N-S trending 348 fractures, 2 to 5 m. E-W trending fractures, while infrequent, are 1-2 m 349 350long (Table 1).

#### 351 **4.1.1. Back scarp zone**

The back scarp zone is approximately 800 m long (Figs. 2 and 4). In the west, the first 200 m is a cliff face that has seen one or more rockslides. Thereafter follows a 20–30 m deep and 10–50 m wide graben that shows ongoing extension. The remaining 500–600 m is an overall open fracture. The extension along the back scarp decreases from the west to the east. The back fracture has a scissor shape, where the maximum width of 20–30 m is found on the western side, while the width decreases towards the east, where the maximum width is 359 0.5–1 m. The depth of the extensional back fracture is difficult to 360 estimate, since the fracture is partially filled with scree, sediments and 361 ice. The estimated depth in the western part is 60 m, and likely decreases 362 to the east. The back fracture shows both vertical and horizontal se- 363 paration with a general extension in the N–S direction, directly down- 364 slope (Figs. 2 and 4).

A striking feature of the back scarp zone is the variability in 366 orientation of bedrock foliation. North of this zone the foliation is 367 nearly slope parallel (Fig. 2). In general, the back fracture is steep to 368 sub vertical (Fig. 4), but changes along strike as the foliation is folded. 369 The folds in the back scarp zone are on metre to decimetre scale, are 370 close to tight and normally symmetrical and have short wavelengths. 371 The axial surfaces are sub-horizontal, and the mean vector for the fold 372 axis is 27° towards ESE. This folding makes the foliation change from 373 sub-vertical to sub-horizontal over short distances as shown in Fig. 4. 374 Where the orientation of the foliation is favourable for extensional 375 fracturing (i.e. when striking ~E-W and dips sub-vertical or down 376 slope), the back fracture follows the foliation (Fig. 4A and C). In con- 377 trast, where the foliation is not favourable for reactivation (i.e. when 378 striking ~N-S and dips sub-horizontal), the back fracture reuses pre- 379 existing fracture sets that commonly have an E-W strike, and is 380 steeply dipping. Locally, the back scarp zone splits into segments that 381 form relay structures. Most relays are hard linked in that connecting 382 fractures are cutting across the foliation between segments (Fig. 4A 383 and B). In the vicinity of the back scarp, extension fractures sub- 384 parallel to the back scarp are common, showing a separation of 10 to 385 12 cm. Riemer et al. (1988) demonstrate that extension preferentially 386 develops along the fold axis, as seen in the back scarp zone at Åknes. 387

In the back scarp zone all three fracture sets mentioned above are 388 present. However, there is a dominance of N–S oriented fractures. The 389 fracture frequency of the back scarp zone in general is low (Table 1). 390 The length of fractures parallel to foliation is about 10 m. In contrast, 391 N–S trending fractures are shorter (<2 m), as are the E–W trending 392 fractures (2–5 m, Table 1). The combination of long and short con-393 necting fractures and low fracture frequency gives the back scarp zone 394 the largest block size of the site.

#### 4.1.2. Western boundary zone

The western boundary zone is the structural limitation of the 397 rockslide to the west (Fig. 2) and is defined by a steeply dipping, 398 NNW–SSE striking strike slip fault. This fault forms a crevasse that is 399 prone to snow and rock avalanches. The crevasse has cliff sides that are 400 10 to 40 m in height (Fig. 2). The boundary fault has an extent that 401 exceeds the rockslide area; it can be traced as a lineament for several 402 km. This pre-existing structure is old, probably dating back to the 403

#### t1.1 Table 1

Fracture frequency  $(m^{-1})$ , continuity of fractures sets, and block size of the different zones of the rockslide area

Fracture frequency (m <sup>-1</sup> )	Back-ground data	Back scarp zone	Western boundary zone	Eastern boundary zone	Internal zones	Toe zone
Non-foliation parallel fractures	2	2	2	3	8	4
Foliation parallel	9	5	7	13	17	11
fractures						
Block size						
(cm)	10-50	20-50	15–50	~10-30	5-20	10-25
Continuity of fracture s	ets (along strike, m)					
Foliation parallel	5-10	~ 10	6-10	10	6-10	10-20
fractures						
N–S fractures	2–5	0-2	2-10	2	~5	~2
E–W fractures	(1-2)	2–5	0.2-2		1-2	~1
NW-SE fractures			0.5-1			

t1.15 The value of each zone is an average based on 3 scan lines, perpendicular to and parallel to foliation, for each locality studied within that zone.

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404 Devonian (Andersen et al., 1997; Braathen, 1999; Osmundsen and405 Andersen, 2001).

In the western boundary zone, the fracture frequency is generally low (Table 1). Continuity of fracture sets reveals similarities to the back scarp, with lengths of 6 to 10 m for foliation-parallel fractures, 2 to 10 m for N–S trending fractures, and 0.2 to 2 m for E–W trending fractures (Table 1). In addition, there is a NW–SE trending fracture set with 0.5 to 1 m length. Within the western boundary zone, N–S oriented, steeply dipping fractures dominate. This fracture set is subparallel to the strike of the fault, which is defined by a zone of heavily 413 fractured rock (Fig. 2). In the upper part of this zone, there are several 414 extensional fractures (Fig. 5A). Some of these fractures follow the E–W 415 and N–S fracture patterns. Extensional separation along these 416 fractures varies from 10 to 50 cm. Three rockslide events have been 417 recorded along the western boundary zone (Kveldsvik et al., 2006, 418 2007). All three appear to have occurred as plane failures, with frac- 419 tures parallel to foliation acting as the basal sliding surface and N–S 420 and E–W oriented fractures acting as release surfaces.



**Fig. 5.** A) Extensional fractures located in the Åknes slope. In sub-domain 1 are pre-existing fractures oriented c. N–S and c. E–W reactivated by extension, with separation up to 50 cm. In sub-domain 2 are extension fractures that are slope parallel observed as shown by example B) and C) These extension fractures are oriented perpendicular to the movement direction (c. E–W trending and 60–90° dip to N–NNE), and are fractures caused by the movement of the rockslide.

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#### 422 4.1.3. Eastern boundary zone

The eastern boundary zone represents the eastern structural limitation of the rockslide area and is defined by a gently NW dipping, NNE–SSW trending fault. This structure is not well exposed in the topography (Fig. 2). The fault zone is characterized by heavily fractured rock sub-parallel to the well defined fault plane (089/48). No fault rock has been found along the fault zone.

The fracture frequency is higher perpendicular to the foliation (7 to 21 f/m), than parallel to the foliation (1 to 5 f/m) (Table 1). Fractures parallel to foliation are the longest, commonly in the range of 10 m, whereas N–S trending fractures reach 2 m (Table 1). E–W oriented fractures are absent. The foliation is in general dipping 44° to the south, which is steeper than the general trend of the foliation in the rockslide.

#### 435 4.1.4. Central zone

The central zone part of the rockslide area (Fig. 1B), within the
boundaries described above and below, reveals a sub horizontal to
gently folded foliation that dips moderately towards the fjord. Folding

causes an undulating geometry of the foliation. Locally, the foliation 439 reveals more intense folds causing significant variation in the orien- 440 tation. In areas where the bedrock is more intensely folded, there are 441 hilltops and scarps suggesting that these sites are more resistant to 442 denudation. A sliding surface in biotite rich to biotite-schist layers is 443 mapable in outcrops of the central zone (Fig. 2). This shear zone is 444 heavily fractured with a width of 20 cm, similar to that described 445 below and illustrated in Fig. 6. Gouge can be observed as pockets along 446 fractures, however most fractures have a rock-on-rock contact. This 447 sliding surface forms the lower limit of sub domain 2.

The fracture frequency of the central zone is the highest recorded in 449 the rockslide area, with the highest frequency perpendicular to the 450 foliation (average of 17 f/m, Table 1). Parallel to the foliation, the fracture 451 frequency is 8 f/m. The length of the fracture sets is comparable to the 452 other zones (Table 1). The high fracture frequency in combination with 453 fracture length gives the smallest block size (Table 1), consistent with 454 the observations that the central zone is heavily fractured and blocky on 455 the surface.



**Fig. 6.** A) Sliding surface of the toe zone where the basal thrust daylights the rock slope surface. Due to upward and forward separation, the transported block is pushed on top of the vegetated slope, causing the formation of a rock overhang that partly has caved in. B) Fault gouge is located as a thin layer (1–2 cm) with undulating thickness along the shear surface. (C) Example of a network of thin gouge layers that fill fractures within the intensely deformed zone of the sliding surface. This sliding surface is exposed for at least 50 m along strike, and the thickness of the gouge rich zone is approximately 20 cm on average along the exposure. The moss growing on the fractured zone indicates water seepage.

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Fig. 7. Geophysical data from Åknes. A) Map showing all the geophysical profiles collected in the rockslide area. The presented profiles b-d are only a selected section of a profile. Profile c and d are the same section, c. 450 m long, while b is 250 m long and its location is marked in profile d. B) GPR profile with NNW-SSE orientation and approximate 35 m depth penetration. Above, profile without interpretations, below; profile showing shallow, undulating structures, interpreted to be foliation-parallel layers that crop out at the slope surface (marked with red lines). C) Refraction seismic; above travel time (ms) vs. distance (m) for first arrival P-wave traces. Below, schematic profile showing four layers with increasing velocities with depth. A-A' is line of cross section for thickness estimates. D) 2D resistivity profile, measured with Wenner configuration, showing layers with different resistivity. Above, profile without interpretation, below; profile showing an undulating resistivity contrast, interpreted as a sliding surface at the bottom of the low resistivity (blue) layer marked with black dashed lines. The profile has the same location as the seismic line (c).
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A striking feature in sub domain 2 is the occurrence of large 457458 extensional fractures striking approximately E-W, perpendicular to the direction of movement of the rockslide area (Fig. 5). These ex-459 460 tensional fractures have an irregular shape and have a dip of 60° to 90° in a northerly direction. The separation on these fractures is slope 461 parallel, and varies from 40 cm to 2 m (Fig. 5). Extensional movement 462 can also be seen on N-S striking fractures, with separations of 10-463 30 cm, which are mapped throughout sub domain 1 and 2. The latter 464 465described fractures are probably rather frequent, but abundant co-466 verage of scree and vegetation makes this a qualitative assessment.

### 467 4.1.5. Toe zone

The toe zone is defined by a major sliding surface that daylights the 468 469 surface (Fig. 6). When observed, this sliding surface is near-parallel but shallower dipping than the topographic slope, with an orientation 470differing from 066/20 to 093/32. In the hanging wall, rocks are 471transported both upward and down-slope forming rock overhangs, at 472places developed into narrow, shallow caves. In these overhangs, slabs 473 of rock have broken off along the foliation, emphasising the position of 474 the sliding surface (Fig. 6A). The sliding surface is defined by fault rock 475along biotite rich layers of the bedrock, as shown in Fig. 6. Locally, the 476 sliding surface is made up of a narrow (<20 cm) heavily fractured 477 478 zone. Along the long, nearly continuous exposures of the sliding 479 surface, the fault rock interval is up to 2 cm thick (Fig. 6B). The gouge layers form along-strike, metre-long lenses and/or continuous 480 membranes of variable thickness, commonly in the range of some 481 millimetres to a few centimetres. They are also seen in a network of 482 483 shear fractures that are filled with gouge (Fig. 6C). The gouge is fine grained and is light grey to dark grey. It contains clay size minerals 484 with some (10-20%) rock fragments. Gouge mineralogy derived from 485XRD-analysis includes micas, quartz and plagioclase, with micas 486 487 spanning from smectite, chlorite, and kaolinite to serpentine. Where 488 the sliding surface is defined by a heavily fractured zone, parts of the 489 sliding surface are characterized by rock-on-rock contact. Groundwater springs are common along the sliding surface (Fig. 6), where 490 both seepage and discrete outlets form. 491

The fracture frequency in the toe zone is in general 11 f/m for foliation parallel fractures and 4 f/m for non-foliation parallel fractures (Table 1), with the foliation-parallel fracture set dominating (Figs. 2 and 6). The length of the foliation-parallel fractures is 10–20 m, whereas the N–S trending fractures are about 2 m and the E–W trending fractures approximately 1 m (Table 1).

### 498 4.2. Subsurface mapping

A total of seven boreholes were drilled and cored at three sites in the rockslide area. Three holes are vertical to 150 m depth; one is inclined by 60° and goes down 150 m, while the remaining three are vertical and 200 m deep. All cores have been logged (Ganerød et al., 2007). The drill hole presented here is from the lowest drilling site that goes down to 150 m depth (Figs. 2 and 8).

### 505 4.2.1. Ground Penetrating Radar (GPR)

The profile shown in Fig. 7B is a 250 m long segment of a profile 506with a total length of 1150 m. Close to the surface, the reflectors of the 507GPR profile are parallel to the slope with a thickness of 1 to 3 m. This 508 509first layer is interpreted to be scree material or other debris (Fig. 7B). Reflectors at depth that are sub-parallel to the surface are interpreted 510 to reflect the foliation-parallel fractures in the bedrock (Fig. 7B). Two 511 of the interpreted reflectors (marked in red, Fig. 7B) daylight the 512surface in the same area as mapable features of the seismic and 2D 513resistivity profiles, indicating that multi-property layers are day-514lighting (Fig. 7). This is interpreted to be a sliding surface. Due to the 515 limited depth penetration, GPR cannot give any information of the 516 extent or depth of the sliding surface. However, the method gives 517 518 detailed information of the shallower subsurface. The second layer, about 25 m thick, is interpreted to consist of heavily fractured and 519 drained bedrock, and is comparable to the second zone identified in 520 the seismic and 2D resistivity profiles as well as in the drill hole. 521

### 4.2.2. Seismic

The seismic profile can be divided into four zones or intervals. The 523 first, near surface zone is up to 5 m thick and has a seismic velocity of 524 350 m/s. This zone (indicated in yellow colour in Fig. 7C) is interpreted to 525 consist of scree material. The second zone has an approximate thickness 526 of 30 m in the upper part and thins down to 3-5 m down slope; the 527 seismic velocity is about 1900 m/s. This zone is interpreted to consist of 528 heavily fractured rock that is unsaturated (light green colour Fig. 7C). The 529 third zone is approximately 65 m thick, extending down to about 100 m 530 depth (green colour). It has a seismic velocity of 3800-3900 m/s. This 531 interval likely consists of water saturated, fractured bedrock (Fig. 7C). 532 The fourth and deepest zone extends below the seismic resolution, and 533 has a seismic velocity of 5500 m/s. This indicates good rock quality and 534 is consistent with less fractured rock that is water saturated (orange 535 colour, Fig. 7C). A schematic cross section is presented in Fig. 6C to 536 illustrate estimated thickness of the different zones, which also match 537 thicknesses from the drill hole (Fig. 8). 538

Between 300 and 330 m along the seismic profile, a low angle zone 539 reaching the surface has a seismic velocity of 2500 m/s. This is a lower 540 seismic velocity than the surrounding material at 3900 m/s, consistent 541 with high fracturing or porous rock (fault rock). The mapable, low 542 velocity layer has a length of 30 m at the surface. Due to methodological 543 weakness it is not possible to map the dip of the zone. The low velocity 544 layer might be a rather thin zone lying as a hidden layer between the 545 1900 m/s (light green) and 3900 m/s (green) layers (Fig. 7C).

### 4.2.3. 2D resistivity

The 2D resistivity profile in Fig. 6D shows zonation that is inter- 548 preted to consist of an approximately 5 m thick layer of scree material 549 at the top (red to orange colour). Below that is a 10–20 m thick layer 550 (light green colour) interpreted to be heavily fractured and drained 551 rock. The next layer has the lowest resistivity (blue colour). This layer 552 is about 20 m thick and is interpreted to consist of heavily fractured 553 rock that is water saturated. This low resistivity layer is undulating 554 and, when followed down-slope, daylights the surface at about 555 1200 m (blue colour, Fig. 7D). Another segment of the low resistivity 556 layer continues down-slope. The sliding surface is interpreted as being 557 located at the bottom of this low resistivity layer (blue colour). Near 558 the base of the profile is a layer with medium resistivity (green 559 colour), interpreted to consist of less fractured and water saturated 560 bedrock (Fig. 7D).

### 4.2.4. Drill hole data

The bedrock logged in the drill cores is the same heterogeneous 563 gneiss as mapped in outcrops, altering from medium grained granitic 564 gneiss, through dioritic gneiss to biotite-rich gneiss (Fig. 8). The folded 565 foliation mapped in outcrop scale (metres) is confirmed in centimetre- 566 scale in the cores, where the folds are close to tight with short 567 wavelength (Fig. 8B). The dominating fractures in the drill core are 568 parallel to the foliation, while the other two existing, steep fracture sets 569 mapped in outcrops are under-represented in the vertical drill hole. The 570 fracture frequency of the drill cores decreases with depth, as shown in 571 Fig. 7. In the upper few metres (0-5 m), fracturing is intense and core loss 572 is abundant (Fig. 8A). The following interval (5-42 m) is heavily 573 fractured with a fracture frequency up to 22 f/m. Several zones up to 574 40 cm thick with crushed rock occur in the interval 18 to 23 m, one with a 575 fracture frequency up to 50 f/m at 22 m depth (Fig. 8A). Fault rocks, such 576 as gouge, occur in narrow zones of up to 5 cm thickness, while a 30 cm 577 thick breccia zone is observed at 21.5 m depth (Fig. 8C). This interval is 578 unsaturated, since the water table is located at 42 m depth (Fig. 8A). 579 However, the groundwater level fluctuates seasonally. The first two 580 intervals have no recording of resistivity or P-wave velocity in bedrock 581

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**Fig. 8.** A) Drill hole and drill core data that show that the fracture frequency decreases with depth. Five zones can be identified on the basis of fracture intervals and responses in the resistivity and P-wave logs of the borehole walls. The colour codes are based on the zonation in the 2D resistivity profile given in Fig. 6D. This colour code is implemented on all subsurface data, to visualise the zonation in the subsurface, with the exception that the blue and green zones in the drill core and 2D resistivity coincide with the green zone in the seismic. B) Drill core showing that the bedrock is a granodioritic gneiss with a folded penetrative foliation on cm-scale. C) An up to 30 cm thick (proto-) breccia that occurs at 21.5 m depth.

due to the lack of water in the drill hole (Fig. 8A, Rønning et al., 2006). 582The next interval (42-70 m) consists of heavily fractured rock that is 583 water saturated. This section has a fracture frequency up to 25 f/m, with 584an average of 7–9 f/m (Fig. 8A). There are large irregularities in both 585 586 resistivity and P-wave response, giving very low values at several depths in this interval. The suggested location of the sliding surface in the drill 587 core is at 62 m depth, where a zone of heavily fractured rock (25 f/m) and 588fault rock occurs. The sliding surface is thin (max 20 cm) and the 589590occurrence of gouge and ultrabreccia is patchy with a maximum thickness of 1-3 cm (Fig. 8). The following interval (70-115 m) is less frac-591tured, with an average of 5-10 f/m. The interval is water saturated 592593(Fig. 7A). The resistivity and P-wave response for this interval reveal high resistivity levels from 8000 to more than 10,000  $\Omega$ m and a seismic 594 velocity of about 5000 m/s (Fig. 8A). There is a local minimum in both 595 resistivity and P-wave at 115 m depth that coincides well with a 50 cm 596 zone of heavily fractured rock in the drill core that has a fracture fre- 597 quency of 50 f/m (Fig. 8A). The deepest interval (115–150 m) has a low 598 fracture frequency (3–5 f/m), which is regarded as background frac- 599 turing. The drill core section is divided into five intervals (Fig. 8A).

### 5. Recorded movements of the slope

The rockslide area at Åknes was brought to the public's attention in 602 1964 by local residents claiming that the back scarp was widening 603

601

**Fig. 9.** Average annual slope movements recorded in the period 2004–2006. The vectors represent average total displacement per year. Measurement points no. 1–8 and 14–23 were established in 2004, while point no. 9–13 was established in 2005. Recordings from points no. 6 and 9–13 are not consistent or significant, and thereby not represented by a displacement vector. Maximum movements are recorded on the upper part of the slope, close to the back scarp and along the crevasse to the west (western boundary zone). The extensometers are placed in telescopic steel pipes fixed on each side of the back scarp and are N–S oriented.

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(Sandersen et al., 1996). Monitoring of the back scarp started in 1986 604 605 with the installation of two pairs of bolts for manual reading. In 1989, an additional five pairs of bolts were installed. A continuous sur-606 607 veillance program started in 1993 with three extensometers, and was extended to five extensometers in 2005 (Fig. 9). Measurements in-608 dicate a more or less steady rate of movement in the back scarp from 609 1986 to the present (Braathen et al., 2004; Kveldsvik et al., 2006); the 610 long-term average velocity varies from 1 to 3 cm/year. The angle 611 612 between extensometers 3, 4 and 5 and the strike of the back scarp is 613 not perpendicular, which implies that the true displacements per year exceed the recorded displacements. 614

A network of GPS-points and reflection prisms for total station 615 geodetic monitoring was established throughout the area in 2004 616 (point no. 1-8 and 14-23) and 2005 (point no. 9-13, Fig. 9). The 617 analysis of the surface movements is based on recordings from 13 GPS 618 points and 16 prisms, of which 5 of the monitoring points were mea-619 sured by both GPS and total station. Measurement campaigns have 620 been carried out twice a year, and show consistent movements in the 621 upper part of the area for most points (Eiken, 2006). In the lower part, 622 the area is vegetated and unsuited for reflection points. Furthermore, 623 the GPS measurements are hampered by uncertainty, due to unfavour-624 able conditions for satellite signal reception. The dataset is summarized 625 626 with displacement vectors in Fig. 9, representing annual displacements, calculated as the average displacement from the two years of recording. 627 The orientation of the vectors is shown as the lateral orientation of the 628 total displacement vector at the last recording in 2006. 629

Two laser distance metres were installed in 2005 for monitoring of the graben structure along the western part of the back scarp (Fig. 9). The graben structure, with an estimated volume of 10–14 million m<sup>3</sup>, is believed to move independently from the rest of the slope failure, with a velocity on the order of 14 cm/year in an S–SE direction (GPS point no. 3).

636Surface displacements indicate creep of the whole unstable rock637slope, divided into two separate sliding bodies, sub-domain 1+2 and

sub-domain 3+4. The maximum movements are recorded on the 638 upper part of the slope, close to the back scarp and the crevasse to the 639 west, which explains the scissor shape of the back fracture with large 640 displacements to SW-SSW at the western side and smaller displace- 641 ments to S-SE at the eastern side (Fig. 9). Sub-domain 1, includes the 642 graben structure moves differently than the other domains with a 643 larger displacement rate in SW to SSW direction (point 18-20, Fig. 9). 644 This is most likely due to the lack of lateral support on the western 645 flank. Sub-domain 2 shows a lower magnitude of displacement, with 646 the vectors oriented in S to SSE direction (point 8 and 17, Fig. 9). In sub- 647 domain 3, represented by points no. 4, 5 and 8, displacements are 648 recorded toward SE on the order of 1.5 to 2.5 cm/year. In sub-domain 649 4, measurements are not consistent and some of the points do not 650 show a significant displacement during the period 2005-2006 (Fig. 9). 651 However, all points (points 10-13) in the lower part suggest a positive 652 elevation change on the order of 1 to 3 cm/year (Fig. 9). This indicates a 653 zone of compression and thereby thickening above a sliding surface in 654 the area, supporting existence of the toe zone mentioned earlier. 655

### 6. Discussion

### 6.1. Geological model based on observations

Observations and interpretations of data described above form the 658 basis for the geological model of Åknes rockslide area, as illustrated in 659 Fig. 10. The style of deformation, with bearing on the geometry of the 660 rockslide area, is that of an extensional fault system at the top and an 661 imbricate thrust fan further down-slope. In other words, the rock 662 slope failure can be divided into two; an upper part experiencing 663 extension (Figs. 1 and 4), and a lower part deforming by compression 664 (Figs. 1 and 6). The basal sliding surface split into four subordinate 665 layers that are partially stacked upon each other and bound by sliding 666 surfaces, two that daylight the surface (sub-domain 2 and 4) and two 667 indicated by geophysics (sub-domain 1 and 3, Fig. 10). The depth to the 668

![](_page_75_Figure_11.jpeg)

**Fig. 10.** Geological model of the Åknes rockslide. The block diagram summarizes field observations and geophysical data. Interpretation of structural data correlated well with structural interpretation from the 2D resistivity profiles, suggesting an undulating basal sliding surface with subordinate sliding surfaces that crop out at several levels of the slope. The rockslide can be divided into four sub-domains with different structural characteristics and displacement patterns, where two of the marked sliding surfaces are mapped at the surface and two sliding surfaces are interpreted from geophysical data (2D resistivity). The depth to the basal sliding surface varies, but in general increases towards the west. Structural constraints of the rockslide area to the east and west are pre-existing faults. The top is delineated by an extensional back scarp. The to zone forms the lower limit of the rockslide are. The foliation is folded, especially in the back scarp zone, but also shows gentle variations down-slope and across-slope, as indicated in the figure by black dashed lines.

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sliding surfaces differs within the four layers. In an east-west crosssection, the depth to the sliding surface shows a general increase to
the west and decrease to the east. Down-slope, the sliding surfaces
have roughly the same depth, and cut up-section near the toe of the
thrust sheet (Fig. 7). The length and width of the sliding blocks are
fairly similar (Fig. 10).

To summarize, the structural mapping of the area shows that the 675 foliation is undulating along and across the slope. It controls the 676 677 development of the back scarp and the basal sliding surfaces and its subordinate thrust zones. Along the back scarp, extension perpendi-678 679 cular to the fold axis is common when it is favourable, giving open 680 fractures along the sub-veritcal foliation (Fig. 4C). Down-slope the foliation is sub-parallel to the topographic slope, and where the 681 682 foliation dips shallower than the slope, there are thrusts daylighting the surface as seen at two levels down-slope (Figs. 2 and 6). The 683 rockslide area can be further divided into five zones based on surface 684 characteristics, where the upper structural limitation of the rockslide 685 is the back scarp zone with its scissor like shape (Figs. 2, 4 and 9). The 686 separation along the back scarp zone is gradual and constant on a 687 yearly basis, with a larger movement of the western side than on the 688 eastern (Fig. 9). The western boundary zone is the structural western 689 limit of the rockslide area with a prominent crevasse formed along a 690 691 NNW-SSE trending strike slip fault. The displacement data show that the upper western part of the rockslide area has the largest movement 692 rates (cm/year), with a SW displacement vector towards the crevasse 693 (Fig. 9). The eastern boundary zone is inferred as the eastern structural 694 limit of the rockslide area (Figs. 2 and 9). The displacement along the 695 696 eastern boundary zone is low or absent, as shown in Fig. 9, most likely due to lack of points of measurement or low satellite signals in the 697 densely vegetated and steep terrain. However, the movement is 698 inferred to be rotational with maximum rotation in the upper part of 699 700the zone. The central zone of the rockslide can be divided into four 701 sub-domains separated by subordinate, low angled, sliding surfaces mapped on the surface and interpreted from geophysics (Figs. 2 and 702 10). In sub-domain 2, several extensional fractures with slope-parallel 703 separation of up to 2 m are located (Fig. 5). These structures have 704 formed perpendicular to the displacement direction (Fig. 9). The total 705 706 displacement of sub-domain 1 and 2 is larger than that measured for the sub-domain 3 and 4 (Fig. 9). The latter sub-domain has less 707 distinct features, but the occurrence of springs increases down-slope. 708 The toe zone is the lower limit of the rockslide and is defined by a 709 subordinate sliding surface that can be observed at the surface (Fig. 6). 710 The sliding surface is mapped as a low-angle thrust that is more or less 711 continuous. The displacement data for the toe zone is somewhat 712 contradictory, but the general trend seems to be a considerable portion 713 of upward movement (Fig. 9). This fits well with the interpretation of 714 715 the sliding surface as a thrust ramp that daylight the surface, consistent with compression in the toe zone (Fig. 10). 716

An interpretation of the eight 2D resistivity profiles in a tied grid, 717 supplemented with drill hole data, GPR and seismic profiles, formed 718 the basis for the mapping of the subsurface with respect to the sliding 719 720 surfaces (Fig. 7) and, furthermore, the geological model of the rock-721 slide area (Fig. 10). The structures interpreted in the subsurface from geophysical data coincide well with structures mapped on the surface. 722 In addition, the geophysical data indicate the position of the sliding 723 surfaces in the subsurface (Figs. 7 and 8). The two sliding surfaces 724 725mapped on the surface (Fig. 2) are also well covered with geophysical data, indicating the extent of the sliding surfaces where they are 726 covered by vegetation and scree. Furthermore, this forms the basis for 727 the extrapolations of the sliding surfaces where they are not exposed 728 (Figs. 2 and 10). The geophysical data indicate two additional sliding 729 surfaces (Fig. 10), with similar signatures to the two observed at the 730 surface. This divides the central zone into four sub-domains as indi-731 cated in Fig. 10. All the sliding surfaces interpreted by 2D resistivity 732 have an undulating character, and daylight the surface along the slope. 733 734 They are shown as low resistivity zones and the sliding surfaces are interpreted to be located at the bottom of these zones (Figs. 7 and 8). 735 The geophysical data also supports the location of the larger structures 736 forming the western boundary, as a sub-vertical structure, and eastern 737 boundary as a gentle, westward dipping structure. 738

The estimated area and succeeding volume of the four sub-domains 739 are given in Table 2. The preliminary volume estimate of the rockslide 740 is based on a sliding surface at 50 m depth, therefore volumes of the 741 sub-domains with a 50 m deep sliding surface is included in the 742 estimate (Table 2). A basal sliding surface at 65 m depth, as presented 743 here, is more likely and following volume estimates are given (Table 2). 744

The displacement pattern of the rockslide area is complex (Fig. 9). 745 Extension in an N–S direction characterizes the back scarp zone. A 746 division into four sub-domains fits well with the pattern of move- 747 ments of the rockslide area. Sub-domain 1, with the graben structure 748 to the west comprises the area with the largest displacement rates, up 749 to 14 cm/year, and has displacement in a S–SW direction (Fig. 9). The 750 displacement rate is less in the other sub-domains and in general 751 decreases down-slope (Fig. 9). Sub-domain 2 shows a more southerly 752 to southeast displacement direction with up to 5 cm/year (Fig. 9). Sub- 753 domain 4 has no displacement direction but shows slight (vertical) 754 upwards movement (Fig. 9), which sustain the indications of com- 755 pression of the toe zone (Fig. 10). 756

All observations of the basal sliding surface indicate that it is 757 undulating. Undulations can be caused by three scenarios; 1) growth 758 of the sliding surface as segments, where the undulating composite 759 surface is made up of several connected slip planes and where undu-760 lations are found at broken fault segments. 2) heterogeneous biotite 761 schist layer extent and distribution, where layers of biotite schist are 762 linked by fractures, and 3) reactivated folded foliation. Due to the 763 evidence that the foliation is folded, as shown in Figs. 2, 3 and 7, and 764 the fact that the foliation controls the development of the basal sliding 765 surface with its subordinate low angle thrust zones (Fig. 6), the reactivated, folded foliation model is plausible, either as the main con-767 trolling factor, or in combination with the other two. 768

### 6.2. Geological model in light of other studies

Considering Varnes` (1978) classification of landslide types, Åknes 770 rockslide does not simply fit into one category but forms a complex 771 landslide, which is a combination of two or more principal types of 772 movement. Creep is considered to be continuous in the Åknes rock- 773 slide, contributing to the general deformation of the bedrock and 774 displacement of the rockslide. In the upper part of the Åknes rockslide, 775 sub-domain 1 and 2, indications of rotational slide movement is 776 observed in the combination of vertical and horizontal movement of 777 the back scarp as well as backward tilting of blocks (Fig. 9). Trans- 778 lational slide movement is observes along the western flank of the 779 rockslide, especially in sub-domain 1, where also the largest displace-780 ment rates are measured. Abundant extensional fractures (tension 781 cracks) in a range of sizes are also observed along the western flank, 782 both with ~N-S and ~E-W strike. However, the toe zone and the 783 eastern flank, primarily sub-domain 3 and 4, of the Åknes rockslide 784 does not fit within Varnes (1978) classification. This is hereby argued 785 due to control of pre-existing structures such as the fault forming the 786 western boundary zone and the ductile fabric of the bedrock, i.e. the 787 foliation controlling the basal sliding surface with its subordinate 788

Table 2				t2.1		
Approximate area and volume estimates of the four sub-domains						
	Approvimato	Min volume $(m^3)$	May yoluma (m <sup>3</sup> )	t2.2		

	Approximate area (m²)	Min volume (m <sup>3</sup> ) – 50 m depth	Max volume (m <sup>3</sup> ) – 65 m depth	t2.3
Subdomain 1	102,264	5.1 million m <sup>3</sup>	6.6 million m <sup>3</sup>	t2.4
Subdomain 2	118,755	5.9 million m <sup>3</sup>	7.7 million m <sup>3</sup>	t2.5
Subdomain 3	155,787	7.8 million m <sup>3</sup>	10.1 million m <sup>3</sup>	t2.6
Subdomain 4	117,454	5.8 million m <sup>3</sup>	7.6 million m <sup>3</sup>	t2.7

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thrusts that daylights at several levels, but is most prominent in the toe 789 790 zone (Figs. 2, 6 and 10). Another argument for the distinctly different appearance is that the Åknes rockslide has not evolved far enough to 791 792 display the common structures occurring in "the zone of accumulation" according to Varnes` (1978) classification. Anyhow, the basal 793 sliding zone with its subordinate sliding surfaces observed at Åknes 794 rockslide form thrusts, giving compression, and display as an imbri-795 cated thrust fan (Fig. 10). 796

Braathen et al. (2004) define Åknes rockslide as a rockslide area, 797 798 given that the rockslide area has a relative low gradient ( $<45^{\circ}$ ), 799 weakness zones sub-parallel to the surface, and movement in the 800 lower parts leading to failures in the upper part of the slope. These weakness zones are, for example, foliation and/or layering or pre-801 802 existing fractures oriented sub-parallel to the slope. Results from this study support the view that Åknes is a rockslide area, but with more 803 complex structures (Figs. 1 and 10). Braathen et al. (2004) further 804 address the rockslide kinematics in a down-slope direction. Results 805 from this study imply that Åknes fits with Braathen et al.'s (2004) 806 model of the combined extension-compression scenario, predicting a 807 high frontal friction. 808

Oppikofer and Jaboyedoff (2007) proposes another type of model 809 of Åknes rockslide based on DEM (digital elevation model) and 810 811 ground-based Lidar image analyses of Åknes and surrounding occurred and potential rockslides. They use the asperity-amplitude 812 method to estimate the roughness of the foliation which gives the 813 geometry of the basal sliding surface, and analysis of spatial distri-814 bution of steep fractures. This provides a model with primarily planar 815 816 back scarps and a stepped basal sliding surface, where the rockslide is gravitational driven giving mainly translational movement. This is 817 a model similar to that of Eberhardt et al. (2004) described as 818 "sequential failure with internal shearing with yielding of rock 819 820 bridges" or "multiple step-path failure with intact rock bridges". The 821 sequential failure model is similar to that proposed for the Randa rockslide (Eberhardt et al., 2004). Eberhardt et al. (2004) demonstrate 822 that where sliding surfaces are predefined and controlled by pre-823 existing structures less internal rock deformation is needed to achieve 824 failure of the rockslide. Tension cracks are commonly an indication of 825 826 internal rock deformation, and with regards to Åknes rockslide both persistent pre-existing structures and tension cracks are observed, 827 indicating a more complex deformation mechanism with possibly a 828 combination of brittle and ductile behaviour (Eberhardt et al., 2004). 829 830 This may be the essential difference between the model of Åknes rockslide presented by Oppikofer and Jaboyedoff (2007) and the one 831 proposed in this work. The model by Oppikofer and Jabovedoff (2007) 832 is not considering the properties of the bedrock as strongly and show-833 ing brittle behaviour, while we propose a model based more on ductile 834 835 behaviour due to the control of pre-existing structures in the bedrock. The most realistic model of Åknes rockslide is probably a combination 836 of that proposed here and that of Oppikofer and Jaboyedoff (2007). 837 However, the morphology of the rockslide models after failure is likely 838 to be similar. 839

840 Giraud et al. (1990) gives examples of rockslides controlled by pre-841 existing structures, such as foliation, where the slope parallel foliation forms potential slip planes, which increase the potential of rockslides 842 progressing into rock avalanches with minor changes in physical or 843 hydrogeological conditions as trigger. These are conditions that are 844 845 valid for Åknes rockslide, and emphasises the importance of preexiting structures as a controlling factor. Giraud et al. (1990) also argue 846 that rockslides with pre-existing structures controlling the slip surface 847 are more likely exposed to translational or rotational types of move-848 ment. Another model that has been proposed is the deep-seated slope 849 gravitational deformation model (DSGSD); a model that shows only 850 extension (Agliardi et al., 2001; Crosta and Agliardi, 2003). An 851 example of such DSGSD is the Ruinon rockslide (20 million m<sup>3</sup>) of 852 the Italian Alps. This deep-seated slope gravitational deformation 853 854 indicates one deep, more or less continuous sliding surface, and collapse of the lower part of the slope (Agliardi et al., 2001). At Åknes 855 there is no indication of collapse in the lower part of the slope and 856 there is evidence of several sliding surfaces as well as a combined 857 extension–compression regime, making this model less viable for this 858 site. Seno and Thüring (2006) propose several different landslides 859 models, based on case studies from the Swiss Alps, varying from 860 rotational rockslides to rock slump, sag or deep-seated creep and 861 retrogressive landslide. However, these examples seems to be driven 862 by gravitation, commonly triggered by alteration in groundwater level, 863 and not controlled by pre-existing structures such as faults and ductile 864 deformation of the bedrock, even though two of the case studies have 865 slope parallel schistosity (Seno and Thüring, 2006).

Numeric modelling tools are widely used for kinematic analyses 867 and stability calculation of rock slope (Eberhardt et al., 2004; Stead 868 et al., 2006), however, due to the large uncertainties in input para- 869 meters the use of numerical modelling is mainly limited to back 870 analysis (Meric et al., 2005). Therefore, the need for geological data to 871 constrain the models is critical, and this work is an attempt of achiev- 872 ing constraints on geological parameters that will be applied in nu- 873 meric models (Kveldsvik et al., 2007; Nøttveit et al., in preparation). 874

### 6.3. Fracture distribution

Two hypotheses are entertained for the existing fracture sets 876 mapped in the rockslide area. Firstly, they are pre-existing fractures, 877 probably of Devonian age, which are reactivated due to movements on 878 the basal sliding surface. Since all recorded fracture sets are present 879 throughout the rockslide area (Fig. 2), this indicates that the fractures 880 are pre-existing. Some of the fractures are reactivated due to shear 881 movement along the basal sliding surface, which coincides with 882 results of Henderson et al. (2006) from regional studies in the vicinity 883 of Åknes. In the second hypothesis, the fracture sets are caused by 884 shear movement along the sliding surface. Slope parallel fractures or 885 fractures that form perpendicular to the displacement direction are 886 indications of fractures caused by movement of the rockslide. Sub- 887 vertical extensional fractures (i.e. tension cracks), which seem to occur 888 randomly, have a strike (~E-W) more or less perpendicular to the 889 direction of movement (SSW) are observed in sub-domain 2 (Figs. 2, 5 890 and 10). This phenomenon is also observed in other sites in the vicinity 891 (Henderson et al., 2006). At Åknes both types of fractures occur, but the 892 majority of fractures are reactivated older structures. In addition, 893 logging of the drill cores show that the fracture frequency decreases 894 with depth, indicating reactivation of pre-existing fractures and/or 895 foliation rather than initialization of new fractures (Fig. 8). 896

### 6.4. Complex groundwater system

The groundwater system of the rockslide area is fed by precipitation 898 both in the rockslide area and in the catchments area. Several seasonal 899 streams flow into the back scarp. Runoff from snowmelt in the 900 springtime brings significant volumes of water into the rockslide area. 901 Several springs are observed at the site and groundwater seepage is 902 common along the observed sliding surfaces. The area beneath the toe 903 zone reveals abundant springs. In the lower part of the rockslide area, 904 towards the fjord, the groundwater table is close to the surface. This is 905 reflected in the shallow penetration depth of the GPR data from the 906 area. Water chemistry, including conductivity, temperature, pH, anion 907 and cation composition from the springs, may reflect the retention 908 time of the groundwater (Derron et al., 2007). The dataset indicates 909 short retention time in sub-domain 2, while springs from sub-domain 910 3 and 4 seemingly have longer retention times. The lowermost springs, 911 beneath the toe zone, indicate the longest retention time. In total, the 912 groundwater chemistry supports the zonation of the rockslide area, as 913 suggested in the geological model (Fig. 10). 914

In the boreholes, the groundwater table is between 42 and 52 m 915 depth (Fig. 8). The water table fluctuates seasonally, increasing by as 916

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further work will elaborate on this issue. The displacement measurements especially in sub-domain 4 987 (points 10-13) show divergence in direction and rates. There is incon-988 sistency among the four lower points, making it difficult to draw 989 sound conclusions. The source of error of the measuring instrument is 990 regarded as 4 mm, and according to Demoulin et al. (2007) variation in 991 groundwater level may contribute to mm-scale of local different vertical 992 displacement. However, all points in sub-domain 4 show an (vertical) 993 upward movement that exceeds the level of uncertainty.

are given on the movements at deeper levels, hence, not included in 981

the proposed model in Figs. 1 and 10. The intention is to propose a 982

### 7. Conclusions

- The aim of this work was to propose a geological model of the Åknes 996 rockslide based on structural mapping and interpretation of geo- 997 physical data from profiling and boreholes. 998
- The folded foliation controls the development of the back fracture, 999 Where the orientation of the foliation is favourable for extensional 1000 fracturing (i.e. when sub-vertical or dipping down slope), the back 1001 fracture follows the foliation. 1002
- The folded foliation controls the development of the basal sliding 1003 surface with its sub-ordinate sliding surfaces as low angle trusts; i.e. 1004 the sliding surface is undulating due to gentle folds in the foliation of 1005 the bedrock. The sliding surfaces are mapped where they daylight 1006 the surface, and are characterized by the occurrence of fault rocks 1007 such as gouge and breccia. In general, the depth to the sliding sur- 1008 faces varies due to the undulation, generally increasing to the west 1009 and decreasing to the east, with a maximum depth of 65–70 m. 1010
- The rockslide area is divided into four sub-domains, confined by 1011 sub-ordinate low angle thrusts that daylight the surface. These sub- 1012 domains have different displacement patterns and rates, and have 1013 the down-slope geometry of an imbricated fan. Extension char- 1014 acterizes the two upper domains of the rockslide whereas com- 1015 pression characterizes the two lower domains. 1016
- The rockslide area is structurally confined with the upper rockslide 1017 limit formed by the back scarp zone, whereas a pre-existing NNW- 1018 SSE strike slip fault forms the western boundary zone. The eastern 1019 boundary zone is a gentle westward dipping pre-existing fault, and 1020 the toe zone forms the lower limit, where a sliding surface daylights 1021 the surface. 1022

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much as 5 m during snowmelt. The drill holes indicate a complex 917 918 groundwater system, with several inflows and outflows at different 919 depths and possibly perched groundwater aquifers. The latter is seen 920 by different temperatures and water conductivity. The depth to the water table in the drill holes is similar to the top of the low resistivity 921 layer (blue) in the 2D resistivity profiles (Figs. 7 and 8). 922

Hydraulic factors play an important role as contributing factors to 923 failure of all types of landslides (Giraud et al., 1990). For example, 924925studies of landslide in the Alps show that a large increase in rainfall over a short time period results in an increase in displacements (Giraud 926 927 et al., 1990; Crosta and Agliardi, 2003; Crosta et al., 2004; Seno and 928 Thüring, 2006). The heavy rainfall and additional runoff from glaciers 929 also worked as a triggering factor for accelerating displacement for the 930 Val Pola landslide in the Italian Alps (Crosta et al., 2004). Seno and Thüring's (2006) study show that alternations in groundwater level, 931 giving changes in pore pressure, is considered to be the main causes of 932 instability. In this light, melt water runoff and heavy rainfall may also 933 act as a triggering factor for the Åknes rockslide. Grøneng et al. (in 934 preparation) addresses the relation between displacement and pre-935 cipitation for the Åknes rockslide. 936

### 6.5. Uncertainties 937

938 The geological model presented is based on all observations described above. However, there are observations that are not in-939 cluded in the geological model, such as mapped layers of low re-940 sistivity in the 2D resistivity profiles measured above the back scarp. 941 942 This may be an indication of an unstable rock mass above the back scarp, in which case the back scarp is not the upper limit of the 943 rockslide area. No field observations indicate movement above the 944 back scarp, other than structures related and close to the back scarp. 945946 Several rockslides have occurred in the area to the west and east of 947 Åknes, resulting in rock avalanche deposits as large as 400 million m<sup>3</sup> covering the fjord bottom (Blikra et al., 2005b). This raises the 948 question of how stable the rock mass outside Åknes rockslide is. The 949 supposed lower limit of the rockslide is the toe zone. However, inter-950 ferometric sonar mapping of the mountainside below sea level reveals 951 952indications of a possible sliding surface daylighting at a depth of -20to -50 m (Longva et al., 2007). 953

The seismic profile presented here is based on only five shot points 954(Fig. 6C), which limits both the data resolution and the details mapped 955 in the section. Therefore, the details indicated in the interpretation of 956 the profile are debatable, especially with regards to the sub-vertical 957 structure indicated at ~300 m. However, the layering of the four layers 958 with different seismic velocities is likely, which indicate that the two 959 960 uppermost layers thins out and truncate at the same area as the 2D 961 resistivity profile indicate a daylighting low resistivity layer (Fig. 7).

There are also indications from the boreholes that there may be 962 deeper sliding surfaces at depths from 115 to 190 m. This is supported 963 by intensely fractured layers encountered in the lower parts of the 964 drill cores (115 m, Fig. 8) and indications of good hydraulic conduc-965 966 tivity at even greater depths (150-190 m). The latter was the likely 967 cause of loss of water during drilling. The loss of water can be explained in several ways; i) a perched groundwater level that was punctured by 968 969the drill holes, ii) a set of open fractures with good hydraulic communication, and/or (iii) a deep sliding surface in a weaker rock layer that 970 971 controls the groundwater flow at depth.

As indicated in the geological models presented in Fig. 10, bedrock 972 with higher fracture frequency than the host rock extends down to at 973 least 100 m. This suggests that there may be multiple sliding surfaces, 974 which is very often the case in deep-seated rock slides in crystalline 975 rock (Bonzanigo et al., 2000). This assumption is further confirmed by 976 the presence of heavily fractured rock at 115 m depth (Fig. 8), and is 977 alluded to in the discussion related to Fig. 10 (i.e. that the rockslide 978 may be deeper than the current geological model predicts). However, 979 980 at this early stage of the investigations no conclusive measurements

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# Paper IV: Numerical analysis of the 650,000m<sup>2</sup> Åknes rock slope based on measured displacements and geotechnical data

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# Paper V: Dynamic analysis of the 800m high Åknes rock slope using UDEC

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# Dynamic analysis of the 800m high Åknes rock slope using UDEC

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# Abstract

The seismic stability of the Åknes rock slope, Western Norway, was analysed by using the distinct element code UDEC. The slope poses a threat to the region as a sudden failure may cause a destructive tsunami in the fjord. The dynamic input was based on earthquakes with return periods of 100 and 1000 years, and in most models the input shear wave was a harmonic function (sine wave). Models with depths of the sliding surface up to 200 metres and with ground water conditions derived from site investigations were analysed, as well as models with ground water conditions assumed from possible future draining of the slope. Friction angles somewhat higher than the friction angles that were required for static stability were used in the dynamic analyses. The analyses indicate that an earthquake with a return period of 1000 years is likely to trigger sliding to great depth in the slope at the present ground water conditions and that the slope will be stable if it is drained. The analyses also indicate that sliding is not likely to be triggered by an earthquake with a return period of 100 years at the present ground water conditions.

# Keywords

Rock slope, dynamic analysis, earthquake, Newmark displacements, UDEC

# **1** Introduction

Earthquake is one of the main triggering factors for landslides. Investigations have been conducted on the types of ground failures induced by earthquakes, statistical distribution of landslides, the relations between the magnitude of earthquakes and epicentral distance to the fall or slide, areas affected by earthquakes of different magnitudes, the relation between earthquake magnitudes and volumes of landslides and the significance of slope angles [1 - 11]. The relations derived in the work mentioned above are useful for estimating landslide hazard on a regional scale. For seismic stability of specific slopes, the Newmark sliding block analysis [12, 13] is widely used. This method also represents a useful and practical tool for evaluation of earthquake-induced landslide hazard [14, 11].

The stability of rock slopes subjected to seismic effects can be analysed using appropriate numerical techniques. Several researchers have attempted using continuum (finite element) and discontinuum (distinct-element) techniques to study the behaviour of rock slopes subjected to dynamic loading. Some continuum codes can consider the effect of pseudo-static earthquake loading by applying a seismic body to each finite element in the model. Such approaches are useful for analysis of underground structures but are considered inadequate for dynamic analysis of rock slopes. In the pseudo-static analysis approach, the ground deformations and/or inertial forces are imposed as static loads and the rock-structure interaction does not include dynamic or wave propagation effects.

When the stability of the rock slope is controlled by movement of joint-bounded blocks and/or intact rock deformation, the use of discontinuum discrete-element codes, which allow one to conduct fully dynamic analyses under plane-strain conditions, are more appropriate than the continuum codes. Eberhardt and Stead [15] and Stead et al. [16] provide an example of dynamic distinct-element analysis of a natural rock slope in Western Canada. In that case, the dynamic input was introduced along the bottom boundary of the model as a harmonic stress function (sine wave) of a specified amplitude, frequency and duration. The model considered an initially stable slope subjected to an earthquake, resulting in yielding and tensile failure of intact rock at the slope's toe. Bhasin and Kaynia [17] illustrated the application of dynamic distinct-element analysis to a 700m high Norwegian rock slope to estimate the potential rock volumes associated with a potential catastrophic rock failure. Liu et al. [18] simulated the dynamic response of a jointed rock slope subject to effects of an explosion using the distinct element code UDEC [19]. Their results showed that the computed velocity history of the toe of the slope agrees well with that of observations. They concluded that UDEC can be used effectively to simulate the dynamic response of jointed rock slopes.

In this study the stability of the Åknes rock slope, Western Norway (Fig. 1), when subjected to earthquakes with return periods of 100 and 1000 years was investigated using UDEC. None of the documented historical (about year 1700 to present) large rock slides / rock avalanches in Norway have been triggered by earthquakes, with the possible exception of the Tjelle rock slide of  $15 \times 10^6$  m<sup>3</sup> which occurred in 1756. Rock falls were reported for the M5.8 Mo i Rana earthquake in 1819. For older events Blikra et al. [20] include the possibility that one or more large earthquake(s) may have caused some of the observed clusters of rock avalanches.

![](_page_85_Figure_3.jpeg)

Fig. 1. Map of Storfjorden showing the location of the Åknes rock slope and the site for earthquake hazard analysis.

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# 2 Description of the Åknes rock slope

Åknes is a rock slope on the Storfjord in Norway (Fig. 1). Massive slides have occurred in the region in historical times, e.g. the Loen and Tafjord disasters [21]. Bathymetric surveys of the fjord bottom deposits show that numerous and gigantic rockslides have occurred in the past [20]. About  $650,000 \text{m}^2$  of the Åknes rock slope is unstable (Fig. 2) and a sudden failure may trigger a destructive tsunami in the fjord [22]. The dip of the slope is about 35° between the top scarp/upper tension crack and the fjord. The rock mass consists of biotitic, granitic and dioritic gneisses. Three distinct fracture sets have been identified: fractures parallel to the foliation and sub-parallel to the slope surface (strike about E-W and dip to the south) and subvertical fractures with strike directions approximately N-S and E-W respectively [23]. Ganerød et al. [23] proposed a geological model for the rock slope, and details about the geology of the site are given in their article. Widening of the upper crack tension has been recognised since the late 1950s / early 1960s. Monitoring of the upper tension crack started in 1986 [24, 25]. Today, movements at the slope surface and movements in the sub-surface are monitored by various techniques as an early warning system has been implemented [26]. The instability is caused mainly by the orientation of the foliation of the gneisses, which is subparallel to the slope surface [27 - 28].

![](_page_86_Picture_3.jpeg)

Fig. 2. Overview of the Åknes rock slope. The white lines indicate the contour of the unstable area (slightly modified after [37]). The length of the "top scarp"/upper crack is about 800m. U, M, L: upper, middle and lower borehole sites. Black dotted line: Profile used in the numerical modelling.

# **3 Model description**

Kveldsvik et al. [22] performed static stability analyses on a number of models of the Åknes rock slope. They used the Universal Distinct Element Code (UDEC), and varied the fracture geometry, fracture friction and ground water conditions based on site specific data. One of the conclusions was that the static models that were unstable to great depths agreed better with shear strength parameters derived from an earlier study [28] than the models that were unstable to smaller depths. Later, after moving the installations for measuring displacements in the upper borehole to a greater depth interval (83 - 133m), displacements have been measured at depths 87m, 97m, 107m and 120m. This study considers two models in which the maximum depths of instability are respectively 110m and 200m.

Kveldsvik et al. [22] defined "limiting" friction angles for the various numerical models, i.e. the friction angles that were just large enough to result in equilibrium in the numerical model. In most models the friction angles were made dependent on the estimated effective normal stresses based on the Barton-Bandis shear strength criterion [29, 30]. Stress-dependent friction angles were used in all models in this study (Fig. 3). Details of the numerical models and the geotechnical data are given in [22] and [28].

The "limiting" friction angles defined above imply that the available friction is fully mobilised in the most critical part of the numerical models. Exposing the numerical models with "limiting" friction angles derived in Kveldsvik et al. [22] would result in sliding, even for very small earthquakes. The numerical models used in this study represent a rock slope where available friction is not fully mobilised and the static factor of safety is somewhat higher than 1. This was achieved by using higher frictions angles than the limiting values derived in [22]. The higher friction angles correspond to a higher Joint Roughness Coefficient (JRC). Fig. 3 shows that increasing JRC by 1 means increasing the friction angles with about 2°. For friction angles from 36° to 54° an increase by 2° means increasing the resisting force by about 7% (tan[ $\phi$ +2°]/tan[ $\phi$ ] ≈ 1.07 for  $\phi \in [36^\circ, 54^\circ]$ ). The friction angles that were actually used in the models are shown in Section 6.

In addition to higher friction angles, the bulk modulus and shear modulus of the rock mass were increased compared to the models presented in [22]. This was performed to fit the models to data on compression and shear wave propagation velocities [31] as described in Section 4.

Kuhlemeyer and Lysmer [32] showed that for accurate representation of wave transmission through a model, the spatial element size must be smaller than approximately one-tenth to one-eight of the wavelength associated with the highest frequency component of the input wave. The zoning of the models used in this study was adjusted to ensure that the spatial element size is smaller than one-tenth of the wave length associated with the sinusoidal input wave with the highest frequency.

Fracture geometry and ground water conditions are identical to the corresponding models in [22], and are shown in Fig. 4 which illustrates the models that were exposed to dynamic loading in the present study. The geometries were identical in all the computed models except that the inclination of the two outcropping fractures were either 20° or 0°. The maximum depth of the fractured area is 200m. Restricting the possible maximum depth of sliding to 110m was obtained by "gluing" the appropriate length of the inner part of the lower outcropping fracture by high shear resistance properties. Viscous (or absorbing/quiet)

![](_page_88_Figure_1.jpeg)

boundaries as shown in Fig. 4 were applied in all models to minimize reflections from outward propagating waves.

Fig. 3. Friction angle vs. depth at different JRC (Joint Roughness Coeffecient) for a fracture with dip 35°. The depth to ground water is 40m. After [22].

![](_page_88_Figure_4.jpeg)

Fig. 4. Block models that were subjected to dynamic loading. Monitoring points were used to track time histories of various parameters.

Draining the slope by a system of tunnels and boreholes is considered as possible mitigation for reducing the hazard posed by the Åknes rock slope, and planning work on this is in progress [33]. Investigation of the slope's capability of withstanding earthquakes after drainage was performed for two scenarios, 100% successful drainage, i.e. a dry slope, and 50% successful drainage. The latter was modelled with unchanged ground water table and water density reduced from 1000kg/m<sup>3</sup> to 500kg/m<sup>3</sup>, i. e. the water pressure was reduced by 50%. Both scenarios are likely to represent a simplification of the assumed effect of drainage in a fractured and inhomogeneous rock mass. To implement the effect of draining, the models were first computed to equilibrium with a ground water table as illustrated in Fig. 4 and a water density of 1000kg/m<sup>3</sup> (assumed present ground water conditions), then the changed/new ground water conditions implied that less of the available friction was mobilised compared to the previous state, implying that the static factor of safety was increased. These models were thereafter subjected to dynamic loading.

## **4** Dynamic input

Seismic hazard curves for a site near Åkneset (Fig. 5) were computed by NORSAR [34]. The peak ground accelerations  $(a_{max})$  of earthquakes with annual exceedance probabilities of  $10^{-2}$  and  $10^{-3}$  (hereafter referred to as the 100-year and the 1000-year earthquakes) were estimated to be  $0.3 \text{m/s}^2$  and  $0.83 \text{m/s}^2$  respectively.

![](_page_89_Figure_4.jpeg)

![](_page_89_Figure_5.jpeg)

A real acceleration time history of an earthquake consists of many cycles of motion and contains a wide spectrum of frequencies. To account for the variability of the earthquake motion, one should in principle use several acceleration time histories as input excitation in an earthquake analysis, and the selected time histories should be representative of the earthquake motions in the region which are usually defined in terms of design response spectra [17]. As the objective of this study was mainly to compare a number of slope models with respect to their capacity to withstand earthquakes, rather than a detailed earthquake investigation, the dynamic input was simplified to a sinusoidal, vertically-propagating shear wave in the models. In addition, one "real" representation of a time history representing 1000-year-earthquake (Fig. 6) were used as input in selected models for comparison with the results from the sinusoidal input. The time history from 6s to 28s was applied. These "real" time histories represented one of many possible representations of time histories for the 100-year and the 1000-year earthquake events [35].

![](_page_90_Figure_2.jpeg)

The dynamic variables for the input shear wave were calculated from Eq. 1 and 2:

$$V_{\max} = \frac{a_{\max}}{\omega} = \frac{a_{\max}}{2\pi f} \tag{1}$$

where  $a_{max}$ : peak ground acceleration;  $V_{max}$ : peak ground motion velocity; f: frequency;  $\omega$ : angular frequency.

$$\tau = C_s \times \rho \times V_{\text{max}} \tag{2}$$

where  $\tau$ : shear stress; C<sub>s</sub>: shear wave propagation velocity;  $\rho$ : rock density.

The shear stress calculated from Eq. 2 was doubled in order to compensate for the viscous boundary at the base of the model as the viscous traction is given by  $-\tau$ .

Three cycles of motion and frequencies of 5Hz and 10Hz were used to represent the 100-year earthquake, while five cycles of motion and frequencies of 3Hz and 6Hz were used to represent the 1000-year earthquake. These numbers define equivalent number of cycles with

maximum acceleration representing the earthquake motions with the magnitude expected in this region.

Shear and compression wave velocities have been measured in boreholes, and Poisson's ratio was calculated from Eq. 3 [31]. The measurements were performed by using a device where the two receivers were spaced 304mm and the distance to the transmitter below the receivers was 914mm. Measurements were taken for each 20cm while the device was moved upwards in the borehole.

$$\nu = \frac{C_c^2 - 2C_s^2}{2(C_c^2 - C_s^2)}$$
(3)

where v: Poisson's ratio; C<sub>c</sub>: compression wave velocity; C<sub>s</sub>: shear wave velocity.

The results from sonic logging are shown in Figs. 7-9. The mean value of the shear wave propagation velocity (2837 m/s, Fig. 7), together with the mean value of the rock density (2738 kg/m<sup>3</sup> as shown in [28]) and the ground motion velocity calculated from Eq. 1, were used to calculate the shear stress input. The different earthquake scenarios with corresponding input in UDEC are summarised in Table 1. A damping of 0.5 % was applied in all the computations.

![](_page_91_Figure_6.jpeg)

Fig. 7. Histogram of shear wave velocities, all boreholes combined. After [31].

![](_page_91_Figure_8.jpeg)

Fig. 9. Histogram of Poisson's ratios, all boreholes combined. After [31].

Return	$a_{max} (m/s^2)$	f (Hz)	V <sub>max</sub> (m/s)	τ (MPa)	2×τ (MPa)	No. of	Duration (s)
period	Fig. 5		Eq. 1	Eq. 2		cycles	
100 y	0.30	5	0.0095	0.074	0.148	3	0.60
100 y	0.30	10	0.0048	0.037	0.074	3	0.30
1000 y	0.83	3	0.0440	0.342	0.684	5	1.67
1000 y	0.83	6	0.0220	0.171	0.342	5	0.83
		UDEC			UDEC		UDEC input
		input			input		

Material properties for the blocks in UDEC were assigned values in accordance with the mean values of the wave propagation velocities measured in the three boreholes (Fig. 7 and 8) in order to obtain wave propagation upwards in the model that was in accordance with the dynamic input. The material properties of the blocks were calculated from Eq. 4 and 5:

$$G = C_s^2 \times \rho \tag{4}$$

where G: shear modulus of the rock.

$$K = C_c^2 \times \rho - \frac{4G}{3} \qquad (5)$$

where K: bulk modulus of the rock.

The calculated values were G = 22GPa and K = 45GPa.

## **5 UDEC analysis versus Newmark sliding block analysis**

Newmark [12] considered the simple model of a block on an inclined plane as an analogy for a slope subjected to earthquake loading. If the inertial forces acting on a potential failure mass become large enough, the static plus dynamic driving forces exceed the available resisting forces and the safety factor drops below 1.0. When the safety factor is less than unity, the potential failure mass will be accelerated by the unbalanced force. The situation is analogous to that of a block resting on an inclined plane [36]. Newmark [12] used this analogy to develop a method for prediction of the permanent displacement of a slope subjected to any ground motion (Fig. 10).

![](_page_92_Figure_9.jpeg)

Under static conditions the safety factor is calculated from:

$$SF_{stat} = \frac{W\cos\beta\tan\varphi}{W\sin\beta} = \frac{\tan\varphi}{\tan\beta}$$
(6)

where  $\varphi$ : friction angle between the block and the plane;  $\beta$ : inclination of the plane.

Horizontal vibration of the inclined plane with acceleration  $a_h(t) = k_h(t)g$  will transmit inertial forces to the block. Note that the effects of vertical accelerations have been neglected. At a particular instant in time, horizontal accelerations of the block will induce a horizontal inertial force  $k_hW$ . When the inertial force acts in the downslope direction (i.e. ground acceleration in upslope direction), resolving the forces perpendicular to the inclined plane yields:

$$SF_{dyn} = \frac{\left[\cos\beta - k_h(t)\sin\beta\right]\tan\varphi}{\sin\beta + k_h(t)\cos\beta}$$
(7)

The dynamic safety factor decreases as  $k_h$  increases and for a statically stable block there will be some value of  $k_h$  that will produce a safety factor of 1.0. This coefficient, termed the yield coefficient,  $k_y$ , corresponds to the yield acceleration,  $a_y = k_y g$ , which is the minimum pseudostatic acceleration required to produce instability of the block. For the block of Fig. 10  $k_y$  is for sliding in the downslope direction:

$$k_{y} = \tan(\varphi - \beta) \tag{8}$$

For sliding in the upslope direction:

$$k_{y} = \frac{\tan \varphi + \tan \beta}{1 + \tan \varphi \tan \beta}$$
(9)

When a block on an inclined plane is subjected to a pulse of acceleration that exceeds the yield acceleration, the block will move relative to the plane. The procedure by which the resulting permanent displacement can be calculated is illustrated in Fig. 11: The inclined plane is subjected to a single rectangular acceleration pulse of amplitude A and duration  $\Delta t$ , where A is larger than the yield acceleration,  $a_y$ . By integration the total relative displacement can be computed as shown in [36].

The total relative displacement depends strongly on both the amplitude and the length of time during which the yield acceleration is exceeded. This suggests that the relative displacement caused by a single pulse of strong ground motion should be related to both the amplitude and frequency content of that pulse. As an earthquake motion can exceed the yield acceleration a number of times, it can produce a number of increments of displacements (slips). Thus the total permanent displacement will be influenced by the duration of the strong motion as well as amplitude and frequency content. In the case of a periodic motion, the total displacement will simply be the displacement caused by one cycle of motion times the number of cycles of motions.

The total relative displacement was computed for the block shown in Fig. 10 for two cases of ground motions defined as a sinusoidal wave:

- 1. Maximum acceleration =  $5m/s^2$ , frequency = 2Hz, duration = 2s (4 cycles of motion) 2. Maximum acceleration =  $5m/s^2$ , frequency = 5Hz, duration = 0.8s (4 cycles of motion)

The yield coefficient,  $k_y$ , in the downslope direction is found from Eq. 8 and Fig. 10:  $k_y = tan(35^\circ - 20^\circ) = 0.268$  which gives the yield acceleration  $a_y = 0.268g = 2.63 \text{m/s}^2$ . The yield coefficient,  $k_y$ , in the upslope direction is calculated from Eq. 9:

 $k_y = \frac{\tan 35^\circ + \tan 20^\circ}{1 + \tan 35^\circ \tan 20^\circ} = 0.848$  which gives the yield acceleration  $a_y = 0.848 \text{ g} = 8.32 \text{ m/s}^2$ . The yield acceleration in the upslope direction is higher than the peak ground acceleration,

implying that relative displacements in the upward direction cannot occur. The results of the Newmark computations are illustrated in Fig. 12 and 13.

![](_page_94_Figure_7.jpeg)

Fig. 11. Variation of relative velocity and relative displacement between sliding block and plane due to rectangular pulse that exceeds yield acceleration between  $t = t_0$  and  $t = t_0 + \Delta t$ . From [36].

![](_page_94_Figure_9.jpeg)

![](_page_94_Figure_10.jpeg)

Fig. 12. Newmark computation of relative displacement and relative velocity for frequency equal to 2Hz. One cycle of motion.

Fig. 13. Newmark computation of relative displacement and relative velocity for frequency equal to 5Hz. One cycle of motion.

In UDEC the dynamic input was applied as velocity at the base of the large lower block (Fig. 10) for the two cases as  $V_{max} = 0.398$ m/s (corresponding to f = 2Hz, Eq. 1) and  $V_{max} = 0.159$ m/s (corresponding to f = 5Hz). Monitoring points were placed both in the large block and in the small upper (Newmark) block to monitor the behaviour of the model. The bulk modulus of the blocks was set to 40GPa and the shear modulus was set to 17GPa. The normal stiffness and the shear stiffness of the boundary (joint) between the two blocks were both set to 100GPa/m. The high stiffness values were used in UDEC as the Newmark analysis is based on infinite stiffness (rigid-plastic behaviour). Some damping was necessary for the model to avoid numerical instability, and the results reported below were computed with 0.01 % damping. Damping up to 1 % was tested with a negligible effect on the total relative displacement. The results of the computations are shown in Table 2.

Table 2. Total relative displacement after 4 cycles of motion computed from Newmark analysis and UDEC analysis

Frequency	Total relative displacement					
	Newmark	UDEC: +xvel <sup>a</sup>	UDEC: -xvel <sup>a</sup>			
2Hz	14.3cm	16.8cm	14.8cm			
5Hz	2.3cm	2.7cm	2.3cm			

 $a^{+}$  +xvel means that the first pulse acts in the downslope direction and -xvel means that the first pulse acts in the upslope direction.

It can be concluded that the differences in relative displacements computed by the two methods are small. Computations were also performed for a smaller friction angle (and smaller yield acceleration) with similar agreement between UDEC and Newmark computations. Time histories of monitoring points in UDEC are shown i Figs. 14 - 17.

![](_page_95_Figure_6.jpeg)

Fig. 14. UDEC. X-velocity of the large block vs. time. Damping 0.01 %.

![](_page_95_Figure_8.jpeg)

Fig. 15. UDEC. X-velocity of the sliding block vs. time. Damping 0.01 %.

![](_page_96_Figure_1.jpeg)

![](_page_96_Figure_2.jpeg)

Fig. 16. UDEC. Shear displacement vs. time at the boundary between the sliding block and the large block (relative displacement of the sliding block to the large block). Damping 0.01 %.

Fig. 17. UDEC. X-displacement of the sliding block vs. time (absolute displacement of the sliding block). Damping 0.01 %.

# 6 Results for the Åknes rock slope

The results of the computations using UDEC are summarised in Table 3. JRC was increased by 0.5 or 1.0 from one computation to the next when the smallest JRC that would result in a stable model was to be defined.

Model:	Earthquake: increasing peak ground motion velocity $\rightarrow$					
max. depth of possible	100y-10Hz <sup>b</sup>	100у-5Нz <sup>в</sup>	1000y-6Hz <sup>c</sup>	1000y-3Hz <sup>c</sup>		
sliding-inclination of						
outcropping fractures-						
"limiting JRC from static						
modelling" <sup>a</sup> -other info.						
200-20-"JRC8"	JRC=8.5	JRC=9	JRC>11 <sup>d</sup>	JRC>11 <sup>d</sup>		
	sd=0.4cm	sd=0.8cm				
110-20-"JRC7"	JRC=8.5	JRC=9	JRC>11	JRC>11		
	sd=0.3cm	sd=3.1cm				
200-20-"JRC8"-drained 100				JRC=8		
%				sd=9.2cm		
200-20-"JRC8"-drained 50 %			JRC=8	JRC=9		
			sd=2.0cm	sd=11.1cm		
200-0- "JRC6"		JRC=6.5	JRC=8	JRC=8		
		sd=3.8cm	sd=5.8cm	sd=590cm <sup>e</sup>		
200-0-"JRC6"-drained 100 %				JRC=6		
200-0-"JRC6"-drained 50 %				JRC=6		
				sd=12.0cm		

Table 3. The smallest JRC reguired to withstand earthquakes of various return periods and frequencies and the corresponding maximum shear displacement (sd) at the lower outcropping fracture.

<sup>a</sup> For the drained models the "limiting" JRC refers to that obtained <u>before</u> draining the model.

<sup>b</sup> Earthquake of return period 100 years, frequency 10 or 5Hz and three cycles of motion.

<sup>c</sup> Earthquake of return period 1000 years, frequency 6 or 3Hz and five cycles of motion.

<sup>d</sup> JRC up to 11 was used in Model 200-20-JRC8, and a JRC of 11 resulted in sliding when the model was subjected to an 1000-year earthquake of frequency 6Hz.

<sup>e</sup> The maximum shear displacement at the lower outcropping fracture was 5.9m at 13.6 seconds and no additional shear displacement occurred during continuing computation, i.e. the model was stable. Whether such a large shear displacement going on for 13.6 seconds without continuing sliding is realistic in the real slope is however, questionable.

Model 200-20-"JRC8" with a JRC of 9 was also subjected to a "real" 100-year earthquake. The model was stable, and the maximum shear displacement at the lower outcropping fracture was 0.7cm larger than for the same model subjected to a sinusoidal representation of the 100-year-earthquake with a frequency of 5Hz. Model 200-20-"JRC8"-drained 100 % with a JRC of 8 was subjected to a "real" 1000-year earthquake. The model was stable and the maximum shear displacement at the lower outcropping fracture was 0.1cm smaller than for the same model subjected to a sinusoidal representation of the 100-year-earthquake with a frequency of 6Hz.

The results of computations for a stable model and for a model where sliding occurred are illustrated in Figs. 18 - 25 by the same model subjected to two different earthquakes: Model 200-20-"JRC8" with a JRC of 9 which was stable when subjected to a 100-year earthquake with a frequency of 5Hz and a duration of 0.6s but sliding occurred when the model was subjected to a 1000-year earthquake with a frequency of 3Hz and a duration of 1.67s.

The maximum shear displacement caused by the earthquake at the lower outcropping fracture in the stable model was 0.8cm. From Fig. 20 it can be seen that the shear displacements occurred both in the positive and negative x-direction during the duration of the earthquake as the magnitude of the shear displacements both increases and decreases, meaning that the rock mass above the lower outcropping fracture also moved to the left (upslope) relative to the rock mass below the outcropping fracture. This behaviour also took place in the model subjected to the 1000-year earthquake but it is not visible in Fig. 21 due to the much larger scale of the y-axis.

The displacements in the x-direction increased towards the top of the model in which sliding occurred (Fig. 19 and 23). This shows that shear displacements (relative displacements between fractures) occurred along all fractures. The total absolute displacement of the lowest block on the upper outcropping fracture was 30.1m after 8s. The lowest block on the lowest outcropping fracture displaced 15.6m during the same period. The reason for this is that the top layer (the upper 10m of the slope sub-surface) above the upper outcropping fracture moved much faster than the rest of the rock mass. The behaviour described above was seen in all versions of Model 200-20 in which sliding occurred.

Fig. 24 shows the particle velocity in the un-fractured area of the model. The three cycles of motion of the 100-year earthquake did not appear simultaneously at all the monitoring points as their distances from the base of the model varied (Fig. 4). It is evident from Fig. 24 that the wave transmission in the model was satisfactory as the wave maintained its input shape and the amplitudes were in accordance with the expected value of 0.0095m/s. The low amplitude waves that is observed after 1.1s, i.e. after the exitation was stopped, was caused by wave reflections. The large amplitude waves observed after the 1000-year earthquake of five cycles of motion and duration 1.67s (Fig. 25) were caused not only by reflected input waves, but also by all the blocks that moved downslope during the entire period of computation. The amplitudes of the input generated wave were as expected, near 0.044m/s.

![](_page_98_Figure_1.jpeg)

Fig. 18. Stable model. Block deformation magnified by 1000.

![](_page_98_Figure_3.jpeg)

![](_page_98_Figure_4.jpeg)

Fig. 19. Model with sliding. Block deformation magnified by 10.

![](_page_98_Figure_6.jpeg)

Fig. 20. Stable model. Shear displacement (m) at the lower outcropping fracture vs. time (s). The depth of the monitoring points increases with increasing numbers. The duration of the earthquake is 0.6sec.

![](_page_98_Figure_8.jpeg)

Fig. 21. Model with sliding. Shear displacement (m) at the lower outcropping fracture vs. time (s). The depth of the monitoring points increases with increasing numbers. The duration of the earthquake is 1.67sec.

![](_page_98_Figure_10.jpeg)

Fig. 22. Stable model. X-displacements (absolute displacements in the X-direction in m) at the upper array of monitoring points (Fig. 4) vs. time (s). The depth of the monitoring points increases with increasing numbers. The duration of the earthquake is 0.6sec.

Fig. 23. Model with sliding. X-displacements (absolute displacements in the X-direction in m) at the upper array (Fig. 4) of monitoring points vs. time (s). The depth of the monitoring points increases with increasing numbers. The duration of the earthquake is 1.67sec.

![](_page_99_Figure_1.jpeg)

Fig. 24. Stable model. X-velocities (m/s) at the montoring points in the unfractured area vs. time (s). The duration of the earthquake is 0.6sec.

![](_page_99_Figure_3.jpeg)

Fig. 25. Model with sliding. X-velocities (m/s) at the montoring points in the unfractured area vs. time (s). The duration of the earthquake is 1.67sec.

# 7 Discussion

The sinusoidal input shear wave (see example in Fig. 24) applied in most of the models represented a simplification of a real earthquake record, and uncertainties are associated with the representativeness of the sinusoidal input. The two models that were computed with "real" representations of acceleration time history show displacements of similar magnitude as the sinusoidal input in the same models. This indicates that use of the sinusoidal input wave was a satisfactory approach. Application of "real" representations of time histories would also imply uncertainties, which could be investigated by computing a large number of "real" time histories. A disadvantage by using "real" time histories is the considerable increase in computation time for time histories of long duration.

The compression wave was omitted for two reasons: (i) The effect of including the compression wave would depend on the phase of the compression wave relative to the phase of the shear wave as it would have positive effect on stability if normal stresses on fractures increases when the shear wave acts in the downslope direction and a negative effect if normal stress on fractures decreases when the shear wave acts in the downslope direction. (ii) The estimation of the phases of the waves relative to each other would be incidental.

Possible shear strength degradation resulting from earthquake-induced displacements was not modelled. Thus, the models capability of withstanding earthquakes may have been overestimated. However, in most of the models the earthquake-induced shear displacements along fractures were smaller than about 12cm. Since 1961, displacements on the slope surface larger than 10cm/year has been documented. Since 1993, when continuous monitoring of the upper tension crack started, the widening of the tension crack at top of the slope has been steady at about 2cm/year [22, 25]. These measured displacements show that the earthquake-induced shear displacements computed in most of the stable models are small compared to the displacements experienced by the slope already and, thus, may not be critical for the stability of the slope.

# 8 Conclusions

The numerical model with maximum possible depth of sliding equal to 200m, outcropping fractures inclined 20° (Model 200-20-"JRC8") and friction angles about 1° larger (JRC=8.5) than the "limiting" friction angles required for stability in the static model was stable when exposed for a 100-year earthquake of frequency 10Hz, i.e. the least severe of the two scenarios of a 100-year-earthquake. With friction angles about 2° larger (JRC=9) than the "limiting" friction angles Model 200-20-"JRC8" was also stable when exposed for the most severe of the 100-year earthquakes. However, even with friction angles as high as about 6° larger (JRC=11) than the "limiting" friction angles, sliding was triggered in Model 200-20-"JRC8" by the least severe of the 1000-year earthquake, i.e. a frequency of 6Hz.

Model 110-20-"JRC7" required friction angles of equal magnitude as Model 200-20-"JRC8" to withstand earthquakes. This is as espected as the sliding caused by earthquakes in Model 200-20-"JRC8" occurred at all depths (see also Fig. 19 and 23). As the "limiting" friction angles from the static modelling was about 2° smaller for Model 110-20-"JRC7" than for Model 200-20-"JRC8" this means that Model 110-20-"JRC7" required a higher static factor of safety than Model 200-20-"JRC8" to withstand the the same dynamic impact. This is interpreted as Model 110-20-"JRC7" showed less capacity than Model 200-20-"JRC8" to withstand earthquakes.

Model 200-0- "JRC6" showed a larger capacity to withstand earthquakes than Model 200-20-"JRC8" as sliding was not triggered by a 1000-year earthquake with a frequency of 6Hz at friction angles about 4° larger (JRC=8) than the "limiting" friction angles meaning that a smaller static factor of safety was required in Model 200-0- "JRC6" than in Model 200-20-"JRC8" to withstand the same dynamic impact. This indicates that the Åknes rock slope is less vulnerable to earthquakes if the outcropping fractures in the slope are less steep than the assumed maximum inclination of 20°.

Model 200-20 with no water pressure in the fractures, i.e. 100 % successful drainage, withstood the most severe scenario of a 1000-year earthquake (f = 3Hz) at the friction angles equal to the "limiting" friction angles and the 50 % successful drainage variant of the same model withstood the least severe scenario of a 1000-year earthquake. Model 200-20 with friction angles about 2° larger that the "limiting" friction angles and 50 % successful drainage withstood the most severe scenario of a 1000-year-earthquake. Model 200-0 with friction angles equal to the "limiting" friction angles and 50 % successful drainage withstood the most severe scenario of a 1000-year-earthquake. Model 200-0 with friction angles equal to the "limiting" friction angles and 50 % successful drainage withstood the most severe scenario of a 1000-year earthquake. Model 200-0 with friction angles equal to the "limiting" friction angles and 50 % successful drainage withstood the most severe scenario of a 1000-year earthquake. Model 200-0 with friction angles equal to the "limiting" friction angles and 50 % successful drainage withstood the most severe scenario of a 1000-year earthquake. These results certainly indicate that draining the Åknes rock slope would be a very effective mitigation measure and that the drained slope will have a good chance of withstanding even a very severe earthquake for this region.

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