Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0290

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

The cause of the landslides of 1st and 2nd April 2016 in Tosbotn, Norway

La cause des glissements de 2016 à Tosbotn, Norvège

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ABSTRACT: On April 1st 2016 a 50 000 m^3 landslide, partly in clay, took a coastline road in Nordland, Norway. While geotechnical engineers and construction workers focused on reopening the road, a much larger slide of about 130 000 m^3 took place the following day, forcing the road to remain closed for 3 months. No lives were lost and no persons were injured by the slides, but the road and three houses were destroyed. The two landslides were very close but not in physically connected. Sensitive clay was involved. Shortly before the slides a water filling took place in a new hydropower tunnel in the mountainside behind the landslides. An investigation committee consisting of the authors of this paper was appointed by the local government in Nordland to identify the technical cause of the slides. The committee's final conclusion was that the landslides most likely were initiated by the increased pore-pressures in the soil after hydraulic fracturing and leakage from the unlined pressurized head race tunnel for the hydropower plant. The paper presents geological and geotechnical conditions, measured rock stresses around the hydropower tunnel, soil properties and results from stability analyses.

RÉSUMÉ: Le 1er avril 2016, un glissement de terrain d'environ 50 000 m^3, partiellement en argile, a détrui une route côtière dans le Nordland, en Norvège. Alors que des géotechniciens et des ouvriers se concentraient sur la réouverture de la route, un 2e glissement d'environ 130 000 m^3 à eu lieu le lendemain, forçant la route à rester fermée pendant 3 mois. La route ainsi que trois maisons ont été complètement détruites par les glissement, mais heureusement personne n'a été blessé. Les deux glissements étaient très proches mais pas en contact physique. Le sol à cet endroit contient de l'argile molle et sensible. Une commission d'enquête a été établi par le gouvernement local du Nordland afin d'identifier la cause technique des glissements. La conclusion finale du comité était que les glissements étaient probablement dus à une augmentation de la pression interstitielle dans le sol après la fracturation hydraulique d'un tunnel d'eau non revêtu pour une centrale hydroélectrique récemment construite tout prêt du site. Ce papier présente les conditions géologiques et géotechniques, les contraintes mesurées dans le roc autour du tunnel hydroélectrique, les propriétés du sol et les résultats d'analyses de stabilité.

Keywords: Landslide; sensitive clay; hydropower tunnel; hydraulic fracturing; excess pore pressures

1 INTRODUCTION

Landslides occur frequently along the Norwegian fjords. Such natural hazard poses a threat to coastal communities and infrastructure often lying at the foot of steep mountains in areas with soft fjord marine deposits. Sloping terrain and high groundwater level are also typical in such areas. The causes of slope failure in soft marine deposits are not always easy to pinpoint; however, such knowledge is essential in order to prevent future events, to learn from the incident and to insure safe developments in such areas.

Following the Tosbotn landslides of April 2016 the local government in Nordland County, Norway, appointed the authors of this paper as an investigation committee. The present paper gives an overview of the two landslides and concludes on the most likely failure mechanisms based on an integrated geological and geotechnical study.

2 THE LANDSLIDES

Tosbotn is situated in Nordland County, Norway, approximately 250 km north of Trondheim. The first landslide in Tosbotn took part of the coastal road "FV 76" in the early morning on April 1st 2016. The road is the only ferry free connection to the city of Brønnøysund. The following day, while geotechnical engineers and construction workers focused on reopening the road, a much larger landslide of about 130 000 m^3 occurred, forcing the road to remain closed for 3 months. No lives were lost and no persons were injured by the landslides, but the road and three houses were destroyed. The two landslides were very close, but not in physical contact, Figure 1. As seen on the photo, the sliding surface for the landslides seems to be deep and rotational movement was involved.

A report (Multiconsult 2016), related to the immediate work of securing the site, describes the course of events on the 1st and 2nd April and suggests possible triggers for the landslides:

Figure 1. Overview. The first slide, Bjørnstokkvika and the power plant in the back, the second slide Bekkevold, in the front. Foto: Jørn Horn.

i) construction activity including vibrations from for instance blasting, ii) heavy traffic, iii) effect of rain or snowmelt, iv) high voltage cable and v) newly established ditch in the area. In addition leakage from the hydropower tunnel of the Bjørnstokk Power Plant was brought into attention. The course of events was further described by Lissman, (Lissman 2016).

2.1 Background

In 2017 the local government in Nordland County appointed the authors of this paper as an investigation committee to assess the failure

Figure 2. Aerial view of Tosbotn, the sites of the slides and the relevant hydropower tunnel.

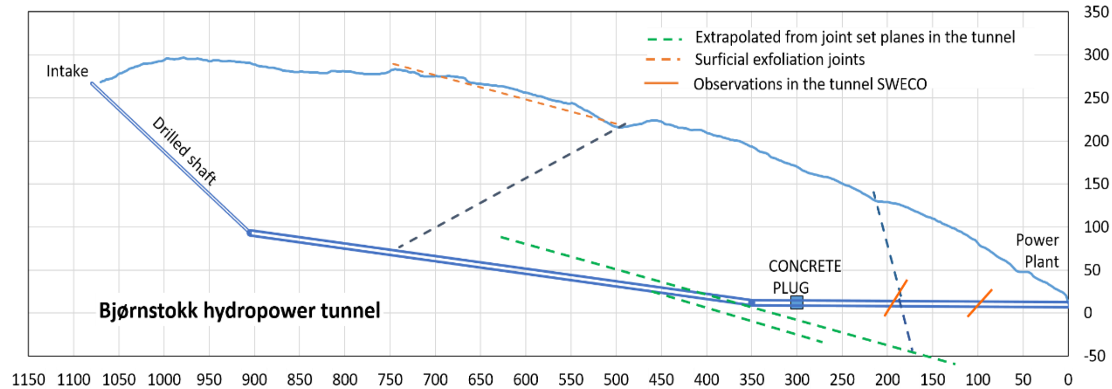

Figure 3. Verical section showing the Bjørnstokk hydropower tunnel and rock joints.

mechanism and the causes of the Tobotn landslides. The committee got access to detailed documentation from the Norwegian Public Roads Administration (NPRA) regarding the road in the area, and from Helgeland Kraft AS regarding this company's design and construction of a new hydropower plant in the mountain side behind the slides. The Bjørnstokk Power Plant is a small size plant of 8 MW and takes its water from a river at about 260 meter above sea level through a partly unlined headrace tunnel in rock, down to a turbine at elevation 11 meter. The turbine house is shown at the far back in Figure 1. NPRA performed new soil investigations in the weeks after the landslides in order to redesign and re-establish the road. The committee visited the site and the hydropower plant and made interviews with key persons involved.

3 THE HYDROPOWER TUNNEL

Figure 2 shows an aerial view of the position of the hydropower tunnel relative to the slides. Figure 3 shows a vertical cut through the waterway to Bjørnstokk Power Plant. The waterway starts with a river intake at elevation 267 meter from which the water goes through a 250 meter drilled shaft, inclined 45^0 downward, where after the waterway continues in a wider, unlined, blasted 600 meter long rock tunnel sloping 8^0 downward

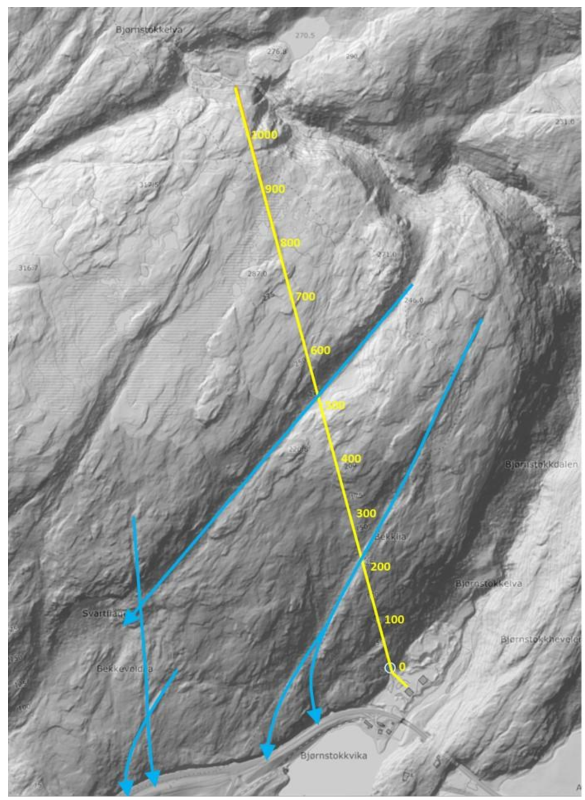

Figure 4. The tunnel (numbered) and possible paths for flow of water (arrows) in rock joints. The Bjørnstokk power plant and slide area at the bottom.

before the blasted tunnel is closed off by a 10 meter long, solid concrete plug.

A cast iron pipe leads the water though the concrete plug and takes it the last 330 meter through an access tunnel to the turbines in the power plant. The design saves the cost of lining the full waterway, but it relies on sufficiently high rock stresses to prevent hydraulic fracturing due to the high water pressure in the tunnel upstream the concrete plug. 100 years of experience in Norway related to such a design, proves that the method works well as long as the minor principal stress in the rock is sufficiently large.

3.1 Hydraulic fracturing

Unfortunately, at Bjørnstokk it became apparent too late that the minor principal rock stress was too low. This caused leakage from the tunnel after hydraulic fracturing when the waterway was filled in order to start production. Ground noise and vibrations felt by people in the area and also recorded by earthquake stations are most likely related to the fracturing. The original waterway design was therefore, after the landslides, redesigned by moving the concrete plug and extending the cast iron pipe 300 meter further into the tunnel. The tunnel was originally designed based on empirically based criteria without measurements of rock stresses. This is not uncommon for small-scale power plants in Norway (Broch 2000).

In the aftermath of the landslides the water tunnel was emptied and new cracks were found in the tunnel wall upstream the concrete plug (Sweco 2016). Measurements of rock stresses were made by both hydraulic fracturing tests in 30 meter long and 64 mm diameter boreholes in the tunnel wall (SINTEF 2016) and overcoring (SMCOY2016). This confirmed that hydraulic fracturing had taken place. The investigation committee studied the rock stress measurements and concluded that the water pressure was higher than the minor principal stress over a length of 200 meters upstream the original concrete plug.

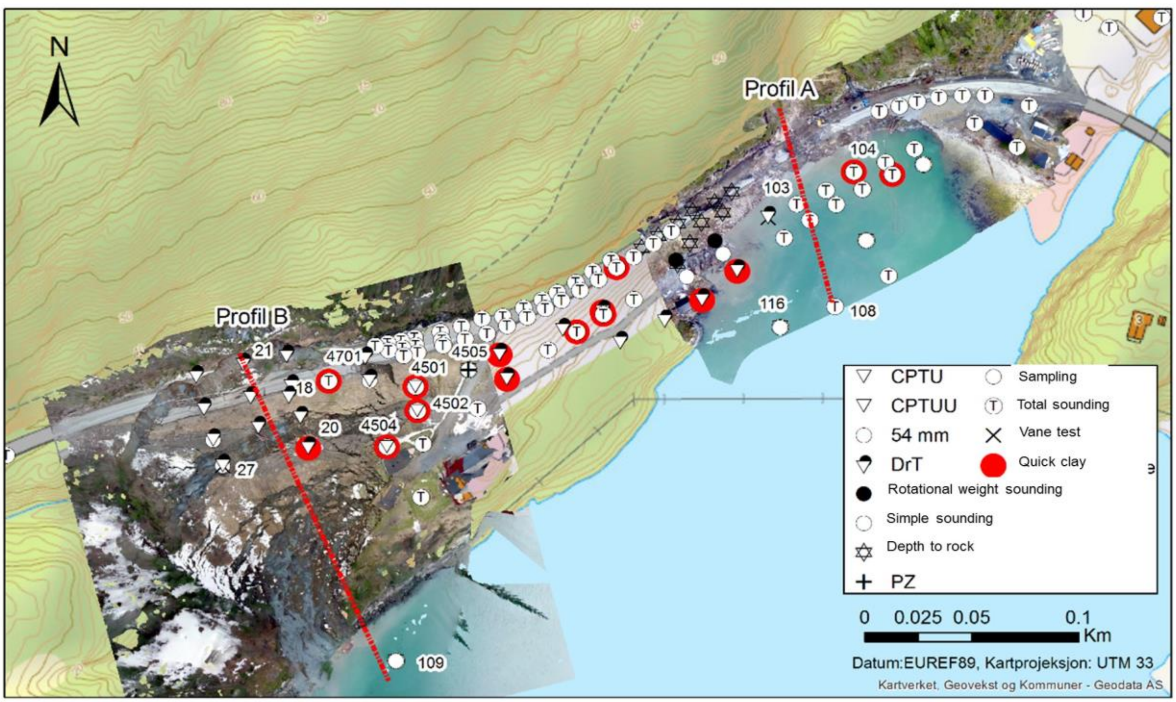

Figure 5. Soil investigations. Total Sounding to bedrock, CPTU, sampling and old vane tests were interpreted to show 4 layers with some road fill material on top, then a silty/sandy layer (Bekkevold only) over the dominating layer of silty clay, partly quick in some locations (red dots), over a gravelly/sandy layer.

Hydraulic fracturing then opened cracks in the rock around the tunnel and sent water through existing rock joints down and outwards in the direction of the fjord and the landslide areas.

Since the fine grained soil in the area has low permeability it acts as a plug preventing free flow from the rock and excess pore pressures in the soils will result. A vital question is how much excess pore pressures the slopes could tolerate. This is studied in the following.

4 SOIL CONDITIONS

Both landslide areas are dominated by a layer of silty clay which is sensitive and partly quick. Local variations are seen by lenses of silt and sand. Most of the soil in Bjørnstokkvika (slide 1) slid into the sea and at Bekkevold (slide 2) the remaining mass is heavily disturbed. Thus it is hard to get the proper soil parameters. They have primarily been estimated based on CPTU, performed in undisturbed soil close to the landslide scars, Table 1. Friction parameters are estimated based on the NTNU method, (Sandven 1990), Table 1. Undrained strengths are estimated based on Karlsrud et al (2005), and active strength is taken to be 25 + 3·z [kPa], where z is depth in meters from terrain. The plasticity index is Ip ≤ 10 (%). Anisotropic undrained strength is used with anisotropy factors cuD/cuA = 0,63 and cuE/cuA = 0,35, see [51]. The unit weight varies between 19 and 19,7 kN/m^3 in the layers.

Table 1. Friction and cohesion for simulations

Soil Layer	friction	cohesion [kPa]
(1) Road fill	42^0	0
(2) Sand/silt	33^0	2
(3) Silty clay	30^0	4
(4) Sand gravel lenses	33^0	2

5 SLOPE STABILITY CALCULATIONS

The stability was evaluated using a typical cross section in the middle of each slide. Three dimensional effects were considered to be very limited and thus conventional plane strain analyses are applied. Two different calculation tools were applied, Slope/W (GeoStudio) and Plaxis 2D, for internal control and as a sensitivity check. The results are quite similar.

5.1 Stability of the original slopes

The initial stability of the slopes were first analysed for the normal situation before the slide, i.e. without any extraordinary loads or porepressures. Drained analyses are considered

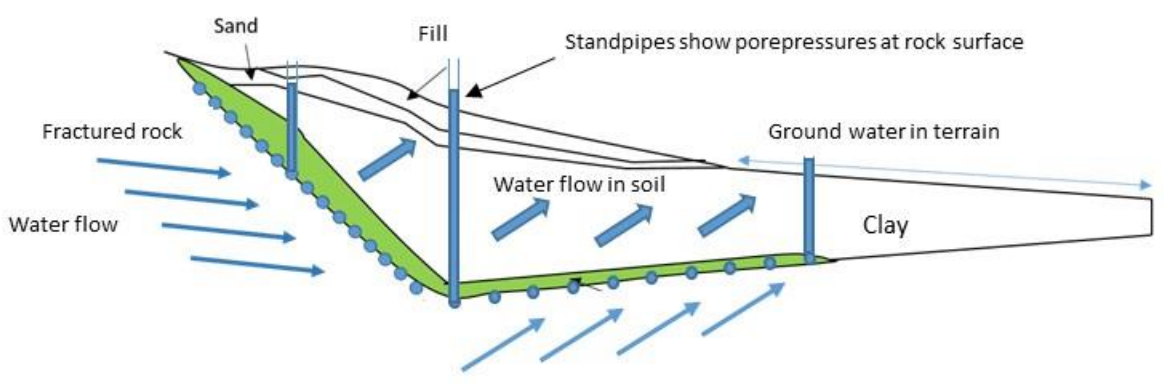

Figure 6. Illustration of how the leakage of water from the hydropower tunnel may create excess porepressures in the coarser soils along the rock surface at Bekkevold. Standpipes illustrate the excess pressures. In principle the same concept can be used for Bjørnstokkvika.

to be most relevant for such a situation. The undrained analyses intends to check the safety of the slopes with respect to rapid loads or rapid stress redistribution. Only the clay layer (3) is considered undrained in the "undrained" analyses since the other layers may drain much faster.

Table 2. Safety factors of the original slopes

Slope	Drained	Undrained
Bjørnstokkvika	1,12 – 1,17	1,10 – 1,17
Bekkevold	1,46 – 1,57	1,05 – 1,14

The results of the stability calculations are shown in Table 2. Intervals are indicated for safety

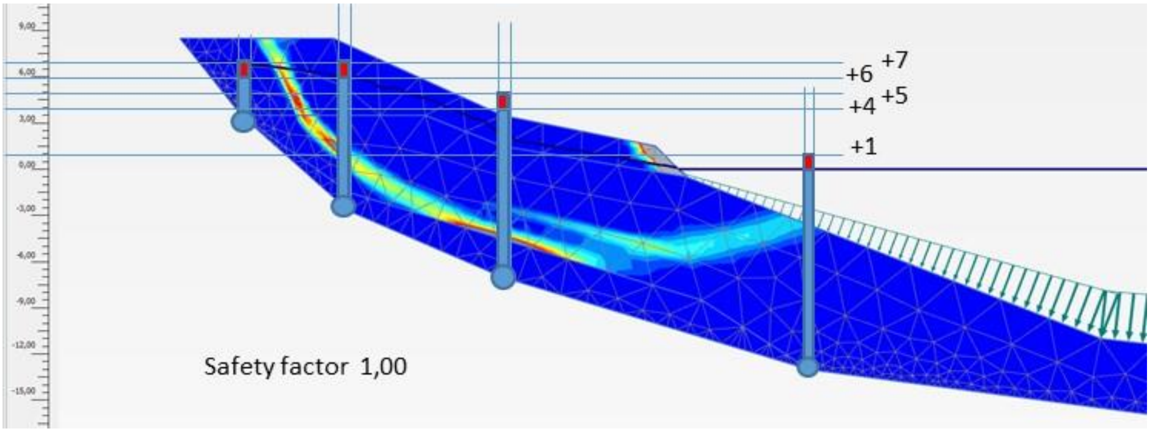

Figure 7. The slope at Bjørnstokkvika failed after an increase in porepressures by less than 1 meter in drained soil layers under and behind the slope. The results are very similar independent on whether the clay is considered drained or undrained. The red colour in the standpipes indicate the porepressure increase leading to failure. Results from Plaxis.

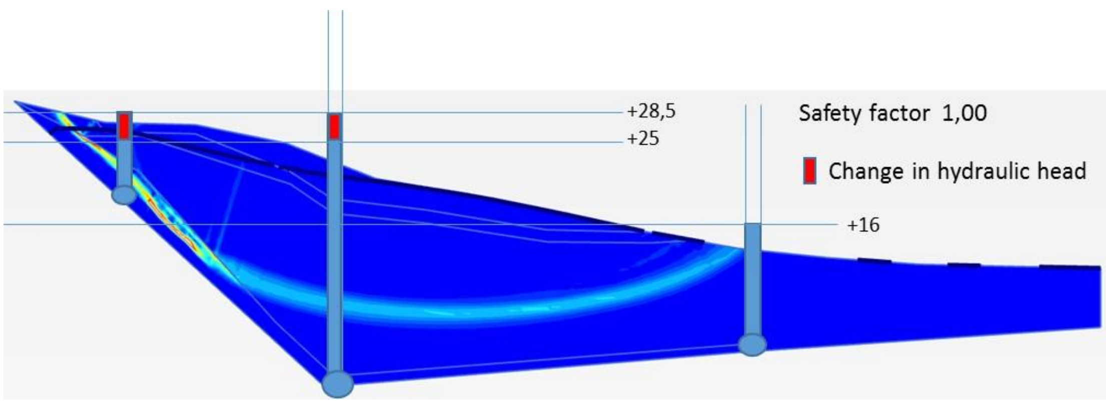

Figure 8. The slope at Bekkevold failed after an increase in porepressures by less than 3,5 meters in drained soil layers under and behind the slope. The clay is considered undrained due to rapid stress redistribution. The red colour in the standpipes indicate the porepressure increase leading to failure. Results from Plaxis.

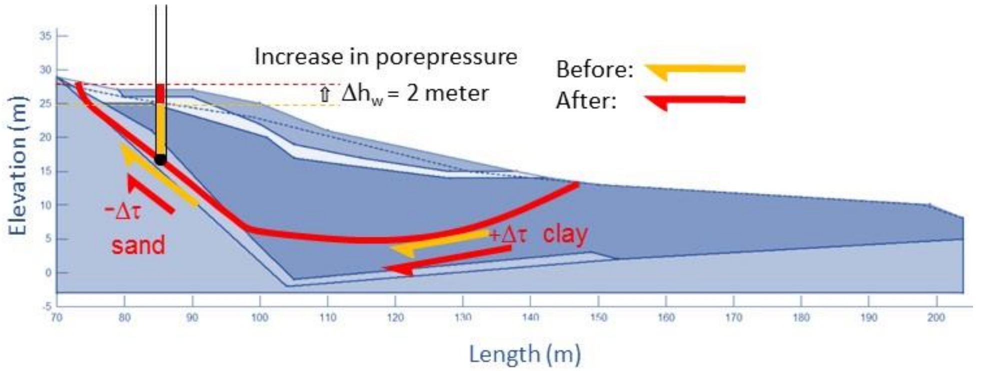

Figure 9. Illustration of stress redistribution due to a porepressure increase. The porepressure increase reduces the shear capacity and reduces the shear stresses in sand layers transferring undrained loading onto the clay.

factors and show the variation found in different analyses. It is observed that while the safety factors are generally low in Bjørnstokkvika, the drained anaysis show good stability at Bekkevold. The undrained analysis at Bekkevold, however, shows very low robustness against anything that might involve undrained stress changes. Figure 9 illustrates how an undrained stess change may occur due to a porepressure increase in the more sandy layers close to the rock surface.

5.2 Reduced stability and failure

A series of slope stability simulations were performed to assess the influence of potential triggering mechanisms. Heavy traffic load one day before the events was evaluated in undrained situations. The load had very little effect on the safety factor, (less than 1%), in part since the most critical surfaces did not extend sufficiently far to include the load (i.e. not critical), but also at Bekkevold because the weight of the soil mass is so large that the heavy vehicle load is insignificant compared to the weight of the soil. The effect of a ditch for a high voltage cable was investigated in the same manner and found to have no influence on stability. The potential for water infiltration through the ditch was also found to be low.

Analyses where performed to simulate the possible effect of excess pore-water pressure deep down under and behind the slopes. The simulations follow the idea sketched in Figure 6. In Bjørnstokkvika a pore pressure increase of 1 meter is enough to initiate failure. At Bekkevold a pore pressure increase of 3,5 meter is necessary to initiate failure. The theoretically predicted mechanism in Figure 7 and 8 fits well with observations from the actual slides.

6 CONCLUSIONS

Based on geological assessment, geotechnical data and stability analysis, a range of potential triggering mechanisms for the landslides at Tosbotn in 2016 were evaluated in this study. Blasting and construction work were ruled out since no significant construction work took place at the time in the area.
The impact of heavy traffic load on slope stability and of a small ditch for a power cable were both found to be insignificant.

The fact that the first landslide happened the very first day after the hydropower tunnel was water filled and kept full by water supply from the reservoir is a strong indication of a connection between the filling of the hydropower tunnel and the landslides.

A geological and structural survey of the tunnel showed that hydraulic fracturing took place during water filling of the tunnel at Bjørnstokk Power Plant. This is further supported by records of ground shaking registered on seismic stations in the surroundings. The hydraulic fracturing within the tunnel lead to significant water leakage and explains the lowering of water level at the intake of the tunnel at Bjørnstokk.

A study of joints and fault zones in the rock indicate that very likely water paths leads towards both slide areas. Considering that the water pressure behind the concrete plug is 2600 kPa (a head of 260 meter) and with a rather impervious clay clogging the outlets, it is not unlikely to see several meters increase in water pressure under and behind the slide areas.

Stability analyses show that a modest increase in pore pressure at larger depth in the slopes was necessary to initiate failure. Such pore pressure increase could not be attributed to rainfall and/or snow melting in the area.

Hence, the landslides most likely were initiated by increased pore-pressures in the soil after hydraulic fracturing and leakage of an unlined water tunnel for a hydropower plant recently constructed in the mountainside behind the slides.

The committe recommends that measurement of rock stresses should be made mandatory in rules and regulations for future design of pressurized hydropower tunnels in similar projects.

7 ACKNOWLEDGEMENTS

This work was made possible by the Local Government of Nordland County appointing the investigation committee. The information provided by Helgeland Kraft AS and the NPRA is highly appreciated, as well as their approval of publishing these findings and the lessons learned.

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