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SAFELAND – ET EU FINANSIERT FORSKNINGSPROSJEKT OM EVALUERING AV SKREDRISIKO

Changing pattern of landslide risk in Europe – the SafeLand project

Bjørn Kalsnes, avdelingsleder Løsmasseskred og risiko, NGI/nestleder ICG
Farrokh Nadim, direktør ICG

SAMMENDRAG

SafeLand var et 3-årig forskningsprosjekt under EU's 7. rammeprogram. SafeLand utviklet metoder for kvantitativ risikovurdering og -håndtering og strategier for skred på lokalt og regionalt nivå i Europa. Med det sørget prosjektet for å forbedre vår evne til å forutsi fare for skred, kartlegge risikosoner, og sørge for egnede prosedyrer for å håndtere skredrisiko. Effekt av klima- og demografiske endringer var en sentral del av prosjektet.

Det vitenskapelige arbeidet var organisert i fem områder:

- Area 1 fokuserte på å forbedre kunnskapen om utløsning av skred
- Area 2 hadde som mål å harmonisere metoder for kvantitativ risikovurdering ved å se på usikkerheter, sårbarhet, skredforventning og skredfrekvens
- Area 3 integrerte forventet klimaendring og endring i demografi og infrastruktur inn i den harmoniserte kvantitative risikovurderingen
- Area 4 så på tekniske og praktiske aspekter i forhold til overvåking og varslingssystemer for skred
- Area 5 så på håndtering av risikoaspekter.

Prosjektdeltakerne kom fra i alt 27 institusjoner fra 13 land. NGI/ICG var koordinator for prosjektet, som hadde et totalt budsjett på nesten 9 millioner Euro.

SUMMARY

The need to protect people and property with a changing pattern of landslide hazard and risk caused by climate change and changes in demography, and the reality for societies in Europe to live with the risk associated with natural hazards, were the motives for the project SafeLand: "Living with landslide risk in Europe: Assessment, effects of global change, and risk management strategies."
SafeLand was a large, integrating research project under the European Commission's 7th Framework Programme (FP7). The project started on 1 May 2009 and ended on 30 April 2012. It involved 27 partners from 12 European countries, and had international collaborators and advisers from China, India, USA, Japan and Hong Kong. SafeLand also involved 25 End-Users from 11 countries. SafeLand was coordinated by the International Centre for Geohazards (ICG) at Norwegian Geotechnical Institute in Norway. Further information on the SafeLand project can be found at its web site http://safeland-fp7.eu/.

Main results achieved in SafeLand include:

- Various guidelines related to landslide triggering processes and run-out modelling.
- Development and testing of several empirical methods for predicting the characteristics of threshold rainfall events for triggering of precipitation-induced landslides, and development of an empirical model for assessing the changes in landslide frequency (hazard) as a function of changes in the demography and population density.
- Guidelines for landslide susceptibility, hazard and risk assessment and zoning.
- New methodologies for physical and societal vulnerability assessment.
- Identification of landslide hazard and risk hotspots for Europe. The results show clearly where areas with the largest landslide risk are located in Europe and the objective approach allows a ranking of the countries by exposed area and population.
- Different regional and local climate model simulations over selected regions of Europe at spatial resolutions of 10x10 km and 2.8x2.8 km. These simulations were used to perform an extreme value analysis for trends in heavy precipitation events, and subsequent effects on landslide hazard and risk trends.
- Guidelines for use of remote sensing techniques, monitoring and early warning systems.
- Development of a prototype web-based "toolbox” of innovative and technically appropriate prevention and mitigation measures. The toolbox does a preliminary assessment and ranking of up to 60 structural and non-structural landslide risk mitigation options.
- Case histories and "hotspots" of European Landslides have been collected and documented. Data for close to fifty potential case study sites have been compiled and summarized. Most of the case study sites are located in Europe (Italy, France, Norway, Switzerland, Austria, Andorra, and Romania); but they also include one site in Canada and one in India. Almost every type of landslide and every type of movement is represented in these sites.
- Research on stakeholder workshops and participatory processes to involve the population exposed to landslide risk in the decision-making process for choosing the most appropriate risk mitigation measure(s).

WORK AREA 1: LANDSLIDE TRIGGERS AND RUN-OUT

<u>WP 1.1: Identification of mechanisms and triggers</u>

The activity was aimed at the development of a new characterisation framework for landslides based on triggering processes rather than on landslide material and kinematics. A scheme based on triggering mechanisms helps in identifying the influence of climate change on landslide activity.

Two main activities were carried out:

1) Production of Deliverable D1.1. The document is focused on the identification of triggering mechanisms and related landslide types, also as a function of the geo-environmental setting. The work included:
    - an extensive and updated literature survey has been used as a starting point to structure the deliverable

- collection of selected literature and original case studies in order to better explain the state of the art of knowledge and modelling of the landslide triggering processes possibly active in the EU context. This also included an overview of the landslides which involve the pyroclastic overburdens covering mountains around Vesuvius:
    i. an introduction on the flow-type landslides and lists the historical events occurred in Campania
    ii. macro-zoning of areas susceptible to flow-type landslides in the territory of the Campania Region
    iii. stratigraphical characteristics of many historical and recent events
    iv. correlation between rainfall infiltration/run-off and flow-type landslides occurrence
- modelling / parametric analysis of specific landslide triggering mechanisms and case histories. This included a study on earthquake induced landslide mechanisms covering:
    i. information regarding the proposed classification of earthquake triggered slides
    ii. parameters affecting the slope stability
    iii. the methods to estimate co-seismic landslide displacements
    iv. illustrative examples of earthquake triggered landslides occurred in Europe and worldwide describing the basic failure mechanisms
- definition of the geotechnical aspects of landslide triggering
- analyses of the landslide triggering phenomena due to changes in slope geometry
- study of the influence of human activities on landslide triggering

2) Modelling of triggering processes for large rockslides and shallow landslides in soil materials. Research included:
    - For rockslides and debris slides, characterisation and analysis activities aimed at modelling the relationships among hydrological triggers (rainfall, snowmelt, groundwater changes) and landslide displacements and displacement rates was carried out. Activities have been carried out with different approaches at the test sites of Courmayeur/La Saxe (analysis of rainfall and snowmelt contribution to rockslide displacements), Ruinon (empirical rainfall intensity-duration-displacement rate threshold definition), and Bindo/Cortenova (development and testing of a visco-plastic modelling approach) and calibrated using monitoring data, in order to support the definition and implementation of Early Warning threshold values in the framework of WP4.3;
    - For shallow landslides, the effect of antecedent rainfall, rainfall intensity and rainfall duration on the triggering of landslides have been investigated with physically-based models. This effect was analysed for present and future climatic scenario, in order to assess the impact of rainfall changes on shallow landslide triggering. This research was initiated during the first reporting period and continued in the second period. The results of this research has been published on Climatic Change (Melchiorre et al, 2011) and presented at the EGU General Assembly 2012 within the Safeland session (Melchiorre et al, 2011).

<u>WP 1.2: Geomechanical analysis of weather-induced triggering processes</u>

The aim of WP1.2, "*Geomechanical analysis of weather-induced triggering processes*" was the reporting of an advanced understanding of weather-slope behaviour relationships (*Objective 1*) to be applied for reliable landslide predictions over both spatial and temporal scales (*Objective 2*).

Deliverable D1.2 "*Geomechanical modelling of slope deformation and failure processes driven by climatic factors: Shallow landslides, deep landslides and debris flows*" is a wide overview on the state of the research around the complex interaction between weather and slopes focusing on the role of weather, which can strongly influence the behaviour of slopes, sometimes leading to catastrophic events. To this aim, paradigmatic cases are considered to highlight factors and mechanisms of slope movements in different geomorphological contexts: in particular, peculiar features of every case are carefully discussed stressing the relationship between mechanisms and mechanics of movements. A part of the report concerns the potential effects of incoming climatic changes, showing that these could be different in areas of the world because of the different climatic inputs that are expected in everyone and of the different mechanical response of different materials to even similar input.
Deliverable D1.3 "Analysis of the results of laboratory experiments and of monitoring in test sites for assessment of the slope response to precipitation and validation of prediction models" concerns laboratory and field experiments and monitoring sites and aims at providing groundwork for all other deliverables within the work package. An introduction to the state of the art is provided in the initial chapters – these chapters describe the need for well thought out experiments on both large and small scale, and for results to be sensibly and thoughtfully utilised. Experiments discussed include centrifuge and flume experiments in the chapter 2, and full scale experiments, such as the Rüdlingen landslide triggering experiment chapters 3 and 4. The final chapter goes on to describe case studies of landslides, which have been subjected to long term monitoring and discusses what lessons can be learned from them. The lesson is that both small and full scale landslide experiments must be used in conjunction with each other and a careful laboratory programme to characterise the key properties in the ground in order to identify and understand the mechanisms governing landslide triggering and subsequent behaviour. Experimental results indicate a great variation in landslide type and behaviour dependent on multiple factors, such as soil type, void ratio and density. The need to understand these mechanisms is clear. D1.3 provides a valuable source of material, organised by both method and soil type, in order to provide a backbone to the work for others to draw from, and for lay readers to learn about known and unknown factors within landslide prediction.

Deliverable D1.4 addresses first as a synthesis of deliverable D1.2 the different key physical mechanisms in landslides and the mathematical frameworks that have been developed in the soil mechanics community to model the time-dependent processes of water flow, pore pressure dissipation and deformation in soil slopes. Guidelines are presented for landslide modelling which provide the user in the first place with some information on the utility of numerical codes. In the second place, these guidelines allow the user to identify the necessary code components for a given landslide problem, select the necessary and most important field data for the model and perform the modelling steps, including data pre-processing, the actual numerical calculation, as well as the post-processing of the results. The geomechanical codes used in the SafeLand project are evaluated with respect to the availability of components which are necessary or which allow to obtain additional expertise on different, particular landslide problems. The last section is dedicated to a discussion on the use of geomechanical modelling for early-warning systems and the prediction of the behaviour of large landslides under different climatic scenarios.

WP 1.3: Statistical studies of thresholds for precipitation-induced landslides

The partners participating in this Work Package applied existing models for estimating empirical thresholds for precipitation-induced landslides. The types of models used were: Intensity-Duration, Antecedent precipitation, Intensity-Antecedent precipitation-Duration (I-A-D), FLaIR, Neural Networks and empirical dynamic models. The models were applied to five datasets from France, Switzerland, Italy and Norway. The dataset where most models were applied was Barcelonnette (France). In this case study, three different models were used. Thresholds were evaluated taking into account the type of landslide under consideration. The results indicate that thresholds for debris flows are adequately predicted by the triggering rainfall only (durations between 1 and 9 hours), without requiring consideration for antecedent precipitation. Soil slides require accounting for antecedent precipitation ranging between 7 and 46 days, depending on the case. Rock falls and rock slides are poorly predicted due to the lack of inclusion of other key factors in triggering conditions, such as freeze-thaw effects. It is important to note that the derived thresholds are applicable only to the case studies under consideration in this Work Package, and any use of these results for other geographical areas or type of landslides is not recommended without a careful adaptation to each particular condition.

The WP1.3 activities result to deliverable D1.5, which is uploaded on the SAFELAND extranet in its final form. A short description of this deliverable follows:

D1.5: *Statistical and empirical models for prediction of precipitation-induced landslides*

This deliverable presents statistical and empirical models for predicting critical meteorological elements and their thresholds for triggering of landslides at local and regional scales. The models are evaluated using rainfall data and landslide observations from five datasets from France, Switzerland, Italy and Norway. The application of models at a local scale to the La Frasse dataset (Switzerland) demonstrates the potential for integrating field measurements of landslide displacements and rainfall observations for effectively performing forward predictions. The Barcelonnette dataset (France) was assessed for thresholds for debris flows and soil slides, which indicated triggering durations in the range of 1-9 hours, and 3-17 hours, respectively. An intensity-duration threshold was sufficient for debris flows, but for soil slides, an intensity-antecedent precipitation-duration threshold was necessary to achieve improved performance.

The critical antecedent precipitation corresponded to 23 days. At a regional scale, antecedent precipitation models were also applied to datasets from two locations in Norway using daily rainfall observations. The results from this model indicate that 1-day and 7-day antecedent precipitation are critical for debris flows and soil slides in a case study in Western Norway, while 46-day antecedent precipitation is critical for earth slides in a selected case study in South-Eastern Norway. The occurrence of rock slides and rock falls is weakly associated with rainfall parameters, suggesting the necessity of incorporating other relevant effects, such as freeze-thaw conditions. Modelling of the occurrence of debris flows requires the use of rainfall observations with a high sampling rate (e.g., hourly). The FLaIR model was calibrated successfully for predicting triggering conditions for soil slides in the Barcelonnette basin, and in three case studies in Italy.

It was not possible to apply all models to all case studies. This was partly due to the suitability of each model depending on the scale and type of data. For example, the Neural Networks and the empirical dynamic models were implemented to be suitable for application to an individual landslide site, but not for a regional dataset. Similarly, the Intensity-Duration and

Intensity-Antecedent Precipitation-Duration models were not suited for evaluation of datasets with low frequency of observations (i.e., daily data). The models incorporated precipitation observations only because this is the only type of data that was available from the partners contributing the datasets. Two appendices present recent experiences of thresholds incorporating the effect of snow-melt.

WP 1.4: Landslides triggered by anthropogenic factors

The main objective of WP1.4 was to improve our knowledge about the impact of human activities on increasing or decreasing the landslide hazard. Landslides can be triggered by both natural and human-induced changes in the environment. Human-induced landslides may result from changes in slope caused by terracing for agriculture, cut-and-fill construction for highways, construction activity, mining operations, rapid draw-down of dams, changes in land cover such as deforestation, and changes in irrigation or surface runoff.

The human-induced landslides are caused by changes of the strength or effective stresses, changes in geometry and boundary conditions, and modifications or changes of the material behaviour. The most common anthropogenic factor leading to slope instability is the modification of slope profile, usually caused by cut-and-fills that decrease the factor of safety. The effects of changes in the pore pressure and ground water regime are several. On one hand this can simply change the behaviour of the material. For instance an artificial increase of the water flow accumulation and/or infiltration can lead to the full saturation of a material which had never been saturated in the past (and initiate a mud-flow). On the other hand the rising up of the water table caused by changes of water infiltration or reduction of permeability because of consolidation (for example by load) may produce new conditions that decrease the factor of safety.

A catastrophic event can result in a slow modification of properties and/or conditions of the stability which increase the sensitivity to triggering factors. However, these modifications could also be the direct trigger for the landslide event because new conditions are encountered that did not exist before. Construction of new infrastructures and changes in land use could also increase the susceptibility to landslides. The problems linked to road cut-and-fills are easy to understand, and uncontrolled quarrying has been a well-identified problem for a long time. Rapid draw-down of dams could lead to the destabilization of reservoir slopes. Some of the most critical issues in terms of risk are linked to the (sub-) surface water flow or the pipe leakage caused by aging that creates shallow landslides in urbanized areas.

It should be noted, however, that in many situations, the human activities have intentionally or unintentionally improved the slope stability and reduced landslide hazard. The anthropogenic factors could therefore play a positive role in reducing the landslide risk. Development of an empirical model for assessing the changes in landslide frequency (hazard) as a function of changes in the demography and population density was one of the main results of the work done in this work package.

WP 1.5: Run-out models

This work package WP 1.5 aimed at presenting models to describe the physical processes involved in landslide propagation, especially in the case of rapid landslides. The results will be used to do complete hazard and risk zonation. The tools allow predicting the velocity and thickness of landslides, as well as the running up along valley flanks and obstacles of different shape and position. The activities contained in this WP 1.5 have been the following:

- Joint R&D work aiming at fulfilling the objectives of this WP, which has included visits and stages of personnel belonging to different SAFELAND Teams.
- There have been contacts between partners in order to write and correct deliverable D1.7
- The main activity has consisted on collecting information on run out modelling alternatives, to check available codes, and to materialize this information in the deliverable D 1.7

The deliverable has been structured into the main following Sections:

- Mathematical modelling framework, where we have described a set of hierarchically structured mathematical models describing the basic phenomena taking part in propagation phenomena.
- Analysis of different rheological models describing the behaviour of fluidized soils. The analysis was based on studying general 3D rheological models from which depth integrated models could be developed.
- Numerical models for propagation. One main result of the R&D work done in this WP which has been used in WP 1.6 is an analysis of the relative advantages and disadvantages of depth integrated models of Eulerian and Lagrangian types. While both approaches are of similar quality in most of the cases, they present relative advantages and disadvantages which we will describe here. We include some of the conclusions:
    - The main advantage of meshless methods, when compared to Eulerian finite elements or finite volume, is the computational cost. In fact, the time of computation is lower than that of classical, Eulerian finite elements, because the computational grid is separated from the structured terrain mesh used to describe terrain topography. The authors of this report have measured differences in computer time of ratios close to 1:30 in favour of SPH when analyzing the propagation of lahars in the Popocatépl volcano. The reason is that only a very small part of the topography was occupied by the propagating lahar. In the case of finite elements, all nodes had to be active, hence the much larger cost. In other occasions, as for instance, when studying the propagation of a mudflow originated by the failure of a tailings dam, times are more similar (ratios close to 1:5) Here the reason is that most of the computational domain was occupied by the flow.
    - Another aspect which favours SPH methods is the mass conservation, which is enforced in a more effective way. Eulerian finite element models of landslide propagation over long distances suffer from a loss of mass which is much larger than that found in SPH methods. The reason is that Eulerian methods used to make zero heights smaller than a threshold value to avoid numerical instabilities.
    - On the other hand, simulation of walls containing a fluid is much easily dealt with finite elements than with SPH methods, which do require special techniques due to their boundary deficiency problem.
    - Finally, one important limitation of SPH methods arises when using hydrographs to apply boundary conditions related to the incoming flow in a domain. Indeed, the solution in SPH is to inject nodes, but then we need to apply initial conditions on them, and not boundary conditions.

- Applications. The purpose of this section has been to illustrate the use of both Finite elements and SPH models. The section includes Benchmarks which will allow the assessment of run out models. We have considered the following groups (i) Problems with an analytical solution (ii) Small scale laboratory tests (iii) Real landslide cases for which we have consistent information.

WP 1.6: Identification of models best suited for quantitative risk assessment (QRA)
The main results from WP 1.6 are synthesized in the two deliverables D1.8 and D1.9, which complement D1.2, D1.3, D1.4 and D1.7 from other WPs. D1.8 and D1.9 provide advice to users on how to work with selected codes, and to warn them of the most frequent error and sources of inaccuracies.

Deliverable D1.8 *Guidelines: recommended models of landslide triggering processes and run-out to be used in QRA* concerns weather-induced and earthquake-induced landslides. The main aspects to consider in the analysis of the slope response are illustrated, and the difficulty to assess some of the factors required to provide a QRA are clearly stressed. Furthermore, suggestions for a quantitative prediction of landslide triggering are provided accounting for the codes already carefully described in deliverables D1.2, D1.4 and D1.7. An entire part of the report is devoted to earthquake-induced landslides which have not been dealt with in other WPs.

Precipitation-induced landslide triggering has been carefully examined in Part I. The available codes usable at a regional/basin scale and those conceived for a slope scale have been separately treated since they pose very different implementing problems. Limit of current methods for analysis, and advantages and constraints of every code have been carefully considered and highlighted. Special consideration has been dedicated to unsaturated soils since the major implementing problems and the fastest landslides concern such materials; on the other hand the most of sloping grounds generally present an unsaturated cover.

Regarding run-out, Part III presents a general overview of recommended models to be used for QRA, including mathematical, rheological and numerical models. It is considered that depth integrated models present an interesting compromise between accuracy and complexity. This subjected is more deeply examined in deliverable D1.9. Earthquake-induced landslides are discussed in Part II, which describes current practice to assess earthquake induced landslide triggering processes with special reference to analysis of hazard and calculation of both safety factor and run-out of landslides of the slide type (Newmark-type methods and dynamic methods). Suggestions are provided to perform correct analyses accounting for the available data.

WORK AREA 2: QUANTITATIVE RISK ASSESSMENT (QRA)

WP 2.1: Harmonization and development of procedures for quantifying landslide hazard
Deliverables of WP2.1 consist of compilations of existing resources (landslide databases) and methodologies for QRA.

The basic goal of the work package is to harmonise landslide data bases and procedures for hazard and risk assessment. More specifically: (a) review the existing databases and propose improvements for achieving interoperability and harmonisation; (b) review current practices in Europe for landslide mapping, regulations and codes; (c) provide recommendations for quantitative risk assessment at different scales. To achieve this goal the following activities have been carried out:

1. Compilation of the European landslide databases (deliverable D2.3)
2. Review of procedures for Quantitative Risk Assessment (QRA) and preparation of the Guidelines for landslide susceptibility, hazard and risk zoning. This task has been split in two phases: the review of the existing practices of landslide hazard and risk assessment in Europe and abroad (deliverables D2.1 and D2.2) and preparation of Guidelines for QRA (deliverable D2.4).

The objective of deliverable D2.1: "*Overview of current landslide hazard and risk assessment practices in Europe*" was to review the current practices, regulations and codes in Europe for landslide mapping, susceptibility, hazard and risk assessment. The contents of this deliverable refer to the existing official practices that are currently promoted or applied by administration offices, geological surveys, and decision makers. New research developments in both qualitative and quantitative landslide hazard and risk assessment are not considered here and are treated in deliverable D.2.4. The reported countries and territories are: Andorra, Austria, France, Italy (selected river basins from southern, central and northern Italy), Norway, Romania, Spain (Catalonia), Switzerland and United Kingdom. A comparison of the European experiences was performed so as to highlight the similarities and differences of the various methodologies, focusing on the policies for hazard and risk evaluation, the existing official documentation and contents, the used methodologies (for different scales and landslide types), and the used terminology and map symbols. The main conclusions resulting from this comparison are:

- The classification criteria for landslide types and mechanisms present large diversity even within the same country. As a result in some cases no landslide mechanisms are specified and in some others there is an exhaustive list. Each mechanism requires its own method of assessment. The differentiation of landslide types and mechanisms is recommended particularly for scales larger than 1:25,000.
- In relation with the types and mechanisms of landslides represented in the maps, in general they are grouped in a few general mechanisms (i.e. rockfalls, slides, flows).
- The effect of hazard amplification due to the spatial superposition of different types of instabilities should also be taken into consideration, as well as the synergistic action of other natural phenomena (i.e. earthquake) wherever applicable, regardless of the mapping scale.

Additionally, within the framework of the WP2.1 a workshop was organized in the Chengdu University of Technology on April 13 and 14, 2010, with the aim of assessing the state of art of landslide hazard and risk assessment in the Peoples Republic of China. To achieve this objective, Chinese experts in landslide hazard and risk assessment were invited to give presentations and write a chapter for a report which form one of the deliverable D2: "Examples of international practices in landslide hazard mapping – India and China", which has been edited and completed by ICG and ITC. The report will also be published as a book in China.

Deliverable D2.2 included contributions from China and India summarising landslide risk assessment work from their respective countries. In China a workshop was organized in the Chengdu University of Technology on April 13 and 14, 2010, with the aim to assess the state of art of landslide hazard and risk assessment in the P.R. of China. For achieving this objective, Chinese experts in landslide hazard and risk assessment were invited to give presentations and write a chapter for a report which would form one part of D2.2. The report will also be published as a book in China. The overview of landslide hazard and risk practices in India was prepared by IIT Roorkee.

Deliverable D2.3 "*Overview of European landslide databases and recommendations for interoperability and harmonization of landslide databases*" made a detailed review of existing national landslide databases in Europe together with a number of regional databases and proposed improvements for delineating areas at risk in agreement with the EU Soil Thematic Strategy and its associated Proposal for a Soil Framework Directive, and for achieving interoperability and harmonization in agreement with INSPIRE Directive. The report was based on the analysis of replies to a detailed questionnaire sent out to the competent persons and organizations in each country, and a review of literature, websites and main European legislation on the subject.

Deliverable D2.4: "*Guidelines for landslide susceptibility, hazard and risk assessment and zoning*", has been prepared aiming at

- recommending methodologies for the quantitative assessment and zoning of landslide susceptibility, hazard and risk at different scales (site specific, local, regional and national);
- proposing specific methodologies for different landslide mechanisms;
- including a selection of the best suited procedures for verification of the models and validation of the results.

WP 2.2: Vulnerability to landslides

Research activities have resulted in the finalization of all methodologies for assessing the vulnerability of different elements at risk (buildings, lifelines, persons etc.) exposed to different landslide hazards. Both physical and socioeconomic vulnerability is addressed. Some of the developed methodologies were also applied to selected real case studies. The outcome of WP 2.2 activities is the three deliverables D2.5, D2.6 and D2.7.

D2.5: *Physical vulnerability of elements at risk to landslides: Methodology for evaluation, fragility curves and damage states for buildings and lifelines*

The report aims at the proposition and quantification of efficient methodologies for assessing physical vulnerability of buildings, persons and lifelines to different landslide hazards using the concept of probabilistic fragility functions or indexes, and appropriate definition of relevant damage states. An attempt to distinguish between different types of landslides and affected assets (building, persons and infrastructure) has been made. The applicability of the developed methodologies depends on a few general parameters such as the landslide type, the typology and classification of elements at risk, the analysis scale and the triggering mechanism (intense rainfall, earthquake). The main landslide movement types considered herein are rockfalls, debris flows and slow moving landslides. Four different analysis scales are considered: small (1:100,000), medium (1:25,000), large (1:5.000) and detailed/site specific (1:2000), requiring different criteria to identify the elements at risk. Finally, various intensity parameters are considered (e.g. permanent displacement, landslide velocity, volume of the landslide deposit, impact force, kinetic energy etc.) depending on the landslide type, the element at risk and the scale of analysis.

D2.6: *Methodology for evaluation of the socio-economic impact of landslides (societal vulnerability)*
This report deals with socio-economic vulnerability related to landslides. After a thorough literature review, it presents an indicator-based methodology to assess vulnerability levels. The indicators represent the underlying factors which influence a community's ability to deal with, and recover from the damage associated with landslides. The proposed method includes indicators which represent demographic, economic and social characteristics as well as indicators representing the degree of preparedness and recovery capacity. When using the proposed method, the societal vulnerability is ranked on a relative scale from 1 (lowest vulnerability) to 5 (highest vulnerability). The purpose of the indicators is to set priorities, serve as background for action, raise awareness, analyze trends and empower risk management.

D2.7: *Case studies of environmental and societal impact of landslides*
Part A of this report presents representative applications of the methods described in Safeland deliverable D2.5 for assessing physical vulnerability of buildings and roads affected by different landslide hazards and at different scales. In particular, the physical vulnerability in terms of building's (homogeneous) aggregates due to different slow moving landslide hazards is assessed at the territory of the National Basin Authority of "Liri-Garigliano" and "Volturno" rivers, Central-Southern Italy at small scale (1:100.000). Moreover, the vulnerability of smaller building aggregated levels affected by slow movements at two study areas (scale 1:25.000) within the already investigated territory was estimated. In addition, the vulnerability of buildings subjected to rainfall induced slow moving landslides located at the test site of San Pietro in Guarano, Cosenza Province, southern Italy (scale 1:2000) is assessed. The physical vulnerability of a representative reinforced concrete (RC) building subjected to earthquake triggered slow moving landslide hazards located near the Kato Achaia (western Greece) slope's crest is investigated. Finally, the vulnerability of the roadway system of Grevena in Greece due to earthquake triggered landslides is assessed. The method has been proposed for seismically induced displacements but it could be equally implemented for the case of hydrological hazards.

Part B of this report presents applications of the developed socio-economic vulnerability model in Safeland deliverable D2.6 for six locations, two in Norway and one each in Greece, Andorra, France and Romania. The purpose of the case studies has been to compare vulnerability levels and to test and possibly improve the methodology proposed in SafeLand Deliverable D2.6 titled *Methodology for evaluation of the socio-economic impact of landslides (societal vulnerability)*.
The vulnerability scores obtained for the two locations in Norway and the locations in Andorra and France were similar (2.0 – 2.1). The vulnerability estimate for Grevena in Greece was higher (2.7), while the highest vulnerability among the analyzed location was Slănic in Romania (3.6). The case studies were repeated with an updated version of the method. The updated method resulted in a similar ranking of vulnerability for the case study locations as obtained with the original method.

The most significant results in WP2.2 within the reporting period are summarized as follows:

- Proposition of a generic vulnerability model which can be used for a large portfolio of rockfall protection galleries. The methodology includes three main steps: (a) definition of the exposure for rockfall protection galleries, (b) resistance modelling for rockfall protection galleries and (c) development of vulnerability curves for rockfall protection galleries; each of these involves various sub-steps. Different sources of uncertainty can be included in the analysis in a quantitative cost-effective manner (ETHZ).
- Literature review on existing models for the quantification of physical vulnerability of persons exposed to different landslide hazards. The most important factors concerning the different aspects of physical vulnerability of persons to landslides are discussed (AUTH).
- Comparison of the derived curves for roads to debris flows developed within the 1st reporting period by TRL-AUTH-UPC with real debris flow events from both Scotland in the UK and the Republic of Korea (TRL).
- Assessment of the physical vulnerability in terms of building's (homogeneous) aggregates due to different slow moving landslide hazards at the territory of the National Basin Authority of "Liri-Garigliano" and "Volturno" rivers, Central-Southern Italy at small scale (1:100.000). Moreover, the vulnerability of smaller building aggregated levels affected by slow movements at two study areas (scale 1:25.000) within the already investigated territory is assessed (UNISA).
- Estimation of the vulnerability of buildings subjected to rainfall induced slow moving landslides located at the test site of San Pietro in Guarano, Cosenza Province, southern Italy (scale 1:2000) based on the corresponding methodology developed in D2.5 (UNISA).
- Investigation of the physical vulnerability of a representative RC building subjected to earthquake triggered slow moving landslide hazards located near the Kato Achaia (western Greece) slope's crest to assess the validity of the derived fragility curves proposed in D2.5 (AUTH).
- Assessment of the vulnerability of the roadway system of Grevena in Greece due to earthquake triggered landslides based on the proposed fragility functions developed in D2.5 (AUTH).
- Improvement of the socio-economic vulnerability model by introducing 3 new indicators referring to critical infrastructure, risk awareness and early warning capacity (ICG).
- The socio-economic vulnerability model is applied to four more locations (expect for the two in Norway already implemented during the first reporting period): Andorra-Spain/France, Barcelonnette -France, Grevena-Greece, Slănic -Romania. (ICG)

<u>WP 2.3: Development of procedures for QRA at regional scale and European scale</u>

Three deliverables, D1.8, D1.9 and D2.11, have been produced in WP 2.3. Deliverable D2.8 "*Recommended procedures for validating landslide hazard and risk models and maps*" proposes methods for the:

- Quantification of the reliability of the assessment, accounting for:
    - data vagueness and uncertainties (relevant to landslide inventory and conditioning factors and material constitutive parameters);
    - accounting for the "limited" knowledge on the physics of the processes (relevant to hydro- and mechanical understanding of the mechanisms and process modelling);
    - taking into account the issue of the "mapping unit", independently of the scale.
- Quantification of the validity of the assessment, considering:
    - validation/evaluation of the maps

  - o the multi-criteria problem of adequacy (conceptual, mathematical) in describing the system behaviour, robustness to small changes of the input data (i.e. data sensitivity), and accuracy in predicting the observed data
  - o the type of the output (susceptibility / hazard / risk).

The proposed methods are summarized to:

- Methods and measures to quantify the reliability of the assessment:
  - o Sources of uncertainties in landslide susceptibility, hazard and risk assessment
  - o Approaches to account for uncertainty on landslide inventories
  - o Scenario-based approaches for landslide susceptibility, hazard and risk assessment
  - o Probabilistic approaches for landslide susceptibility, hazard and risk assessment
- Methods and measures to quantify the validity of the assessment
  - o 'Statistical' validation methods:
  - o 'Data-driven' validation
  - o 'Expert or knowledge-driven' validation methods

Deliverable D2.9: "*A toolbox for landslide quantitative risk assessment*" is composed by three tools (computer applications) and a manuscript that is addressed to the end-users and it includes the description of each toolbox, the prerequisite inputs, the obtained outputs, the followed methodology and possible limitations for its use. The three tools serve at:

- Landslide quantitative risk assessment
- Rockfall quantitative vulnerability of buildings
- Rockfall quantitative risk assessment

The objective of Deliverable D2.11 "*QRA case studies at selected "hotspots"*" was to present some practical applications of QRA that may serve as examples that might be followed by scientists and practitioners depending on the afore-mentioned factors (landslide type, scale, risk descriptors etc). The added value of them is that, in comparison with the current state-of-the art (see Deliverable D2), they incorporate innovations related to the calculation and hazard and vulnerability in order to incorporate them into the risk assessment. The goal of this deliverable is to cover a range of different cases as far as it concerns:

- the application scale: regional, local, site-specific;
- the landslide type: debris flow, deep-seated landslides, hyper-concentrated flows, rockfalls;
- the source of input data: empirical to remote sensing;
- the inclusion or not of the run-out modelling;
- the vulnerability assessment: buildings or people, empirical or analytical, deterministic or probabilistic, element at risk-orientated (detailed) or generalised;
- the used risk descriptors: qualitative or quantitative, and in what terms;

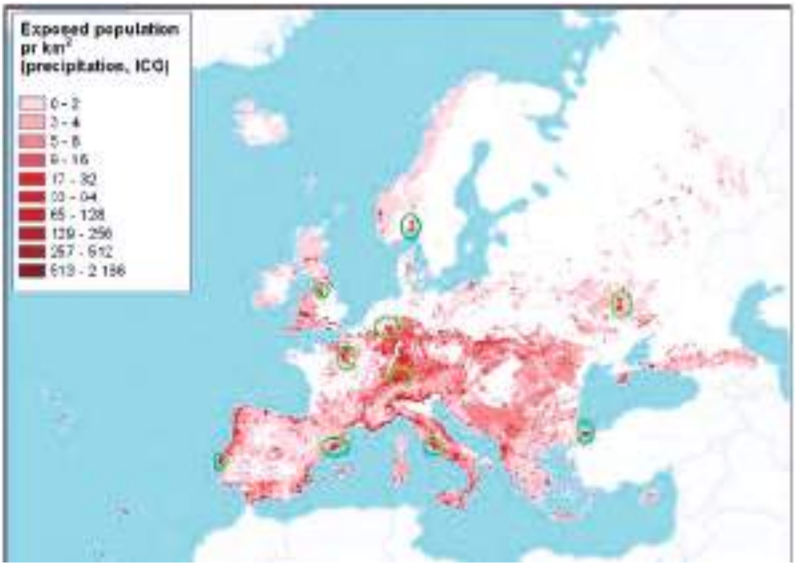
To this end, five-case studies were presented in this deliverable:

- Debris slides – rapid earthflows at Castellamare de Stabia, Naples province, Italy
- Deap-seated landslide in Ancona, Italy
- Hyperconcentrated flow at Nocere Inferiore, Italy
- Rockfalls at the Solà d'Andorra, Andorra
- Rockfalls at Fiumelatte, Italy

WP 2.4: Identification of landslide hazard and risk “hotspots” in Europe

The objective of WP2.4 is to perform a first-pass analysis of landslide hazard at European scale to identify the landslide hazard and risk "hotspots", where hazard and risk are highest. Hotspots of landslide hazard and risk were identified by an objective GIS based analysis for Europe. The results show clearly where landslide pose the largest hazard in Europe and the objective approach allows a ranking of the countries by exposed area and population. In absolute numbers Italy is the country with the highest amount of area and population exposed. Relative to absolute number of inhabitants and area, the small alpine countries such as Lichtenstein and Montenegro score highest where as much as 40% of the population is exposed. It is obvious that the type and quality of the input data is decisive for the quality of the results. Especially the estimation of extreme precipitation needs improvement.

The results can be found at the final version of the deliverable D2.10: “*Identification of landslide hazard and risk "hotspots" in Europe*”. The final version also includes a detailed description of the applied models and a discussion of the differences between model results.



Exposure map for Europe with possible hotspots marked in green.

WORK AREA 3: GLOBAL CHANGE

WP 3.1: Climate change scenarios for selected regions in Europe

Different regional climate model simulations over Europe (from the EU FP6 project ENSEMBLES) at a spatial resolution of 25 x 25 km² have been used to perform an extreme value analysis for trends in heavy precipitation events. Furthermore, potential causes for trends in heavy precipitation have been investigated by analyzing a variety of thermodynamic and dynamic variables as simulated by the regional climate model REMO. Deliverable D3.1 reports on the performed work.

Climate change simulations with the regional climate model REMO are performed at a resolution of 10 x 10 km² for three selected regions over Europe: Italy and the Alps, Northern Europe, and Eastern Europe. The simulations have been carried out for the time period 1951-2050, employing the SRES A1B emission scenario. A detailed description of the simulations and an analysis of the changes of temperature and precipitation in the simulated future climate are given in deliverable D3.2.

These climate simulations are delivered to the Centro Euro-Mediterraneo per i Cambiamenti Climatici (CMCC) in order to be used as boundary conditions for further model simulations performed at a resolution of 3.8 x 3.8 km². The usage of the model output data for simulations on an even more refined grid is expected to improve the ability to simulate even localized heavy precipitation events in regions where rain-induced landslides occur on a regular basis.

The climate simulations at a resolution of 3.8 x 3.8 km² are presented in Deliverable D3.3. They have been performed by CMCC on four selected areas in Europe, using the regional model COSMO-CLM. The regions are

1. Nedre Romerike, Southern Norway
2. Pizzo d'Alvano, Campania, Italy
3. Barcelonnette, French Alps
4. Telega, Romania

Results of two-meter temperature and total precipitation averaged over the time periods 1971-2000 and 2021-2050 have been presented, in order to highlight the variations expected in the future, with respect to the past period. Furthermore, an extreme value analysis for projected future changes in heavy precipitation is carried out for the different regions and separately for summer and winter.
Finally, deliverable D3.4 presents a synthesis and discussion of the results. The analysis concentrates on projected future changes in heavy precipitation for four target regions in Europe: Southern Norway, Southern Italy, the Alps and Romania.

Main results D3.1:

- In winter we see a general trend towards more heavy precipitation events across all analyzed regional climate model simulations. This could partly be explained by an increased amount of vertically integrated water vapour.
- For summer, we could find a slight increase of heavy precipitation in Northern Europe and a general decrease in Southern Europe in all regional climate model simulations. The models suggest an increase of the air temperatures all over Europe and particularly in the southern part. Nevertheless, the trend of the vertically integrated water vapour does not follow the temperature trend linearly.

Main results D3.2

- The strongest warming is found in the southern regions in summer and over cold regions in spring and autumn, where the warming is amplified due to the snow-albedo feedback. Precipitation is projected to increase in cool and moderate regions, but decreases in the warm regions during the warm seasons.

Main results D3.3

- In the area of Nedre Romerike (Norway) strong increases of temperature are projected especially in winter, while a general increase of precipitation is expected in winter, with a general increase of extreme events which is most pronounced in the western part of the domain.

- In the area of Pizzo d'Alvano (Italy), a growth of temperature is also projected, even if less evident than the previous case. In winter, strong increases of precipitation (with strong extreme events) are expected in the area of Pizzo d'Alvano, In summer slight reductions are expected for the average monthly precipitation over the whole domain, which is in contrast to a projected increase in daily precipitation extremes in the Pizzo d'Alvano region and along the western coast line.
- In the area of Barcelonnette (France) significant increases of temperature are expected in the future, up to 3^{o} C, in both seasons, but especially in winter. An increase of precipitations is expected in small sub domains in both seasons, with slight changes of extreme events on the whole domain.
- In the area of Telega (Romania), a general increase of temperature of about 1.5^{o} C is expected over the whole domain, for both summer and winter. In winter an increase of precipitation is expected, while a general significant reduction is expected in summer; an increase of extreme events is expected in winter and summer in the north of the domain with the magnitude of the changes being higher in winter.

Main results D3.4

- Both the analyses from D3.1 and D3.3 show mainly positive trends of heavy precipitation in winter. Strong changes are particularly found in mountainous regions, where the impact on landslides may be large. The summer trends in Northern Europe are generally weaker than the winter trends. In warm and rather dry regions, such as the Pizzo d'Alvano domain, which is located in Campania in South-Western Italy, or the Telega domain in Romania, the average summer precipitation is projected to decrease. In other words, dry regions tend to become even drier. For extreme events on the other hand, increasing trends are found for some of these regions, in particular in the high-resolution COSMO-CLM simulation. This indicates that especially the typical convective events in summer may occur with higher probability. For regions where the average precipitation decreases and the soil dries, the drainage of the soil is reduced and consequently the occurrence of heavy precipitation events may have strong impacts, such as flooding. On the other hand, regions which already possess a moist climate, such as Southern Norway, tend to become even wetter on average and also in the extremes.

The results of Work package 3.1 are used in Work package 3.3, in particular in Deliverable 3.7.

<u>WP 3.2: Human activity and demography scenarios</u>

The initial objectives of WP 3.2 were to provide information on prospective human activity and demography evolution in Europe and in selected sites in correspondence with IPCC scenarios at four selected dates: 2030, 2050, 2070 and 2100. In order to provide such data, gathering, compilation and interpretation of available data had been necessary at both national and European level.

<u>Task 1: European and national levels</u>

The objective of deliverable D3.5 was to respond to the following core questions:

1. What are human activities and population characteristics that affect or are affected by landslides?
2. What recent and existing scenario projects are relevant for the activities of the SafeLand project?
3. What relevant data are available for which scale, and are there forecasts for the years 2030, 2050, 2070 and 2100?

Based on results from other work packages, anthropogenic factors modify the three dimensions of landslide risk: exposure, hazard and vulnerability and the main factors that have an influence are demography, economics, and land use / land cover. To fulfil the second and third objectives, global scenarios (Project context, scenario specification, results and limitations), European scenarios (Project context, scenario specification, results and limitations) and some national scenarios have been reviewed. For each scenario reviewed, the context, the scenario specifications and the results and limitations have been presented.

The main result of this review was to stress that an abundant supply of data on all kinds of issues is available throughout Europe and the rest of the world. However, most countries and organizations that collect the data use different definitions of indicators, different methodologies and different territorial units. Depending on the region, data is more or less complete and reliable.

Task 2: Local sites
Subsequently, the evolution of human activity factors impacting landslide risk from 2030 till 2100 and at the level of test sites has been considered. The idea was to check data availability at the level of selected hotspots. When they exist, prospective data are used. Unfortunately, data are sparse, rarely spatialized and not always adapted to the local context. However, this lack of information can be partially compensated by the analysis of past and present trends. Satisfactory data have been collected for the Barcelonnette site and have allowed the elaboration of demography scenarios at local level by 2030. The land use change scenario by 2100 has been studied. Acknowledging significant uncertainties, the demographic forecasts can be extended from 2030 to 2100. The economic changes scenario has not been treated as such a scenario is really difficult to implement at a local scale and also to integrate in risk analysis process. Demographic scenarios have been partially developed for the Nedre Romerike site (Norway). Concerning the other proposed test sites: Pizzo d'Alvano (Italy) and Slanic (Romania), data concerning human activities are not sufficient for the moment to elaborate any kind of prospective scenario.

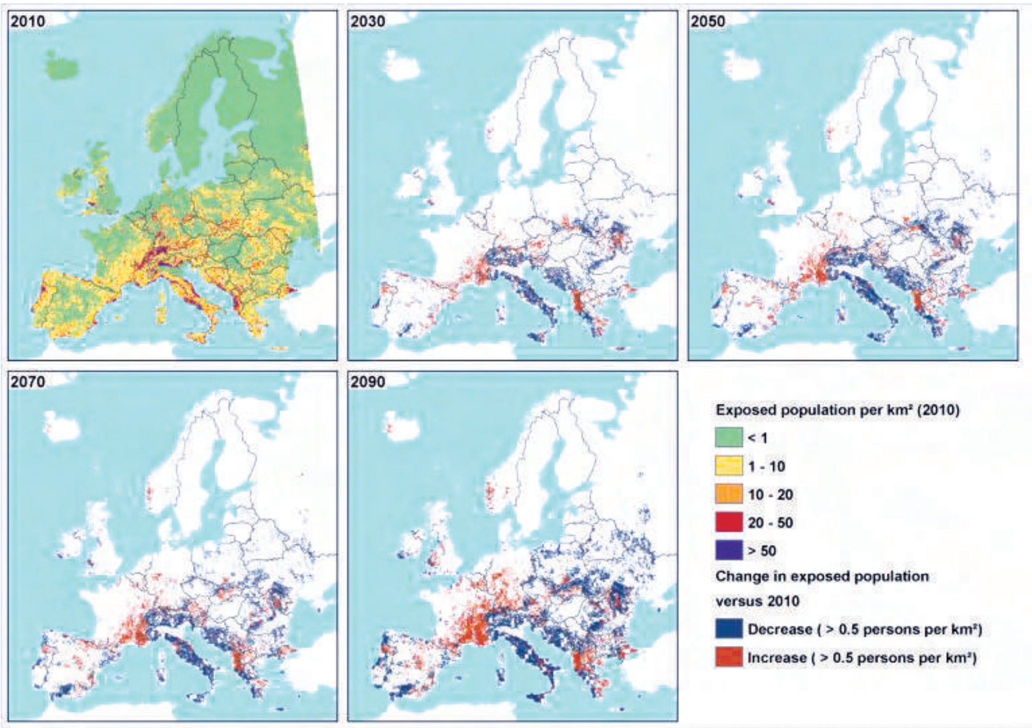
<u>WP 3.3: Landslide risk evolution in selected "hotspots" areas</u>
The initial objectives of WP 3.3 were:
1. Provide updated risk maps according to global change at four "hotspots", representative of different landslide types and contexts in years 2030, 2050, 2070 and 2100
2. Analyse of impacts of risk evolution due to global change for mitigation strategies and risk management purposed on "hotspots"

Integration of the climate change scenarios:

This first objective has been mainly addressed in the deliverable 3.7: "*Expected changes in climate-driven landslide activity (magnitude, frequency) in Europe in the next 100 years* and deliverable 3.8 – *Changing pattern in climate-driven landslide hazard at selected sites Europe (focus on Southern Italy, the Alps, and Southern Norway) in the next 50 years"*. The European-scale analysis of present and future landslide hazard and risk has required many simplifications. The main difficulty was to find homogenous datasets that cover all of Europe with the same accuracy. This problem is even increased when the datasets have to cover future predictions.

The climate model results used in this study are based on a physical climate model and have a reasonable level of uncertainties in the future predictions. On the other hand, land cover and population datasets are secondary products based on climate simulations and economical modelling, which naturally include more errors in the process and are far more uncertain. In this context, the predicted changes in landslide hazard and risk in Europe, although certainly indicative, have to be investigated and used with care.

The main changes in landslide risk at European scale are mainly due to changes in population pattern in Europe. The results are showed in relative changes on the maps below:



The climate change scenario of WP 3.1 has been integrated in landslide hazard assessment at site specific scale in deliverable D3.8. The impact of climate change on landslide hazard has been assessed on the three focused areas: Pizzo d'Alvano for Southern Italy, Barcelonnette for the Alps and Nedre Romerike for Southern Norway. The data provided for the Romanian and Spanish sites were not sufficient to realize the full study of hazard assessment. In order to provide estimates for the Spanish site, the lack of climatic data has been dealt with by using climatic data from another regional climate model.

Different methods (statistical, empirical and physically-based methods) have been used on the different sites. Even if these sites present different contexts in view of landslides causes (climates, size of landslides), the analyses show that climate change is likely to induce similar trends in landslide activities. Based on the IPCC A1B scenario and on the resulting climate change scenario at local scale, the different models predict a very increase in landslide activities. This change would materialize either as an increase in the frequencies of landslides or as an increase in surface area of the potentially unstable areas. However, these models require precise data, not only for calibration but also for prediction, and so climate models should be adapted to such resolutions, like in this study.

The results differ from the predictions provided by larger scale models. These differences might be explained by the finer calibration processes used for local scale analysis and also to the finer climate model used, which, for example, take into account the influence of topography on climate (mostly on precipitation). So, if large scale models are useful to determine where landslide activities will vary relatively to the other regions, the different kinds of local scale models are necessary for urban planners and all local authorities to estimate what would be the future risks in their communes or valley, with for some of the models, spatial information. However, these models require precise data, not only for calibration but also for prediction, and so climate models should be adapted to such resolutions, like in this study.

Synthesis of the results for the different sites and comparisons with larger scale model:

Local site	Results from D3.7	Methods used in D3.8	Results at local scale
Norway	No significant changes	Statistical modelling	Increase in superficies of areas exposed to higher hazard ranks
French Alps	Hazard ranking slightly decreases	Physically-based model	Increase in surface area of instable slopes and their probability of failure.
Italy	Hazard ranking decreases	Physically-based model	Increase in the number of mountain basins involved during rainfall events
Spain	no significant change (scenario IPCC A1B)	Empirical model	Increase in landslide frequency (scenario IPCC B1)

Integration of land cover and human activity exposure scenarios and related vulnerability on selected "hotspots"

This objective has mainly been targeted by deliverable D3.9 – "*Methodology for predicting the changes in the landslide risk during the next 50 years at selected sites in Europe. Changing pattern of landslide risk in hotspot and evolution trends Europe according to global change scenarios.*"

The potential effect of climate change on landslide triggering depends on the type of landslides considered. For rainfall induced landslides, the hazard evolution is tightly linked to the variation of precipitation in time as threshold parameters. Other types of landslide may be impacted differently by climate change, for instance rock fall main triggering parameters are the frost and defrost cycles; but those cases were not developed in the work performed within Work Area 3. This work presented means of assessing landslide risk evolution with climate change scenarios. Those methods depend on the availability of input data. If climate change scenarios and land cover evolution scenarios can be developed quite accurately; the scenarios of population and human activity evolution are rougher; especially at site scales.

Nevertheless three studies of landslide risk assessment have been performed on French, Norwegian and Scottish sites. The results seem to show a similar trend: an increase of landslide risk which is more or less significant depending on the considered sites. Due to a high level of uncertainties on population and traffic evolution scenarios, precautions need to be taken when interpreting and using the results.

Some new avenues of research for a more precise assessment of the future landslide risk evolution at site specific scale have been investigated. A methodology has been developed to obtain simplified and rapid estimations of the influence of suction change due to vegetation/canopy on the factor of safety. The approach considers only the influence of the root water uptake caused by evapotranspiration and other phenomenon due to the presence of vegetation is not taken into account. In parallel, time-dependent fragility analysis of corroded RC buildings impacted by co-seismic permanent landslide displacements has been developed. In the future, this methodology could to be adapted to rainfall triggered landslides in order to evaluate risk evolution as a combination of hazard changes, exposure changes and vulnerability changes.

WORK AREA 4. MONITORING TECHNOLOGY

WP 4.1: Short-term weather forecasting for shallow landslide prediction

The work of WP 4.1 is presented in detail in the Deliverable D4.2 "*Short-term weather forecasting for prediction of triggering of shallow landslides – Methodology, evaluation of technology and validation at selected test sites*". A model has been developed being a combination of an infinite slope stability calculation with a transient, analytic solution for pore pressure response to steady state and transient rainfall infiltration. A complex operative chain has been set up based on forecasted rainfall, by means of the COSMO-LM model, a hydrological model and a simple infinite-slope stability code. The code has been developed coupling an infinite-slope stability analysis with a three-dimensional analytical solution for transient pore pressure response to steady state and transient rainfall infiltration, allowing a regional slope stability evaluation in a Geographic Information System framework. The numerical weather prediction model used to produce forecast rainfall is COSMO-LM. The Lokal-Modell (LM) is a non-hydrostatic limited-area atmospheric prediction model.

The definition of a prototype tool for early warning of rainfall-induced landslide has been completed. A large number of test cases have been simulated, providing an adequate verification of the tool together with knowledge about its weak and strong points. At the same time research activities for the development of the different simulation codes and of the software linking the different simulation models has been continued by AMRA, CMCC and UNIFI. The implemented tool, that simulates the "hydrometeorological simulation chain", has the goal to evaluate the modification in the slope safety factor (FS) using precipitation forecast coming from numerical weather prediction models. The tools consists of several components: numerical weather prediction (NWP) models, tools for precipitation downscaling techniques realizing the link between atmospherical models and stability analysis models and software codes for the production of safety maps on scales ranging from individual slopes to regional level.

The tool has been tested on the instrumented Cervinara site, the Tuscan region and on the Ischia isle. The choices of the test cases is mainly due to the availability of well documented data in the selected area about soil structure, properties, and initial conditions together with hourly precipitation observations from different in situ stations. The test cases covers very different meteorological situations such as advection rain events, with a long time period, convective rainfall happening in a short time periods and also days in which the rainfall is not intense. With regard to the stability models, the test cases covered various spatial scales and soil with different properties. Generally, quite good agreement was found between the forecast and the observation of the soil conditions after precipitation. Some problems need to be solved such as the effect on the soil of snow melting (increasing of the water availability at the soil level).

<u>WP 4.2: Remote sensing technologies for landslide detection, monitoring and rapid mapping</u>

The WP4.2 "*Remote Sensing technologies for landslide detection, monitoring and rapid mapping*" has the general objective of analyzing the use of remote sensing imagery (Spaceborne radars, Airborne and VHR space borne optical sensors and Airborne geophysics) in landslide studies. In particular the specific objectives are:

- Define and validate a common methodology for detection, rapid mapping, characterization and monitoring of landslides at regional and catchment scales using advanced remote sensing techniques;
- Define and validate a common methodology for the rapid creation and updating of landslide inventories and hazard maps at regional/catchment scale using advanced remote sensing techniques;
- Prepare user-oriented guidelines for the incorporation of advanced within integrated risk management processes and best practices.

The WP deliverables are:

- D4.1 – Review of monitoring and remote sensing methodologies for landslide detection, fast characterisation, rapid mapping and long-term monitoring
- D4.3 – Creation and updating of landslide inventory maps, landslide deformation maps and hazard maps as input for QRA using remote sensing technology
- D4.4 – Guidelines for the selection of appropriate remote sensing technologies for monitoring different types of landslides
- D4.5 – Evaluation report on innovative monitoring and remote sensing methods and future technology (together with WP4.3)

D4.1 – Review of monitoring and remote sensing methodologies for landslide detection, fast characterisation, rapid mapping and long-term monitoring

This review aims at representing a common reference for the different deliverables of SafeLand Area 4. In addition to being a state-of-the art overview, this deliverable provides helpful and extensive support for non-specialists and students interested in the application of new techniques to different mass movements. The core of this deliverable consists of two main chapters (2 and 3), which aim at developing the basic technical knowledge for (a) landslide detection, (b) fast characterization, (c) rapid mapping and (d) long-term monitoring.

D4.3 – Creation and updating of landslide inventory maps, landslide deformation maps and hazard maps as input for qra using remote sensing technology

The deliverable provides a comprehensive view on the latest developments of remote-sensing technologies as applied for the creation and updating of landslide inventory and deformation maps by the members of the SafeLand WP 4.2. Furthermore, chapter 4 gives a broad overview of input datasets for hazard and risk assessment that can be obtained through remote sensing, and in chapter 5 suitable updating strategies as well as steps toward a better linkage between the recent technological developments and QRA methods are discussed.

D4.4 – Guidelines for the selection of appropriate remote sensing technologies for monitoring different types of landslides

This document provides condensed guidelines for the selection of the most suitable remote sensing technologies according to different landslide types, displacement velocities, observational scales and risk management strategies. The main part of the document gives an overview of the capabilities of different techniques to detect, characterize, map and monitor landslides and can be used to initially constrain the choice of methods to a few techniques that seem most feasible for the landslide process at hand. Before final decisions on the methods to be used are taken, further information and expertise will typically be required. Therefore, links to relevant SafeLand project deliverables are provided throughout the text. For further information Annex 1 provides an overview of recent scientific studies that applied the mentioned techniques. Links to relevant database and software tools can be found in Annex 2. This Annex also provides a list of expert institutions that could be consulted for recommendations on observational strategies.

Users of this document should consider that it provides a snapshot of the currently available knowledge and technology. In the near-future, the launch of new satellites, better data access (e.g. Global Monitoring for Environment and Security - GMES), lower data prices and on-going enhancement of processing algorithms, will lead to the maturing of many currently new or experimental techniques into methods suitable for operational use (see also SafeLand deliverable D4.5); at the same time, other traditional methods may become obsolete. In this document we focus mainly on technological and geomorphological aspects. Social aspects (such as preparedness, awareness) are only briefly touched (Chapter 2.5) and we refer to deliverables D5.5-D5.7 where these important aspects are discussed in more detail.

D4.5 – Evaluation report on innovative monitoring and remote sensing methods and future technology

This deliverable is a joint deliverable between WP4.2 and WP4.3 and had the aim of making an evaluation of the most innovative landslide monitoring and remote sensing technologies used at present, as well as suggesting needs for research and technical developments of the existing methodologies. Amongst all the ground based techniques employed in landslide studies, the ones which in recent years showed the most promising improvements were selected and reviewed, emphasizing the recent trends in their development and application and stressing the latest scientific and technological advances. The same approach was pursued with remote sensing techniques, making a clear distinction between the use for detection and mapping and the use for monitoring purposes.

The objectives of the deliverable were achieved through these main steps:

- Overview of recent and emerging ground based techniques for landslide analysis.
- Overview of recent and emerging remote sensing technologies for landslide analysis.

- Questionnaire on landslide monitoring methods. The Questionnaire on National State of Landslide Site Investigation and Monitoring was prepared and was disseminated among European institutes and representatives within the frame of the SafeLand project. The results of the questionnaire were reported and discussed.
- Questionnaire on remote sensing technologies. The aim of the questionnaire was to collect information about the usefulness of remote sensing for landslide study and to evaluate its applicability for landslide detection, mapping, monitoring and early warning. This questionnaire was circulated within and outside Safeland consortium. The results of the questionnaire were reported and discussed.
- A relevant part of the deliverable was focused on the application of these innovative techniques within SafeLand case studies, clearly stating which technical and scientific improvements were achieved for each technique thanks to SafeLand project.
- Evaluation of ground based, airborne and space-borne techniques based on the literature review, on the aforementioned questionnaires and on the results coming from the Safeland case studies.

<u>WP 4.3: Evaluation and development of reliable procedures and technologies for early warning</u>
The activities within WP 4.3 contained three main tasks:

Task 1: Assessment of current state-of-art in monitoring and early warning (technology)
The current state-of-art in monitoring and technology of early warning has been proceeded through:

- Contributing to Deliverable D4.5 (Remote sensing technologies for landslide detection, monitoring and rapid mapping/ Evaluation and development of reliable procedures and technologies for early warning - Responsible: UNIFI/ITC, Delivery month: 24).
- A questionnaire study on “National Mass-movement Investigation and Monitoring”, and “Questionnaire on remote sensing” focused especially on the use and reliability of field investigation, remote sensing, and monitoring techniques for landslides which were presented in deliverables D4.5 and D4.6 (“Report on Evaluation of Mass Movement Indicators”, Delivery date: 32 month).
- A screening study of existing EWS systems worldwide, which has been made for deliverable D4.8 “Guidelines for monitoring and early warning systems in Europe (Delivery date: 32 month).

Task 2: Exploring the role of “geo-indicators” (mass movement parameters) as early warning parameters (processes and related parameters)
The task on role of monitoring and early warning parameters is the main goal of the deliverable D4.6 “*Report on Evaluation of Mass Movement Indicators*”. This task was mostly based on analysis and evaluation of monitoring field data of unstable slopes at SafeLand test sites. The project partners have been collecting the raw monitoring data and provided their analysis from 14 test sites through Europe.

Task 3: Method evaluation and implementation of guidelines for monitoring and early warning.
This task is mostly based on the practical implementation of early warning systems. It is presented in D4.7 and D4.8 and it summarizes the theoretical and practical information to derive general rules to provide effective EW centres worldwide.

The outcome of the work has been reported in four deliverables:

D4.5: Evaluation report on innovative monitoring and remote sensing methods and future technology (together with WP 4.2) (Delivery date: 24 month).
This deliverable is a joint activity together with WP 4.2, see section on WP 4.2 for details.

D4.6: Report on Evaluation of Mass Movement Indicators.
The deliverable "Report on Evaluation of Mass Movement Indicators" focused on physical parameters which could be monitored in relation to landslide triggering processes, and which could potentially be used as early warning parameters of slope instabilities. The first part of the report reviewed potentially available monitoring parameters, including basic definitions, units, typical values, and formulas. Not only the well-established parameters were included, but also those with a potential for a future application as early warning parameters. Based on a questionnaire study, the parameters were evaluated by means of their abundance, reliability and early-warning potential. The third part of the deliverable presented and summarized results from the SafeLand test sites, i.e.: three test sites in Austria, two in France, three in Norway and one test site in Spain. The analysis of each of the monitored parameters was described with a focus on investigating the correlation between each of the parameters. An additional goal was to define their critical values (alerts/thresholds) in relation to the triggering of mass movements and to evaluate their role as an early warning parameter on the background of their geological settings. The monitoring results from the WP 4.3 provide an excellent basis for future research in the field of early warning parameters and thresholds. However the results of the deliverable show that there is a need for long-term monitoring experiments, exceeding the three year period of SafeLand.

D4.7: Report on the development of software for early-warning based on real-time data.
This deliverable described new software specifically developed to support technical staff in data analysis and the decisional process. The programming of the software started in August 2010. The project intended to realize a centralized interface for early warning centres to manage data from different monitoring stations. The proposed software should be a separate and independent tool for real-time analysis and evaluation of geo-scientific monitoring data, including threshold evaluation. The basic concept was to develop software that could integrate and automatically analyze data from a variety of sensors. The integration and predefined analysis and correlation of different sensors would help the user in operative early warning centres to increase the quality of the geo-scientific evaluation. As a result, this report gives a brief description of the application structure and all necessary steps to start up a system. Finally, this report describes the data analysis in one of the test sites included in this part of the project.

D4.8: Guidelines for monitoring and early warning systems in Europe - Design and required technology.
The D4.8 deliverable summarized how landslide early-warning systems should be designed and operated and presented a screening study of existing EWS systems worldwide. The document was elaborated as the last deliverable of area 4 and aimed at facilitating the decision process for stakeholders by providing guidelines. For the purpose of sharing the globally accumulated expertise, a screening study was realized amongst 14 early warning systems. As a result, the report presented a synoptic view of existing monitoring methodologies and early-warning strategies and their applicability for different landslide types, scales and risk management steps. Several comprehensive checklists and toolboxes were also included to support informed decisions.

In parallel to the discussions on the content of the deliverables, several monitoring projects were started / continued at several SafeLand test sites to provide the necessary field data for analysis within WP 4.6 and WP 4.8 in real time.

WORK AREA 5: RISK MANAGEMENT

WP 5.1: Toolbox for landslide hazard and risk mitigation and prevention measures
The WP 5.1 activities include four deliverables, D5.1 to D5.4. A short description of the work done for the four deliverables follows:

Deliverable D5.1: Compendium of tested and innovative structural, non-structural and risk-transfer mitigation measures for different landslide types
A categorisation system for the different structural mitigation and prevention measures was developed and a total of about 60 measures were selected for further documentation and evaluation. The draft report on the web site describes a number of these measures with a brief discussion of the classification of the possible mitigation measures; guidance on the applicability and effectiveness of each mitigation measure considered to different types of landslides; information on the maturity of the technology, which can range from “prototype development” to “obsolete”; information on current design methods, their maturity and associated uncertainties; and comparative (qualitative) information on costs. The measures are evaluated for different types of ground movements and slides. Each measure was then ranked with “scores” and placed into an applicability matrix that will be used later in the toolbox. The report on the compendium is on the SafeLand website. There is still a lot of work remaining for the completion of the compendium. To accelerate the productivity and to enable the workings of the toolbox (deliverable D5.2), a parallel evaluation of the 60 mitigation measures and the non-structural measures are being documented and evaluated at ICG, as part of the D5.2 work. At the conclusion of the SafeLand project, it is recommended that this task be continued even after the SafeLand project is completed to develop an even higher quality product.

Delivery D5.2: Web-based toolbox of structural and non-structural mitigation measures with decision-making guidance
A piece of web-based software was prepared and is now being tested. It includes at the present time only the mitigation measures that were available by 1st November 2010 in the compendium (Deliverable D5.1). The toolbox is implemented for local landslide hazards and includes typical examples. It assists the user with the following: selecting the type of ground movement expected; assessing the level of hazard associated with the ground movements; evaluating the consequences of the ground movement; evaluating the risk class and determining the need for mitigation; and for selecting the most appropriate mitigation approaches to use, and comparing them. The toolbox contains default implementation criteria and “scores” for each mitigation and preventing measure. The user can at any time introduce his own “scores” and “weights” for each of the mitigation measures. In term of non-structural measures, only early working systems are included at this time. The other non-structural measures refer to WP 5.2 (Stakeholder process for risk management) for their evaluation and ranking. The report on the toolbox and the toolbox itself is uploaded on the SafeLand website. In addition to the input of the data for each of the mitigation measures, the toolbox was tested and case examples were included in the toolbox. A disclaimer is now under preparation and will be included before public release of the toolbox.

Delivery D5.3: Quantitative risk-cost-benefit analysis of selected mitigation options for two case studies

Decision making in general is a difficult issue due to the significant underlying uncertainties and complex interrelation of events and choices affecting the benefits and losses associated with decisions. Typical decision problems are subject to a combination of inherent, modelling and statistical uncertainties. This is primarily due to the fact that the understanding of the issues involved in the decision is often incomplete and that the processes of physical phenomena and human interactions can be modelled only in uncertain terms. If all aspects of a decision problem would be known with certainty, the identification of optimal decisions would be straightforward by means of traditional cost-benefit analysis. Due to the existing uncertainties, it is not possible to assess the results of decisions in certain terms. There is hence no way to assess with certainty the consequences resulting from the decisions we make. However, what can be assessed is the risk associated with the different decision alternatives. Based on risk assessments, decision alternatives may then be consistently ranked on the basis of their associated utilities and benefits/losses, thereby providing a rational basis for societal decision making. This report provides a framework and methodology for carrying out a risk-cost-benefit analysis for decision-making. Two case studies applying the proposed methodology – one involving the analysis and management of risks arising from debris flow phenomenon in Barcelonnette, and the other with risk analysis and risk management for risks posed by different flow-like phenomena in Nocera Inferiore are described in the report.

The report concludes that risk assessment and risk management can be seen as an essential and integral part of the decision planning, decision support and decision-making processes. Decision problems in general and especially in natural hazards management are generally subject to a combination of inherent, modelling and statistical uncertainties. What can be assessed is the risk associated with the different decision alternatives. Based on risk assessments, decision alternatives may then be consistently ranked on the basis of their associated utilities (which may be more useful for engineering decision problems) and cost-benefit analyses (which may be relevant for life safety and overall risk management problems), thereby providing a rational basis for societal decision making. The proposed framework for carrying out a risk-cost-benefit analysis for decision making provided convincing results for the two case studies. The usefulness of the Life Quality Index (LQI) approach for the evaluation of the acceptance of the mitigation options with regard to investments into life safety and the evaluation of the optimal risk mitigation alternative was demonstrated.

Delivery D5.4: Quantification of uncertainties in the risk assessment and management process

The consideration and treatment of uncertainties is an essential part of any risk assessment and risk management process. Uncertainties can either be naturally inherent or modelling and statistical related. This deliverable provided a rational basis for the quantification of the different uncertainties existent in the risk assessment and risk management processes. To obtain a complete picture of the issues and aspects concerning the treatment, quantification and management of uncertainties in the risk assessment, risk management and decision making processes, this deliverable report should be read in conjunction with the report of Deliverable D0.3.

The uncertainties were differentiated into aleatory and epistemic uncertainties, primarily for the purpose of setting focus on how uncertainty may be reduced. Focus was directed on the uncertainties in the different models used for the quantification of risk and the characterisation of parameters in the models. Guiding principles and a general basis for the modelling and representation of the underlying uncertainties in the use of these models for the quantification and estimation of risks were provided. A Bayesian approach was advocated for the representation, handling and management of uncertainties in the context of decision making. The deliverable presented an example on the modelling and management of uncertainties associated with rock fall hazards following a Bayesian approach.

The report concluded that a rational and consistent understanding and consideration of uncertainties is vital for any risk assessment and risk management process and for ensuring rational and optimal decision-making. It is hence necessary to think about the nature of the various types of uncertainties, particularly in the context of risk communication. While communicating the results of risk assessments and analyses with the outside world, it is important to distinguish primarily between the objective probabilities related to scatter and uncertainty from a natural origin on the one hand and subjective probability estimates for knowledge (epistemic) uncertainties on the other hand. When considering updating and incorporation of new knowledge, it is important to understand how uncertainties change characteristics as functions of both the point in time where they are looked upon and as functions of the scale of the modelling used to represent them. This also influences the level of detail required for the treatment of uncertainties in any risk assessment and risk management process.

WP 5.2: Stakeholder process for choosing an appropriate set of mitigation and prevention measures

WP 5.2 activities included the deliverables:

D5.5 Five scoping studies of the policy issues, political culture and stakeholder views in the selected case study sites – Description of methodology and comparative synthesis report

This period of work focused on the continuation of efforts set in place during the previous reporting period where project researchers for each of the five case studies were recruited, methodology determined in a London workshop and synergies across the WP explored in a meeting at IIASA. Continuing work led to the completion of data collection for each of the five case study sites and the synthesis of these reports into a final project document and deliverable which was submitted to schedule. Work has also been presented at a WP workshop at the University of Salerno, one paper authored by Scolobig and Sharma has been accepted for publication by Natural Hazards and a second targeted at Global Environmental Change with all researchers as co-authors has been submitted.

D5.6 Development and testing of spatial multi-criteria evaluation for selected case sites

The work done under this deliverable started with a meeting in Vienna in December 2010 in which ITC, IIASA and UNISA agreed to develop and test spatial multi-criteria valuation on the Nocera Inferiore dataset. In June 2011, ITC participated in one of the meetings organized under deliverable 5.7 (see next section) as observer and was given an introduction to the study area to gain better understanding of the local situation. During a second visit to UNISA later in June 2011, the available digital data was structured in a first draft of SMCE in close collaboration with colleagues of UNISA. In the following months this draft SMCE was further refined and several approaches were tested. In November 2011 Prof. Ferlisi of UNISA came to ITC to finalize the SMCE and to discuss its advantages and limitations in comparison to other risk assessment methods. This resulted in the first draft of the deliverable document that was eventually submitted in April 2012. Currently the main contributors are reworking the document into a paper that should be published along with other SafeLand papers.

D5.7 Design and testing: A risk-communication strategy and a participatory process for choosing a set of mitigation and prevention measures

The core research work for this deliverable has been performed during this reporting period. It included the organization of the participatory process (1 public open meeting, 5 meetings with 15 selected residents, evaluation and feedback via questionnaire, informal meetings with local activities, parallel working groups); a questionnaire survey (questionnaire piloting, collection of 373 questionnaires); communication and education activities (setting up of a website, online discussion group, press releases and contacts with local media, simulation exercise with students at the LARAM school organized by UNISA).

The results of this work were synthesized in the deliverable which includes a detailed description of the case study, the methodological approach, and the key results of: the qualitative work, the participatory process, the questionnaire survey and the communication and education activities.

WP6: DEMONATRSTION SITES AND CASE STUDIES FOR VERIFICATION/CALIBRATION OF MODELS AND SCENARIOS

The main objective of WP6 was to document case histories and "hotspots" of European Land-slides (including potentially unstable slopes), and to provide the technical data for the case studies to be used in other work packages in SafeLand, in particular:

- WP1.1 Identification of mechanisms and triggers
- WP1.2 Geomechanical analysis of weather-induced triggering processes
- WP1.3 Statistical analysis of thresholds for precipitation-induced slides
- WP1.5 Verification and calibration of run-out models
- WP2.2 Calibration of models for vulnerability to landslides
- WP4.2 Remote sensing technologies for landslide detection
- WP4.3 Technologies for early warning
- WP5.1 Toolbox for landslide hazard and risk mitigation measures
- WP5.2 Stakeholder processes for choosing appropriate mitigation strategy

As of June 2012, data for 47 potential case study sites were compiled and summarized in deliverable D6.1. These comprise 45 sites in Europe located in Italy, France, Norway, Switzerland, Austria, Andorra, and Romania; as well as one site in Canada and one in India. Almost every type of landslide and every type of movement is represented in these sites.

FRA KVIKKLEIRE TIL DYPFORVITRET BERG
Resultater fra grunnundersøkelser langs ny rv. 23 mellom Linnes og Dagslet i Lier og Røyken kommuner

FROM QUICKCLAY TO DEEPLY WEATHERED ROCK
Results from preinvestigations for new highway (rv. 23 between Linnes and Dagslet in Lier and Røyken municipality

Siv.ing. Jan K.G.Rohde, Sweco Norge AS
Ingeniørgeolog Knut Henrik Skaug, Sweco Norge AS

SAMMENDRAG

Ved utarbeidelse av detalj- og reguleringsplan for rv. 23 fra Linnes i Lier kommune til Dagslet i Røyken kommune er det utført relativt omfattende grunnundersøkelser i form av grunnboringer og prøvetaking, seismisk profilering, kjerneboringer og resistivitetsmålinger. Undersøkelsene har avdekket områder med mulig kvikkleire, artesisk trykk og potensielt dypforvitret berg. Det er likeledes avdekket flere svakhetssoner som krysser tunnelen, blant annet der berget er dekket av marine avsetninger. På bakgrunn av undersøkelsene forventes det omfattende stabiliserings- og setningsreduserende tiltak ved større konstruksjoner i dagsonen og i tunnelen.

SUMMARY

For preparation of the feasibility study for the new rv. 23 from Linnes in Lier municipality to Dagslet in Røyken municipality there is undergone fairly extensive investigations in the form of geotechnical drillings, seismic surveys, core drilling and resistivity surveys. Investigations have revealed areas with possible quick clay, artesian pressure and potentially deeply weathered rock. There is revealed several potential weakness zones which cross the tunnel alignment. The ground conditions will require extensive grouting and rock support in the tunnel plus stabilization measures for foundation of large structures,.

INNLEDNING

Statens vegvesen, Region sør, planlegger ny motorveg (rv. 23) fra Linnes i Lier kommune til Dagslet i Røyken kommune. Til den nye vegstrekningen inngår en ca. 1900 m lang toløps tunnel.

Prosjektet omfatter en dagstrekning over flatt jordbruksareal med kryssing av Lierelva med ny bru og en ca. 300 m lang betongkulvert før påhugg til bergtunnel i vest. Bergtunnelen blir ca. 1900 m lang og får to løp med profil T9,5. I øst skal det støpes en ca. 55 m lang portal før kryssing av Daueruddalen med en ca. 180 m lang bru. Foran kulverten ved Linnes og ved Dagslet i øst skal det etableres planfri kryss med på- og avkjøringsramper. En oversikt for prosjektet er vist på oversiktskart i Figur 1.

Det stod ferdig detalj- og reguleringsplan for prosjektet i 2003. Siden den gang har nye standarder og krav blitt innført. Den gamle planen er nå revidert og oppdatert med justert trase og utforming i henhold til nye standarder, krav og retningslinjer. Arbeidet er utført av Sweco Norge AS.

Figur 1 ***Oversiktskart***

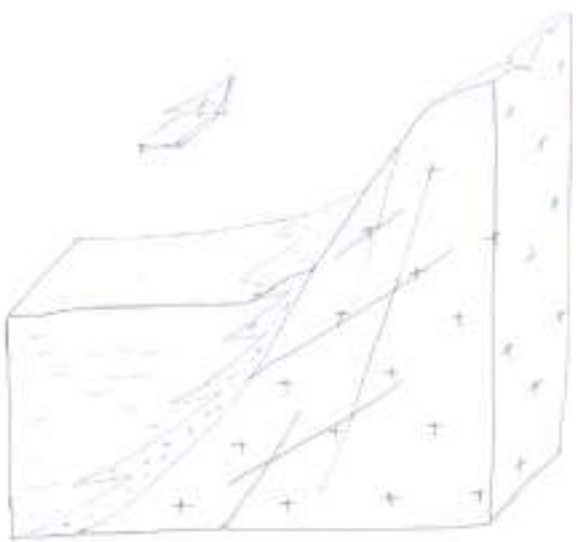
GEOLOGI OG GRUNNFORHOLD

Område Linnes i vest

Området ved Linnes er et flatt deltaområde dannet av Lierelva. Store deler av avsetningene består således av sandige og siltige masser, men det er påvist ved boringer lommer av marin leire, til dels sensitiv og stedvis kvikk øst for Lierelva.

Dannelse av sensitiv leire bør sees i en hydrogeologisk sammenheng. På oversiden (østsiden) av området finnes en rekke kildeutspring og det er også grunn til å anta at en god del vann tilføres det underliggende sandsiktet mellom berggrunn og leire via vann fra sprekker i granitten. Dette medfører artesiske tilstander over store deler av prosjektområdet, noe som bidrar til utluting av elektrolyttinnholdet i de overliggende leirlag (se Figur 2).

21.3



Figur 2 Tolkning av utlutning av elektrolytter i marin avsetning på underliggende sandlag.

Tunneltraseen

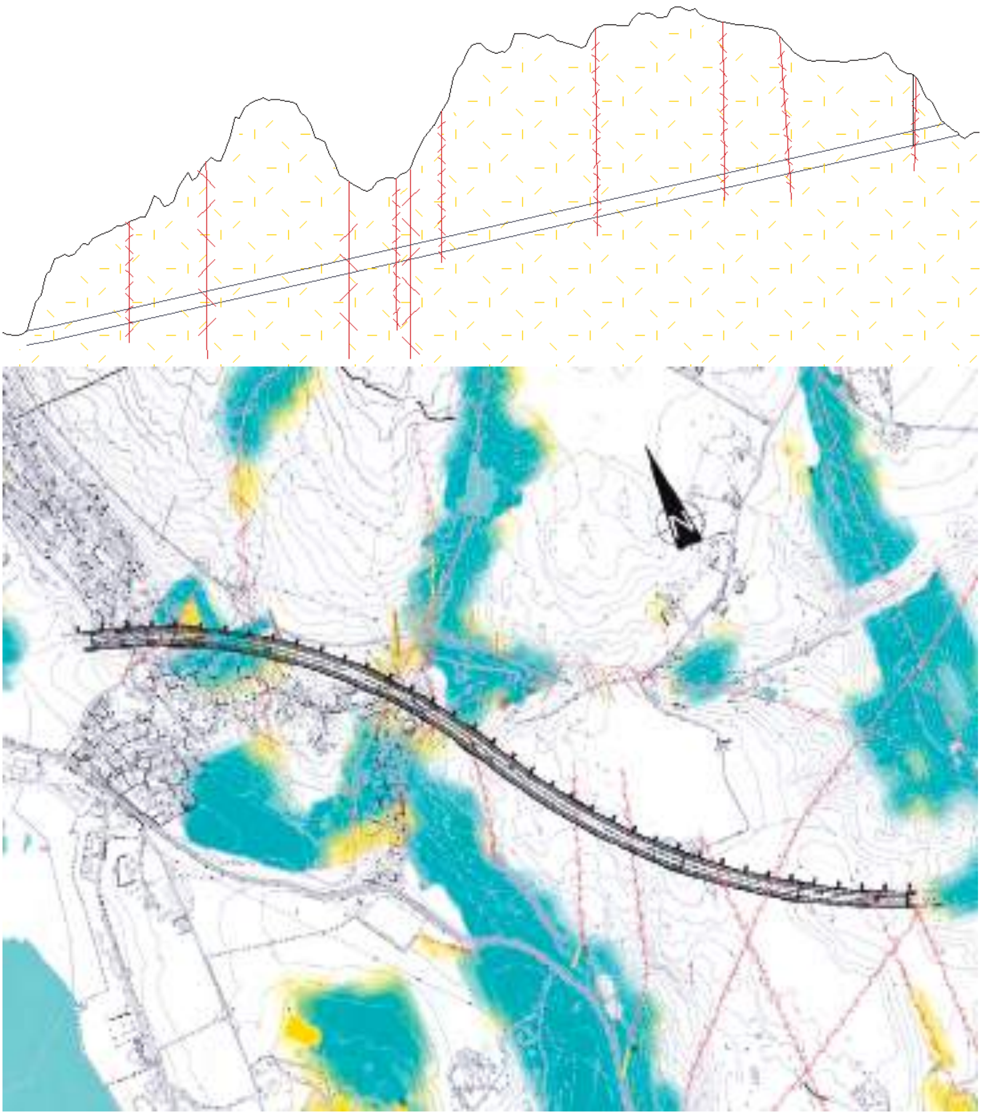
Ved forskjæring og påhuggsområdet ved Linnes, består løsmasseavsetningene dels av urmasser og noe ablasjonsmorene. Forskjæringen ligger nær bebyggelse og i deler av området er det fyllmasser for veg og eiendommer.

Videre langs tunneltraseen mot øst er det noe skog samt jordbruksareal. Løsmassene der er hovedsakelig marin leire og silt. Forskjæring og portalområde i øst ligger i et ravineområde ved Daueruddalen der det er registrert flere kilder.

Berggrunnen langs tunneltraseen består i sin helhet av drammensgranitt som er hyppig gjennomskåret av syenitt- og diabasganger. Langs traseen er det også påvist større og mindre svakhetssoner. Magnetiske målinger fra NGU viser at det sannsynligvis er dannelse av dypforvitring langs de mest markerte sonene [ref.1]. Plan og profil for tunnel med svakhetssoner og dypforvitring er vist i Figur 3.

Område ved Dagslet

Øst for tunnelen krysses Daueruddalen. Daueruddalen er en ravinedal. Daueruddalen krysses i bru før den går gjennom et jordbruksareal og villabebyggelse ved Dagslet. Løsmassene består her av marin leire og silt med varierende og til dels stor dybde til fast berg. I området langs Daueruddalen er det påvist flere vannkilder.



Figur 3 ***Plan og profil for tunnelen med svakhetssoner, eruptive ganger og magnetisk data (dypforvitring). Sannsynlig/mulig dypforvitring med lysblå/gul farge (fra [ref.1]).***

UTFØRTE GRUNNUNDERSØKELSER

I forbindelse med detalj- og reguleringsplanen er det, i tillegg til feltkartlegging, foretatt en rekke undersøkelser i form av boringer i løsmasser, kjerneboringer i berg, seismisk profilering og resistivitetsmålinger. Boringer i løsmasser er i denne fasen utført av Mesta AS, kjerneboringer av Entreprenørservice AS, seismikk av Geophysix AS og resistivitetsmålinger av Ruden Ltd [ref.3]. I tidligere planfaser har det også blitt foretatt seismisk profilering og grunnboringer, da i regi av Statens vegvesen. Tabell 1 gir en oversikt for utførte grunnundersøkelser og lokaliteter for undersøkelsene er vist i Figur 4.

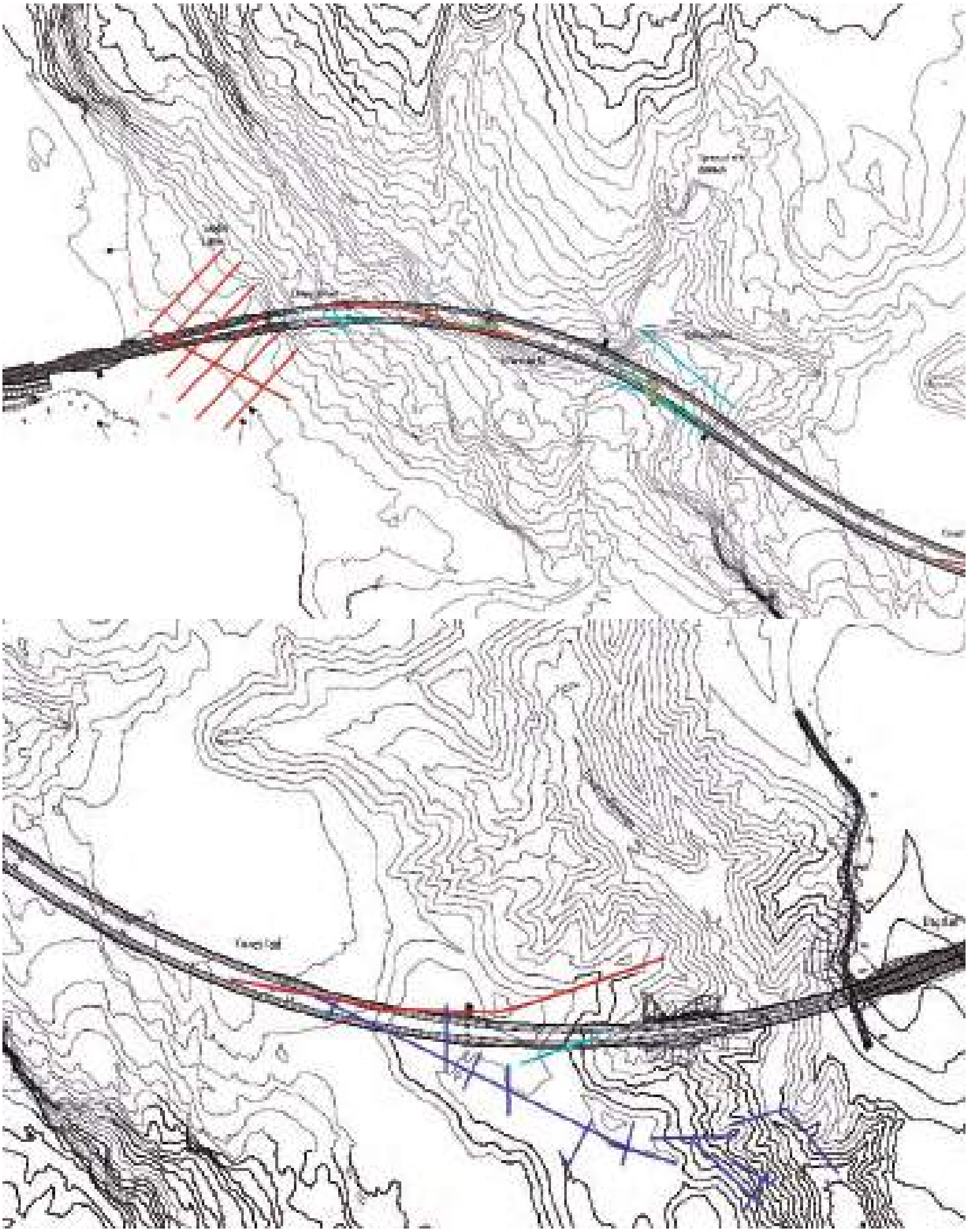
Tabell 1 Utførte grunnundersøkelser

	Antall	Samlet lengde (m)
Refraksjonsseismisk måling (2011)	5 profiler	730
Refraksjonsseismisk måling (1990)	25 profiler	1430
Resistivitetsmåling	9 profiler	2705
Kjerneboring	3 hull	435
Grunnboring		

Første runde med undersøkelser ble foretatt med seismisk profilering for å kartlegge bergoverdekning og antatt større svakhetssoner, og med grunnboringer for å kartlegge løsmasseavsetninger. Boringene som ble foretatt på Linnes påviste sensitiv leire og kvikkleire. De seismiske målingene viste at det var svakhetssoner langs tunneltraseen med stor mektighet. På bakgrunn av disse resultatene, samt antatt dypforvitret berg langs svakhetssoner, ble det igangsatt et relativt stort program med resistivitetsmålinger og kjerneboring.

På grunn av jordbruksarealer og tett bebyggelse har det vært utfordringer med å gjennomføre undersøkelsesprogrammet med resistivitetsmåling og kjerneboring slik som planlagt. Etter tilpasning til de stedlige forhold ble undersøkelsene likevel gjennomført på en tilfredsstillende måte.

21.6



Figur 4 *Oversikt for seismisk profilering (lys blått 2011/mørk blått 1990), resistivitetsmålinger (rødt) og kjerneboringer (grønt)*

RESULTATER FRA RESISTIVITETSMÅLINGER

Resistivitetsmålinger ble utført på grunn av påvist kvikkleire ved Linnesjordet samt løsmasser over tunneltraseen, deriblant marine avsetninger langs den østre del. Resistivitetsmålingene ble utført sammen med IP-måling. IP-effekten som måles kan supplere målingene og gi bedre tolkning av svakhetssoner.

Resistivitetsmetoden gir ofte gode resultater ved påvisning av bruddsoner og ganger der disse er vannførende og det var derfor interessant å få kartlagt hvor berggangene krysser tunnelen og hvor det er større svakhetssoner. Det ble også målt resistivitet på et jorde ved Linnes der det skal bygges en ca. 300 m lang kulvert inn til påhugget. Ved tidligere boringer har det blitt påvist kvikkleire i tre hull og det var derfor av interesse å kartlegge utbredelsen av kvikleiren bedre.

Beskrivelse av resistivitetsmåling
Ved resistivitetsmåling settes to sett med elektroder ned i bakken. Det ene settet fører strøm ned i bakken, mens det andre settet måler resistiviteten i bakken fra det påsatte elektriske feltet. Elektrodene har ulik konfigurasjon avhengig av hvilket formål undersøkelsen har.

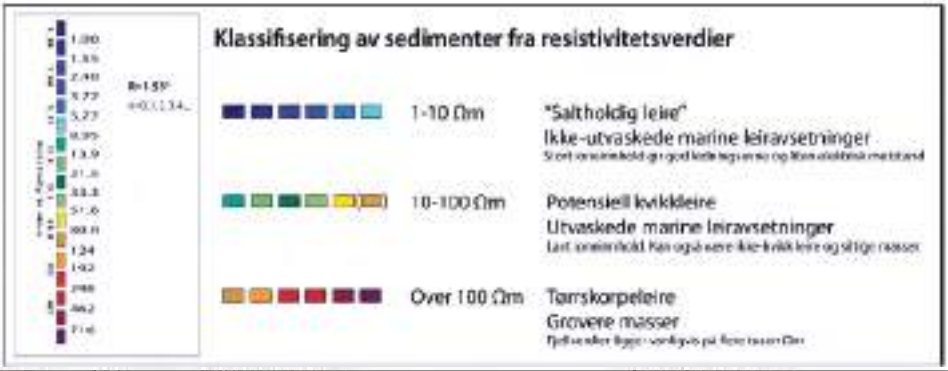
Resistivitetsmålingene gir ofte et tydelig skille mellom ulike typer berggrunn og løsmasser. Følgende resistivitetsverdier (Ωm) er vanlig å bruke som skille mellom ulike materialer [ref.2]:

- Magmatiske bergarter — 1000 - 100 000
- Sedimentære bergarter — 7 - 100 000
- Forvitrede bergarter — 5 - 50 000
- Grus og sand (tørr) — >1400
- Grus og sand (vannmettet) — >100
- Leire — 1 - 100
    - Ikke-utvasket — 1 - 10
    - Utvasket — 10 - 100

Figur 5 viser fargeskalaen som er brukt på resistivitet- og ip-profilene.

Utvasket og sensitiv leire har verdier i området 1-100 Ωm. Verdier innenfor dette intervallet trenger derimot ikke indikere kvikk leire. Både utvasket, ikke-kvikk leire, leirig morene og siltige sedimenter kan ligge i dette området. Dette skaper derfor utfordringer ved tolkning da man ikke alltid kan si noe konkret utifra resultatene fra målingen.

Resistivitetsmålingene har blitt kombinert med måling av ip-effekt. IP-effekten måler restspenning i bakken etter at strømmen er slått av. IP-effekten som måles kan supplere resistivitetsmålingene og gi bedre tolkning av svakhetssoner og leirtyper.



Fra	Til	Formasjon	IP fargekode
0.00	0.10	Meget sensitiv leire	Blått
0.10	0.25	Mulig sensitiv leire	Grønt
0.25	0.40	Kohesiv leire	Gult - rødt

Figur 5 Fargeskala på de inverterte resistivitets- og ip-profiler (fra [ref.2])

Resistivitets- og IP-profilering ved dagsone Linnes

Profileringen ved Linnes hadde som formål å kartlegge mulige kvikkleireforekomster på et jorde hvor det skal bygges en 300 m lang kulvert. Jordet har et areal på ca. 66 000 m2 og store deler av matjorden skal graves opp og fjernes der kulverten skal plasseres. Det er derfor av stor interesse å kartlegge omfanget av kvikkleire i dette området for planlegging av grunnarbeidene.

Det er utført en betydelig mengde geotekniske boringer på jordet tidligere som i hovedsak viser ikke-sensitiv leire som i stor grad er siltig. Ved tre boringer, boret på hver sin side av jordet, er det derimot påvist kvikkleire.

Generelt gjelder følgende forhold ved tolkning av formasjonene ved Linnes:
- **Marin leire** har generelt svært lav resistivitet og høy IP-effekt.
- **Sensitiv leire** har middels lav resistivitet og lav IP-effekt.
- **Berggrunn og sand med ferskt porevann** har svært høy resistivitet og lav IP-effekt.
- **Sand med salt porevann** har lav resistivitet og svært lav IP-effekt.
- **Mineralisert berggrunn** (grafitt, sulfider) kan ha høy resistivitet og høy IP effekt.

Det ble målt resistivitet langs syv profiler langsetter jordet, se Figur 6. Profiler er vist i Figur 7 og Figur 8. Resistivitetsprofilene har nyanser av fargene grønn, gul og rød/Lilla. Rød/Lilla farge viser til de høyeste resistivitetsverdiene og indikerer berggrunnen. Gul farge viser til lave resistivitetsverdier som er høyere en ca 70 Ωm og grønn farge viser til verdier i området ca. 20-70 Ωm.

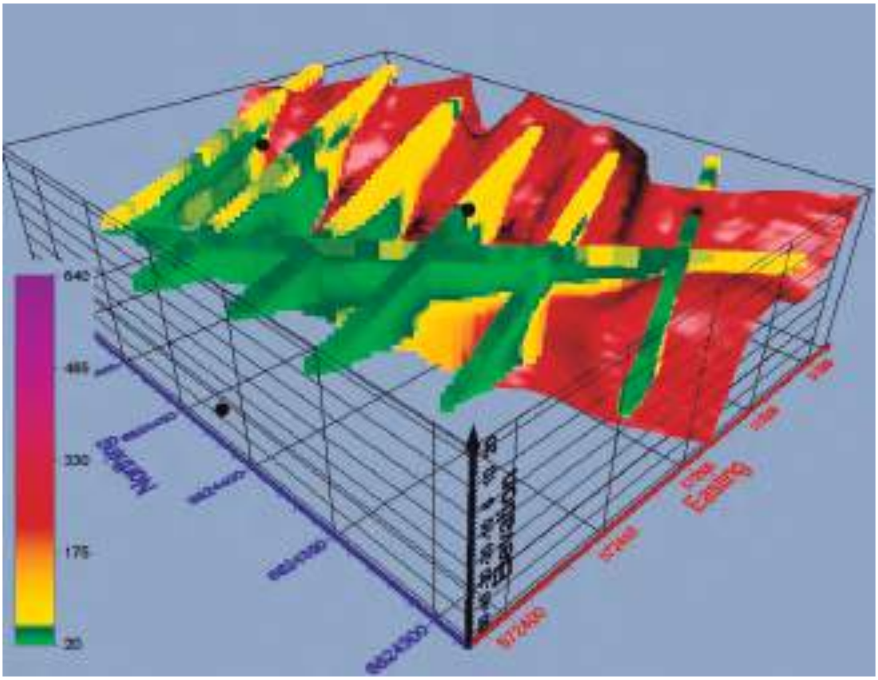
Målingene bekrefter at øvre lag på jordet i stor grad består av leireavsetninger. I tilegg kan det ses høyere verdier i bunnen av profilene som tolkes som sand/grus. Den underliggende berggrunn trer tydelig frem på både IP og resistivitetsprofiler. Et evt. (sannsynlig) ferskvannsførende sandlag over granitten kan ikke sees med disse metodene grunnet beskjeden mektighet og liten kontrast.

Basert på signaturene fra kombinerte resistivitets- og IP-målinger virker en overveiende del av de undersøkte leirmassene å være gjennomgående sensitive, men i varierende grad. Dette skyldes sannsynligvis forskjellige grader av utluting, noe som fremgår av resistivitetsverdier mellom 20-100 Ωm (grønt – gult). Det samme sees på IP profilene, hvor det samme indikeres ved IP-verdier mellom 0,0–0,2 mV/V (blått – grønt).

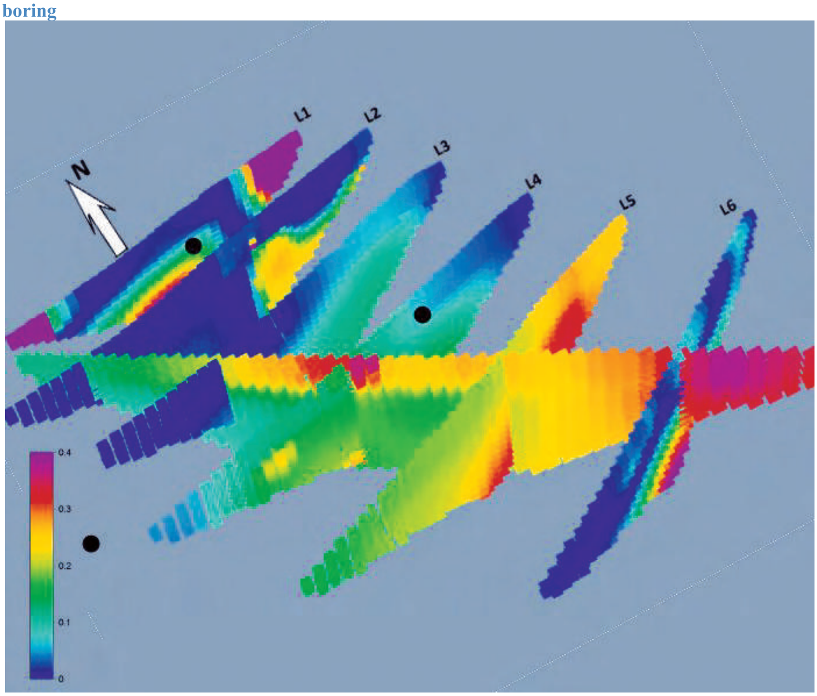
Det må noteres at hele prosjektområdet Linnes har vært gjenstand for omfattende bakkeplanering for ca 40 år siden. Den opprinnelige tørrskorpeleiren samt opprinnelig ravinering fra kilder er derved ødelagt.

Et stort antall boringer på jordet har derimot ikke påvist kvikkleire, bortsett fra ved tre boringer som er indikert med blått i Figur 6. Boringene tyder på at kvikkleiren ligger som lommer på noen meters dyp. Det er usikkert hvor stor utbredelse kvikkleirelommene har og det er ikke entydig samsvar mellom boringer og de geofysiske målingene.

Figur 6 Resistivitetsprofiler ved Linnes gård. Blå prikker indikerer påvist kvikkleire ved boringer.



Figur 7 2.5D-fremstilling av resistivitetsprofil nr. 1-7 ved Linnes gård. Sorte prikker indikerer påvist kvikkleire ved boring



Figur 8 2.5D-fremstilling av IP-profiler nr. 1-7 ved Linnes gård. Sorte prikker indikerer påvist kvikkleire ved boring

Resistivitets- og IP-profilering langs vestre del av tunnel

Det var planlagt å måle resistivitet langs nær hele tunnelraséen, men på grunn av utfordrende topografiske forhold og kabler i bakken ble det ikke gjennomført måling på midtre del av traséen. Midtre del av traséen har tidligere vært undersøkt med seismikk som ikke har samme begrensninger med hensyn til kabler og ledninger i bakken.

Resultater fra måling ved vestre del av traséen er vist i Figur 9. Både resistivitet og IP profil indikerer en overliggende mantel på ca 20 m. Signaturene kan indikere jordfylte sprekkinnfyllinger, med mye klastisk materiale. Dette laget er preget av mye støy fra antropogenetisk aktivitet. Berggrunnen under toppskiktet har imidlertid helt andre signaturer. Profilet kan deles i tre segmenter, atskilt av 2 svakhetssoner merket hhv. A og B. Profilet starter fra NV.

Segment 1 (venstre del av profilet) har en resistivitetssignatur som indikerer massivt berg. Imidlertid indikerer IP målingene en formasjon med til dels meget høye IP verdier. Vanligvis skyldes høye IP verdier elektrodialyse av leirpartikler. Da dette åpenbart dreier seg om en konsolidert og massiv bergart kan IP-verdiene skyldes elektrokinetisk effekt, av uvisse årsaker, og en sannsynlig konklusjon vil da være: massiv granitt, med ukjent mineralisering.

Segment 2 og 3 indikerer den samme 20 m mantelen. Underliggende formasjoner synes også her å være massive bergarter, med høye resistivitetsverdier og med "normale" IP-verdier. Høyre del av profilet har en mer ujevn fordeling av resistivitetsverdier og dette tyder på mer sprekker. Ved enden av profilet reduseres imidlertid undersøkelsesdypet.

Sone A indikerer en vertikal bergartsgrense, sannsynligvis assosiert med en sprekksone, mens IP målinger indikerer lite leirmineralisering. Sonen sammenfaller med en lavhastighetssone påvist ved seismikk ca. 15 m lengre mot SV.

Sone B indikerer en betydelig sprekkesone. På bakgrunn av topografiske forhold er det tidligere antatt at denne sonen går gjennom profilet ved rød pil, men resistivitetsprofilet viser at sonen ligger lengre mot vest. Mektigheten er ca 25 - 35m, og sonen er vertikal, som indikert ved rektangel "B". Imidlertid indikerer IP målinger en betydelig leirmineralisering mot dypet. Sonen er derfor tolket som en nær vertikal, leirmineralisert sprekksone. Magnetiske målinger [ref.1] viser at det sannsynligvis er dypforvitring langs forløpet til denne sonen og kjerneboring som er gjort gjennom sonen over tunneltraséen bekrefter at det er tett oppsprukket og leiromvandlet berggrunn på samme sted som svakhetssonen er indikert på resistivitetsprofilet.

Resistivitets- og IP-profilering langs østre del av tunnel

Opprinnelig skulle målingen starte ved sørøstlige ende av det foregående profil. Imidlertid viste det seg at både topografi, potensiell elektrisk støy (høyspent i bakken) og adkomst i løpet av de første ca 700 m av profilet gjorde elektriske målinger i dette terrenget urealistiske. Profilet ble derfor kortet ned til et ca 630 m profil, som vist på Figur 4.

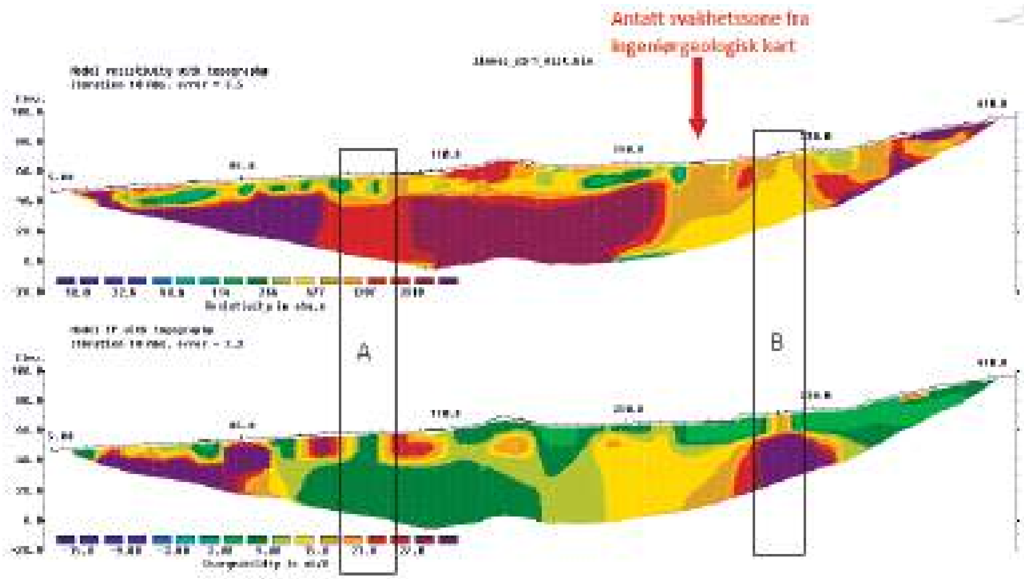
21.12

Resultater fra måling ved østre del av traséen er vist i Figur 10Figur 9. Ved ca. 400 m er det indikasjoner på en sone med lav resistivitet. Resistivitetsanomalien følges av en tilsvarende økning av IP-verdier ved ca. 360 m, dvs. ca 40 m lengre mot SØ, og med et tilsynelatende fall mot NØ. Anomalien indikerer en leiromvandlet knusningssone. Sant strøk og fall kan ikke beregnes ut fra et 2D profil, men sannsynligvis er strøkorienteringen i N-NNØ retning, og fallet er mot vest.

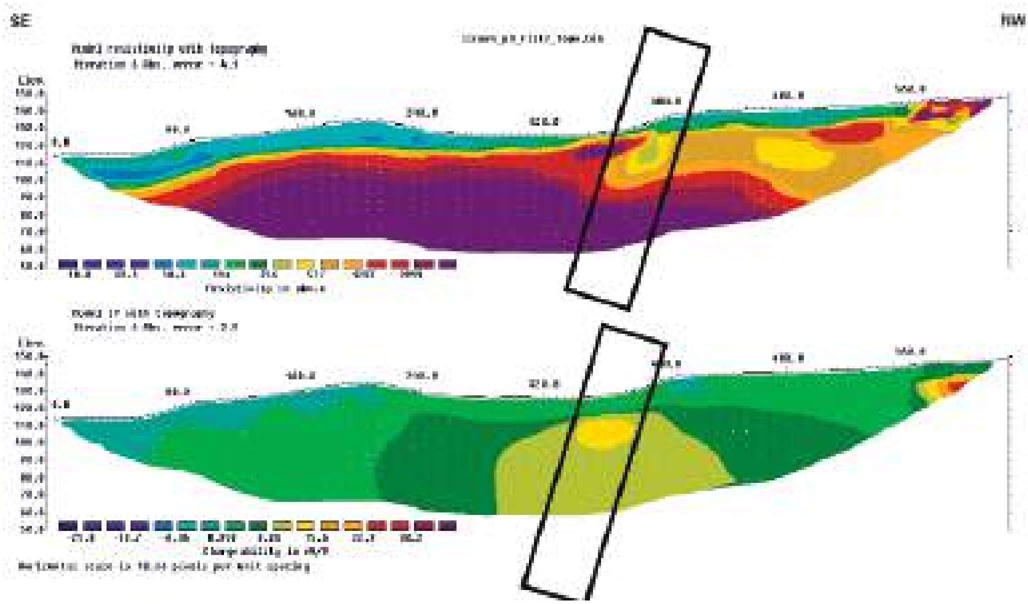
Venstre del av profilet indikerer massivt berg uten synlige tegn til svakhetssoner. Dette samsvarer med seismiske undersøkelser langs profiler lengre mot sør.

Høyre del av profilet viser generelt lave resistivitetsverdier som er langt lavere enn det som forventes i denne typen bergart (granitt). Dette tyder på at berggrunnen her er mer oppsprukket enn lengre øst.

De kartlagte berggangene som forventes å gå gjennom området hvor profilet er lagt er ikke påvist ved målingene, med unntak av anomalien ved ca. 400 m som kan være en svakhetssone/berggang. Det kan være at berggangene i området er lite vannførende og derfor ikke blir registrert med resistivitetsmålingen.



Figur 9 Resistivitets- og ip-profil for vestlige del av tunneltraséen (resistivitetsprofil øverst)



Figur 10 Resistivitets- og ip-profil for østlige del av tunneltraséen (resistivitetsprofil øverst)

SLUTTBEMERKNINGER

Grunnundersøkelsene langs ny rv23 fra Linnes til Dagslett har avdekket grunnforhold som vil kreve til dels omfattende tette-, stabilitets- og sikringstiltak.

Før bygging av kulvert ved Linnes, er det pr. i dag lagt opp til grunnforsterkning med kalkstabilisering samt forbelastning med vertikaldren for å fremskynde setningsforløpet.

I bergtunnelen er det antatt at tung sikring med forbolting foran stuff og armerte sprøytebetongbuer vil være nødvendig ved kryssing av dypforvitrede og leirinfiserte svakhetssoner. Sålestøp vil være aktuelt. Det er også forventet vannførende sprekker og soner i berget. For å unngå utilsiktet grunnvannsenkning, er det satt et lekkasjekrav på maksimalt 10 l/min pr. 100m pr. tunnelløp, og det er pr. i dag lagt opp til omfattende injeksjon langs deler av strekningen.

Tilkomst til brufundament i Daueruddalen er planlagt ned dalsidene. Dette vil bli en stor geoteknisk utfordring, da skråningene ligger nær naturlig rasvinkel. Det er også regulert for adkomst langs dalbunnen, men denne løsningen ønskes unngått på grunn av ravinedalens naturlige landskap og biotop.

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Ph. D. Kristin Hilde Holmøy, SINTEF
Leder av Norsk Bergmekanikkgruppe

LEDERENS 10 MINUTTER

Innlegget gitt muntlig på konferansen uten utgivelse av skriftlig referat.

FJELLSPRENGNINGSTEKNIKK

BERGMEKANIKK/GEOTEKNIKK 2012

INJEKSJON OG FRAKTURERING PÅ SOKKELEN

Injection and Fracturing on the Continental Shelf

Therese Scheldt, Statoil ASA
Jamie S. Andrews, Statoil ASA

SUMMARY

Injection and fracturing processes may be important for increasing oil- and gas production, or reducing harmful discharge to the environment. However, there are many factors to consider to ensure that injection is performed safely without -unwanted fracture growth into non target formations. Rock mechanics has an important role in both the planning and the operation of injection wells. Possible leakage routes, the injection process and data acquisition, fracture growth, change of in situ stresses due to cooling and heating, injectivity and water quality will be discussed. Finally, some examples will be presented.

SAMMENDRAG

Injeksjon og frakturering kan brukes for å øke olje- og gass produksjonen, eller for å redusere utslipp av skadelig utslipp til miljøet. Det er imidlertid mange faktorer å ta hensyn til for at vi skal oppnå sikker injeksjon uten oppsprekking utenfor target formasjoner. Bergmekanikk har en sentral rolle både i forbindelse med planlegging og styring i injektorer. Mulige lekkasjeruter, selve injeksjonsprosessen og datainnsamling, sprekkvekst, endring av in situ spenninger på grunn av kjøling eller oppvarming, injektivitet og vannkvalitet vil bli diskutert. Avslutningsvis vil noen eksempler bli presentert.

INTRODUCTION

Oil- and gas production may be enhanced by injecting water or gas into the reservoir; Injection fluids are pumped into the reservoir through dedicated injection wells which are placed to optimize the flooding processes that drive hydrocarbons to the producers and maintain reservoir pressure. In other cases, the primary purpose of the injection may be to reduce harmful discharge to the environment by subsurface waste disposal which include slop injection and cuttings re-injection.

The risk of out of zone injection shall be evaluated for all types of injection wells. Statoil's governing documentation require that a rock mechanical study is performed in order to analyze the potential for out of zone injection for all projects where injection processes are used for reservoir management, waste management or geological sequestration.

POSSIBLE LEKEAGES ROUTES

Failed well integrity can lead to leakage of injection fluids out of planned injection zone. Good cement quality/hydraulic isolation, deep set production packer and installation of two mechanical barriers will however minimize the probability of leakage.

Naturally existing geological structures such as faults, fractures and vertical injection structures may represent zones of weakness. To minimize the risk of reactivation and leakage along these features they need to be mapped both in the reservoir and in the overburden on the best possible datasets. Small scale features may however always exist below data resolution and this risk needs to be properly communicated. The risk can be mitigated by good monitoring and understanding of what effects to look for. Large scale clinoforms and channel erosions may also represent geo hazards by representing natural leakage paths. Again proper mapping on the best available datasets will minimize the risk.

For injection processes that require fracture propagation, the uncontrolled vertical propagation of a fracture through the cap rock is a third possible leakage route.

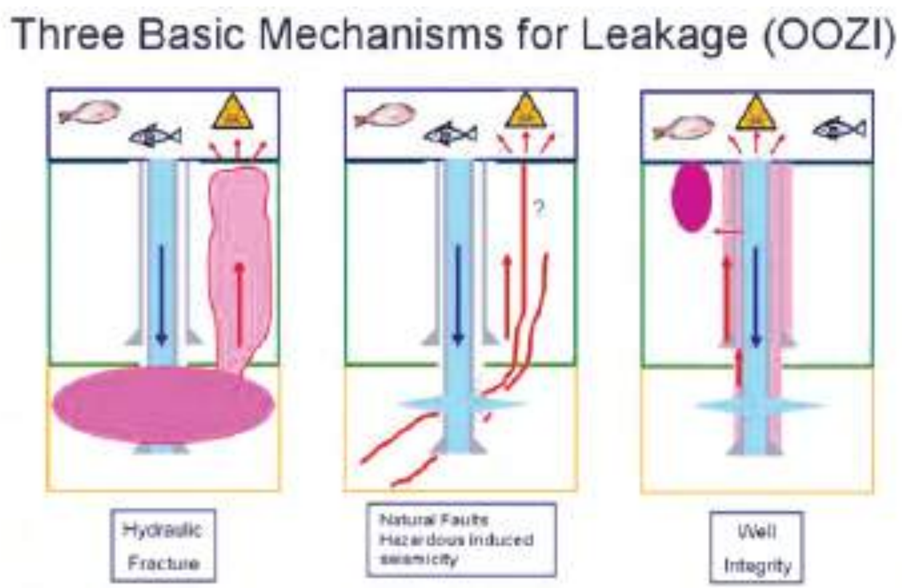

Fig.1 Basic mechanisms for leakage

INJECTION PROCESSES AND DATA AQUSITION

Fracture growth is primarily governed by the *in situ* stress profile, fluid leak-off properties and (to a lesser extent) the stiffness contrasts in the subsurface. Particle content in the injection fluid is also an important factor in determining the rate of fracture growth (since it will change the fluid leak-off characteristics) together with the injection rate.

Out of target fracture growth is potentially hazardous since it may result in breaching to surface. The difference in pressure gradient of the injected fluid (i.e.1 bar/10m for water) and the horizontal stress (typically 1.6 bar/10m) means that once a fracture has obtained a certain height it will have sufficient force to break through any local stress barriers and this can then lead to rapid migration upwards (like a bubble moving up through a denser liquid). Injection leads to increased pore pressure and less safety margin against shear failure in the formation.

The most typical weakness is fault zones and fault reactivation is a likely damaging mechanism for injectors both inside and outside the reservoir. A local zone of abnormally high pore pressure around the wellbore could pose a threat of casing shear and problems during future drilling operations in the area.

Typical data acquisition required for specific injection projects may consist of:

- Target zone injectivity analysis and/or MDT injectivity tests. This can be important when no suitable overburden tests of core material have been performed to calibrate synthetic permeability models based on log responses.
- Coring in order to confirm mechanical and flow properties of the injection formation. Coring of the cap rock and overburden should also be considered.
- Information on stress directions and possible stress magnitudes may be possible from image logs and sonic log data.
- Shallow, high definition seismic
- Seabed topographical surveys
- Compatibility data of injection stream and cement quality for well integrity purposes

FRACTURE GROWTH

In the vast majority of cases, water injection at depth creates a vertical fracture that will be aligned in the direction of the major horizontal stress (perpendicular to σ_3). The length of the fracture will depend on the properties of the rock, the injection rate, the quality of the injected water (plugging) etc. Typically, the length may be from a few meters to several tens of meters or even hundreds of meters. Fracture width is narrow and may be from a few mm to few cm thick.

An open fracture connected to the well will greatly enhance the injectivity. However, fracturing can result in uneven injection profiles if only parts of the targeted formations are fractured. Conversely, if the fracture grows out of zone, this can result in poor sweep in the desired formation.

In the absence of flow restrictions, the water front will be elliptic, with the long axis along the fracture, i.e. pointing in the direction of the major horizontal stress. In practice faults and other flow restrictions will of course influence the shape of the flooded area significantly. The overall reservoir sweep pattern from the injector to the producers can also influence the geometry of the water front and its relation to the in-situ stress field.

Fracture growth is often episodic, and may continue throughout an injector's life. As the fracture faces get plugged, pressure at the tip may build up, resulting in fracture growth episodes exposing fresh formation.

For particle rich injection streams such as crushed cutting slurries in CRI, injection can't be maintained by radial matrix flow and fracturing is a prerequisite for continued injection capacity.

WATER INJECTION AND IN SITU STRESSES

Water injection changes the *in situ* stress by two mechanisms: Cooling/heating and reservoir pressure build-up around the injector.

When a cold fluid is injected into a formation, the temperature of the formation and the fluid will be equalized more or less immediately due to the smallness of the pores. This means that the formation around the wellbore will be cooled to the temperature of the injection fluid, while the fluid outside a given distance will have the original formation temperature.

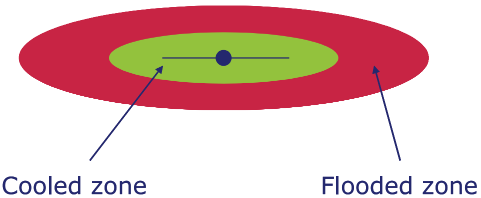

Fig.2 Extent of cooled/flooded zone

The cooling front will be initially sharp, but will more spread-out after prolonged injection due to heat exchange with the surrounding layers. The cooling front will lag significantly behind the water saturation front. The position of the cooling front may be estimated from the heat capacities of the injection fluid and the formation and the formation porosity. It is found that the cooling front typically propagates at 30–50 % of the speed of the flooding front for water injection.

Note that the region of significant stress reduction due to cooling is thus significantly smaller than the flooded zone. In fact, the stress field in the entire reservoir and the overburden / side burden will be influenced by such injection and a full geomechanical analysis is required to model this completely.

The same arguments apply for the case of "warm" water injection but where the local stresses can increase as a result of injection.

The magnitude of the stress reduction due to cooling depends on the amount of cooling, the geometry of the cooled zone and the properties of the rock, mainly the elastic stiffness. As a rule of thumb, a stress reduction of between 0.2 and 3 bar per degree cooling may be expected. The effect is more pronounced in deeper, stiffer rock than in shallower, softer rock. Thus in a deep reservoir, a stress reduction of several tens of bar may easily occur. This is a significant change of the original stress field. The same arguments apply for cases of warm water injection but, in this case, the increasing temperature will lead to an increase in local stress.

Since the cooled zone normally extends farther than the fracture, the entire fracture sees the reduced stress, and hence the fracture will tend to be confined to the cooled layer. (A significant pressure build-up around the well may counteract this effect.) Gradual "erosion" of such thermally induced stress contrasts will occur due to conductive heat exchange to bounding (and not necessarily permeable) layers.

The confining effect of the cooled zone may be utilized, especially during initial fracturing. If height confinement is desired a slow start is beneficial, while a quick start may aid height growth in cases where that is wanted.

If the fluid has a higher temperature than the formation the fracture will see an increased stress. This is the opposite effect to that of aided confinement in a cooled layer. In this case the risk of out of target injection and fracturing to surface is increased. Heating is especially relevant when injecting produced water.

Depending on the injection rate and the permeability and volume of the formation, a significant pressure increase may occur in the near well area. This may reduce the height confinement effect of cooling. Further, the amount of pore pressure build up must be taken into account if a step rate test is used to estimate the *in situ* stress. Note that fracturing may be hindered in cases where the increased back stress from the pressure inflation due to injection in isolated sand bodies is significant. The change in the horizontal stress is typically between 0.4 to 0.7 bar for every 1 bar change in reservoir pressure.

Cooling of the formation during injection is a relatively fast process: The cooling front propagates at 30–50 % of the speed of the water front. Reheating, on the other hand, takes place by heat diffusion from over- and underlying layers, and is a much slower process: The heat "footprint" from the injection will remain in the reservoir for several thousands of years but the heat profile is gradually spread-out over an ever (slowly) growing volume. It will take several years before a 10 m thick cooled formation approaches initial temperature after the injection is stopped.

Thus drilling into a zone flooded several years earlier, one may still find significantly reduced stress with consequences for drilling. For infill drilling, it is important to assess the possible consequences of low-stress zones due to previous water injection.

INJECTIVITY

The performance of a water injector should be regularly assessed by step-rate tests, i.e. by plotting the pressure versus rate as indicated in the figure.

Typically, two distinct regions will be seen, matrix injection and fracture injection. The crossing of the two lines is an indication of the local minimum stress near the fracture.

As the name tells, a step-rate test is in principle performed by letting the rate and pressure stabilize at some pre-selected values.

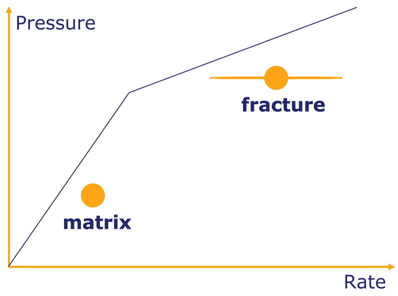

Fig. 3 Injectivity as function of injection mode

However, it may be more convenient to change the choke continuously using a pre-selected bean-up and bean-down rate. This may be easier for the offshore operators, and gives satisfactory data. Note however, that depending on the tendency for pressure build-up around the fracture, the actual slopes of the lines will depend somewhat on the speed of rate change chosen. Due to low fracture pressure and flow meters operating out of range, it may be difficult to sample pressure and rate data for the matrix injection part of the curve. In such cases more steps in the step rate test at the low rate part of the curve should be evaluated.

The response curves are expected to change over time as a result of change in stress due to cooling or pore pressure build-up, due to plugging etc. A few basic signatures are discussed below (the original situation is the dark blue lines, the new situation the dark red lines):

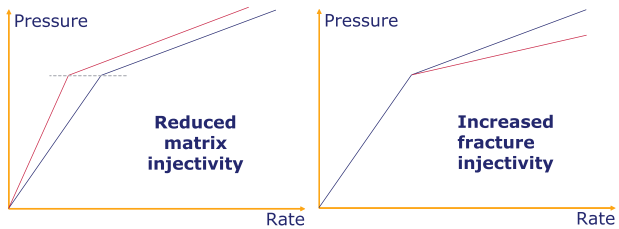

Fig. 4 Injectivity interpretations

Fig.4 (left) shows the effects of a decrease in the matrix injectivity (for example due to plugging), while everything else is unchanged. Note how the pressure at the crossing of the lines, indicating the stress level, is unchanged.

Fig 4(right) shows the result of an increased fracture injectivity.

Fig. 5 (left) shows the effect of a stress decrease due to cooling. The slopes of the lines are unchanged, but the intercept has moved to the left, indicating a reduced stress around the fracture.

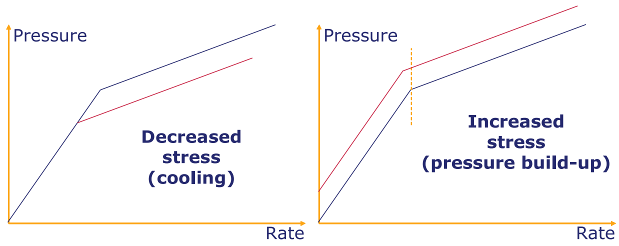

Fig. 5 Injectivity interpretations

Fig 5 (right) shows the effect of a pore pressure build-up. The fracture line moves the right because of the change in *in situ* stress, while the matrix line is translated upwards (keeping same gradient) since injection is now against a higher pressure. Note how the intercept occurs at a lower rate.

The fracturing simulator **StimPlan** has in the last few years been enhanced with a module taking into account the thermal, poroelastic and plugging effects during long term injection. This is a tool tool that requires an expert user.

For a quick analysis of the thermal stress changes around water injectors, a spreadsheet program can be used. The temperature profiles are determined from energy balance calculations, allowing for heat loss into the surrounding formations. The stress changes are quantified using the theory of Perkins and Gonzales. This tool gives quick results for risk evaluation for drilling into cooled zones, especially in the case of sidetracking an old water injector.

In offshore operations, produced water is substantially hotter than injected seawater. Normal static reservoir temperatures vary between 70 ^{o}C (Gullfaks), 90 ^{o}C (Statfjord) and 120 ^{o}C (Veslefrikk).

Some cooling may take place during processing, but produced water will under all circumstances be less viscous than seawater. For injection purposes, this reduced viscosity will directly influence mobility, and it will have a positive effect on injectivity.

Since matrix permeability is constant, the mobility ratio for the two temperatures is simply given by the viscosity ratio according to *eq. 1* as follows:

$$\text{Mobility ratio} = m_{40^0C} / m_{20^0C} = (k/\mu_{40^0C}) / (k/\mu_{20^0C}) = (k\mu_{20^0C}) / (k\mu_{40^0C}) = \boldsymbol{\mu_{20^0C} / \mu_{40^0C}} \qquad (1)$$

m – mobility
k – permeability (mD)
μ - viscosity (cp)

For instance, increasing the temperature from 20 ^{0}C to 40 ^{0}C will lead to an improved matrix Injectivity-Index by a factor of 1.4. This statement assumes matrix injectivity and that no damage is occurring during PWRI. Of course, under matrix injection, PWRI injectors will plug faster than SW injectors and will generally, over time, have higher skin values which will counteract the viscosity effect.

For injection above fracture pressure, the effect of temperature dependent viscosity on injectivity is more subtle. If the PWRI-period is preceded by a period of seawater injection where significant cooling has already occurred and has resulted in thermally aided fracturing, it is likely that the initial response of changing to PW will be an improved injectivity due to the viscosity effect. However; for continued PW injection where the cooling effect is gradually negated it is likely that PW injectivity will become lower than the preceding SW injectivity. In cases where no preceding thermal fracturing has occurred, but where fractured injection is required, it is likely that the PW injectivity will be lower than that if compared to seawater injection.

EFFECT OF COOLING/REHEATING ON FRACTURING PRESSURE

Seawater injection will reduce formation stress, due to shrinking of the formation rock. Reduced temperature will result in a reduced fracturing pressure, thereby reducing the surface injection pressure at a given rate (thermoelastic effect).

The Perkins-Gonzalez paper gives a discussion of thermoelastic stress changes around a fractured well. The cooled zone is approximated to an ellipsoidal shape in the horizontal plane, and with a finite vertical height.

The extent by which formation stress is reduced is determined by rock properties, in particular the Young's modulus. Where the Young's modulus of the rock is low, loss of injectivity due to reduced water viscosity may outweigh the effect of stress reduction.

In nearly all cases, a higher water temperature will be disadvantageous with respect to fracturing pressure. Typically, calculated fracturing pressure will increase by 0.2–3 bar per degree temperature change. Increasing the water temperature by 10 ^{o}C will then increase theoretical fracturing pressure by 2–30 bar.

EFFECT OF WATER QUALITY

Unlike seawater, the produced water contains a significant quantity of suspended inorganic solids and dispersed oil droplets. As long as injection is maintained above fracture pressure, the inorganic solids and oil would in most cases be expected to cause a reduction, but not a severe decline in injectivity.

On the rock surface, particle retention will relatively quickly result in formation of a filter cake, upon which a strong reduction in observed permeability will be seen. This is the main reason why injecting in a fractured mode is important when particle content increases.

Although injecting above fracturing pressure, it is known that fracture face plugging will eventually lead to fracture growth. Fracture growth is then necessary to compensate for reduced leakoff. The geometries of hydraulically induced fractures during PWRI are calculated to be of considerable wing length (50–100 m or even more). Strong/stiff formations (high Young’s modulus) will generally give long fractures with a small fracture width.

EXAMPLES

Tordis Incident 2008
http://www.npd.no/Global/Engelsk/3%20-%20Publications/Norwegian%20Continental%20Shelf/PDF/10%20faulty%20geology.pdf

Veslefrikk leakage 2009
http://www.ptil.no/getfile.php/Tilsyn%20p%C3%A5%20nettet/Granskinger/Rapport%20veslefrikk%20lekkasje%20fra%20injeksjonsbroenn.pdf

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FJELLSPRENGNINGSTEKNIKK
BERGMEKANIKK/GEOTEKNIKK 2012

ANALYSE AV SKRÅNINGSSTABILITET I BJØRNEVATNBRUDDET I SYDVARANGER

Analysis of Slope Stability in the Bjørnevatn Open Pit in Sydvaranger

Siv.ing Ida Soon Brøther Bergh, SINTEF Byggforsk
Sjefforsker/Professor II Eivind Grøv, SINTEF Byggforsk/NTNU

SAMMENDRAG

I forbindelse med planer om videre drift og en utvidelse av Bjørnevatnbruddet i Sydvaranger Gruve AS ble det gjennomført en masteroppgave ved NTNU som så på stabiliteten i dagbruddsveggene. Det ble foretatt ulike typer stabilitetsanalyser, med hovedvekt på kinematisk analyse og numerisk analyse.

Resultatene fra den kinematiske stabilitetsanalysen viste hvilke utglidningstyper som er mulige i ulike deler av dagbruddet. Den numeriske analysen gav informasjon om endringer i ulike bergmekaniske parametere som spenninger og tøyninger samt indikasjoner på hvor store deformasjoner som kan oppstå ved utvidelse av bruddet.

SUMMARY

A stability analysis of the pit walls in the Bjørnevatn open pit was carried out as part of a master thesis at NTNU. This was done in connection with the plan of expanding the Bjørnevatn open pit towards the deep. Different types of stability analysis were carried out, with the main focus on kinematic and numerical analysis.

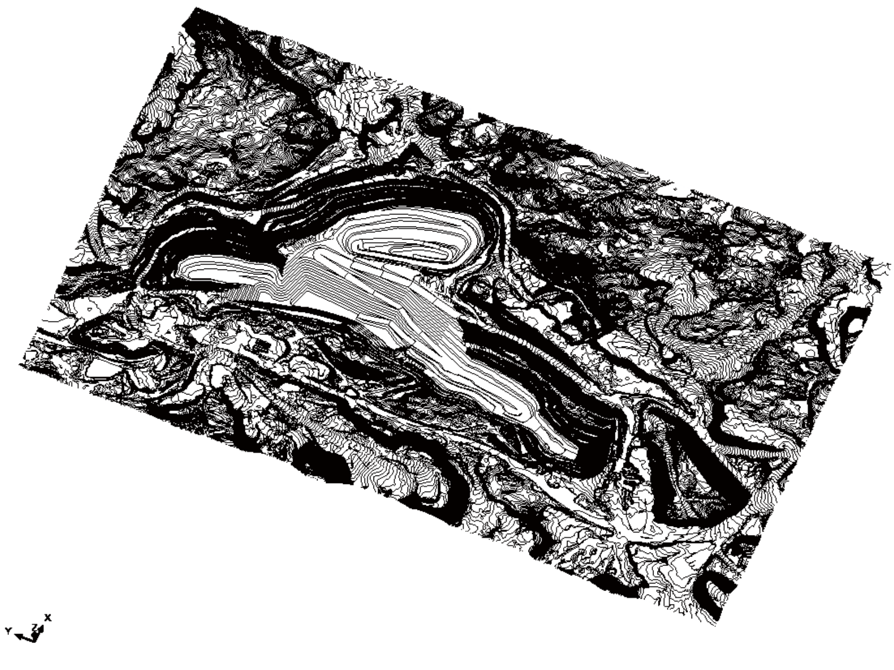
Results from the kinematic analysis showed potential failure in different sectors of the open pit. The numerical modelling gave information about changes in stresses and strains, and indications on how large the deformations can get when excavating the pit further.

INNLEDNING/BAKGRUNN

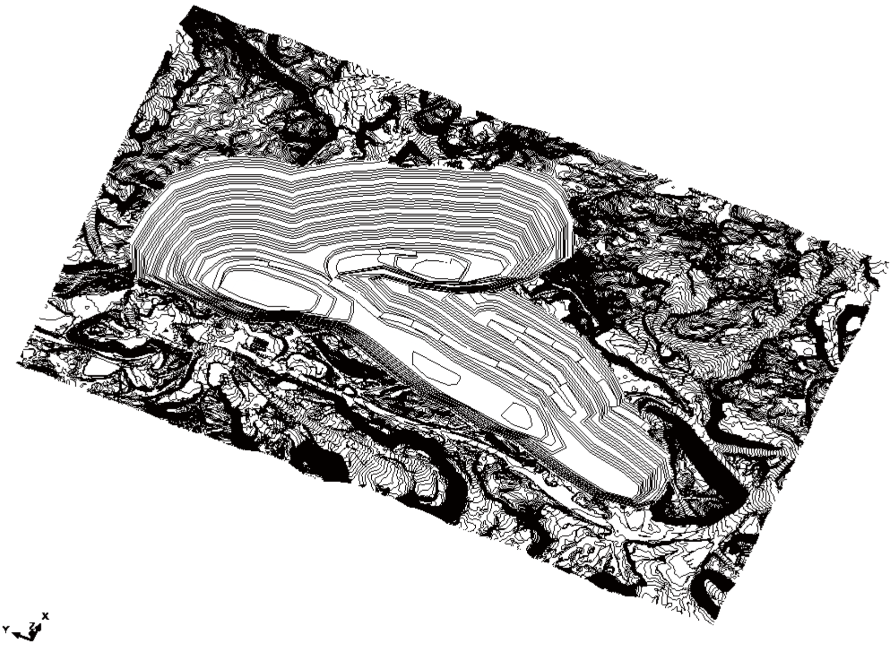
Det er planlagt videre drift i Bjørnevatn, og det innebærer en planlagt utvidelse av dagbruddet mot dypet. I den forbindelse ble det gjennomført et fordypningsprosjekt og en påfølgende masteroppgave ved NTNU Masteroppgaven hadde tittelen "Analyse av skråningsstabilitet i dagbrudd i Sydvaranger Gruve AS " ved NTNU som så på stabiliteten i dagbruddsveggene i Bjørnevatnbruddet. Bjørnevatnbruddet består i dag av tre delbrudd; Bjørnevatn Øst, Vest og Nord. Mellom Bjørnevatn Øst og Vest står det igjen et bergparti kalt sadelen. Bruddgeometri fra mars 2012 er vist i figur 1. I forbindelse med en utvidelse av Bjørnevatnbruddet vil sadelen bli delvis fjernet, men det er likevel viktig å vurdere stabiliteten i dette området, spesielt fordi adgangsrampen til bruddet delvis går langs dette partiet.

Det er tidligere gjennomført flere stabilitetsstudier i Bjørnevatn. Den mest omfattende av disse studiene ble gjort av SINTEF på 1980-tallet, og resulterte i rapporten "Stabilitetsforholdene i Bjørnevannsbruddet A/S Sydvaranger" fra 1985 (Hanssen & Myrvang 1985). Hovedformålet med denne studien var å finne de maksimalt anbefalte skråningsvinklene i Bjørnevatn. Rapporten resulterte i en anbefaling om å dele Bjørnevatnbruddet inn i seks designsektorer med ulike anbefalte maksimale skråningsvinkler. Det er i all hovedsak fortsatt denne rapporten som legges til grunn for vurderinger av skråningsvinkler i Bjørnevatn i dag.

Figur 2 viser bruddgeometrien for det endelige bruddet, slik planen var i mars 2012. Den planlagte utvidelsen medfører en økt dybde på over 100 meter i noen områder.



Figur 1: Bruddgeometri fra mars 2012

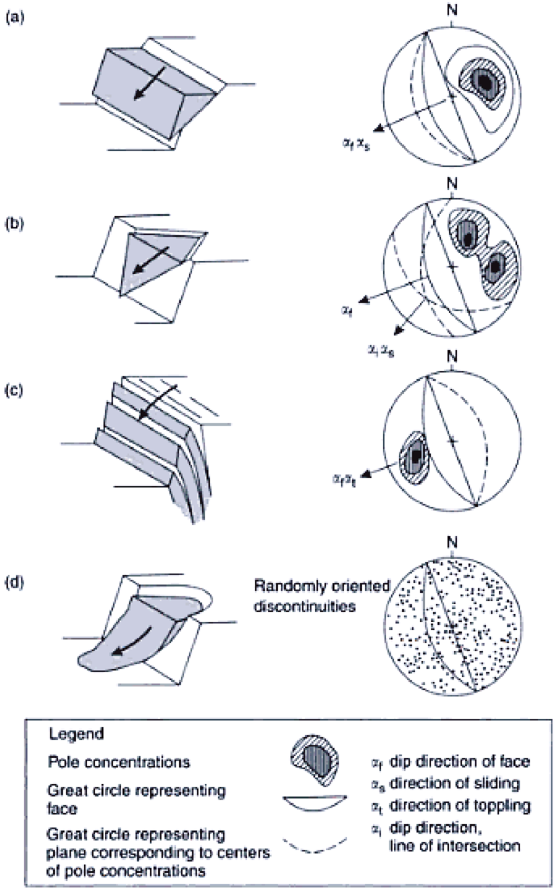


Figur 2: Det planlagte endelige bruddet

TEORETISKE BETRAKTNINGER OM DAGBRUDDSSTABILITET

Det er flere faktorer som virker inn på stabiliteten i en dagbruddsvegg. Det er i tillegg flere måter å klassifisere stabilitet på. Stabilitet kan blant annet klassifiseres etter tidsperspektiv, som korttidsstabilitet og langtidsstabilitet, eller etter størrelse på de ustabile områdene, som detaljstabilitet eller totalstabilitet. I et dagbrudd er det vanskelig å unngå problemer med detaljstabiliteten, men det er viktig å prøve å unngå utrasinger i den grad det er mulig. Store utrasinger hvor flere paller eller hele vegger raser ut utgjør en stor fare for både personell, maskiner og videre drift i et dagbrudd. Spesielt er det kritisk om adgangsrampen til bruddet raser ut.

Utrasinger kan klassifiseres etter geometri. I sterke bergarter er det i hovedsak tre typer utglidninger som forekommer. Det er plane utglidninger, kileutglidninger og toppling. De ulike utglidningstypene er vist i figur 3, som også viser de strukturgeologiske forholdene som vanligvis forårsaker de ulike utrasingstypene. I tillegg til de tre utglidningstypene nevnt tidligere viser figur 3 utglidning langs en krum flate. Denne utglidningstypen er ikke vanlig i sterke bergarter, som de som finnes i Bjørnevatn. I tillegg til de viste utrasingstypene forekommer steinsprang, som er utrasing av en enkeltblokk.



Figur 3: Utglidningstyper: (a) plan utglidning, (b) kileutglidning, (c) toppling og (d) utglidning langs en krum flate (Wyllie & Mah 2004)

BJØRNEVATN

Berggrunnen

Bergartene i Bjørnevatn er hovedsakelig prekambriske bergmasser som er en del av det fennoskandiske skjoldet. Høye horisontalspenninger er vanlig i områder med slike bergmasser. Spenningsmålinger foretatt i 1971 og 1990 (Myrvang m.fl. 1993) bekrefter at det er relativt høye horisontalspenninger i Bjørnevatn. Målingene fra 1990 viser at begge horisontalspenningskomponentene er på rett over 20MPa.

Høye horisontalspenninger påvirker stabiliteten i Bjørnevatn. Spenninger som virker parallelt med malmkroppens strøk vil virke stabiliserende, mens horisontalspenninger som virker normalt på malmkroppens strøkretning kan føre til spenningskonsentrasjoner som kan virke destabiliserende.

Ved bryting av dagbruddet i Bjørnevatn påvirkes spenningsforholdene i berggrunnen. Det vil oppstå spenningskonsentrasjoner langs foten av dagbruddsveggene. Hvis spenningene overskrider bergartens styrke kan slike spenningskonsentrasjoner forårsake utrasinger.

Stabilitetsanalyser

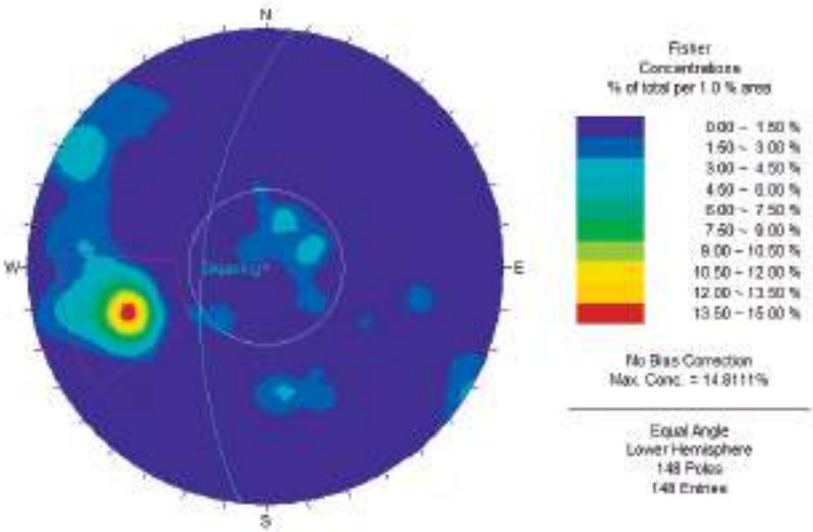
I Sydvaranger er det tidligere gjennomført omfattende stabilitetsanalyser. Disse analysene førte til en plan hvor bruddet er delt i seks designsektorer. De seks sektorene har alle en anbefalt maksimal skråningsvinkel.

Historisk er det registrert flere utrasinger, både plane og kileutglidninger og toppling. I Bjørnevatn Øst er det blant annet spor etter flere kileutglidninger i vestveggen. Det har vært flere mindre utrasinger i Bjørnevatn hvor deler av enkeltpaller har rast ut. I tillegg har det vært noen større ras, hvor flere paller har rast ut samtidig. To av kileutglidningene i Bjørnevatn Øst er utrasinger hvor flere paller raste samtidig.

I masteroppgaven ble det gjort en kinematisk analyse basert på eksisterende sprekkedata for å kunne si noe om potensielle utglidningstyper, og i hvilke områder av bruddet de ulike utglidningstypene kan oppstå. Numerisk modellering ble gjennomført for å kunne vurdere endringer i bergmekaniske parametere som spenninger og tøyninger etter hvert som dagbruddet utvides. I tillegg gav modellene indikasjoner på hvor store deformasjoner som kan forventes.

Kinematisk analyse

Den kinematiske analysen indikerte muligheter for plane utglidninger, kileutglidninger og toppling i ulike deler av bruddet.



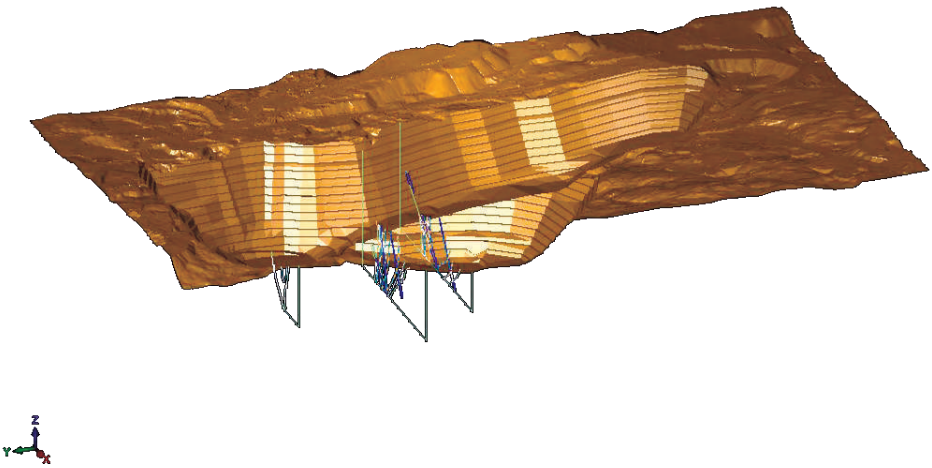
Figur 4: Polplott fra lokalitet i østveggen i Bjørnevatn Vest

Polplottet i figur 4 er basert på sprekkedata fra et område langs østveggen i Bjørnevatn Vest. Polplottet viser hovedsakelig en polkonsentrasjon, og som vist på figur 3 betyr det at det i dette området er fare for toppling.

Resultatene av den kinematiske analysen stemmer godt overens med det som er observert i dagbruddet. I vestveggene i Bjørnevatn er det spor etter kileutglidninger og plane utglidninger, mens det i østveggen i Bjørnevatn Nord er observert spor etter toppling.

Numerisk analyse

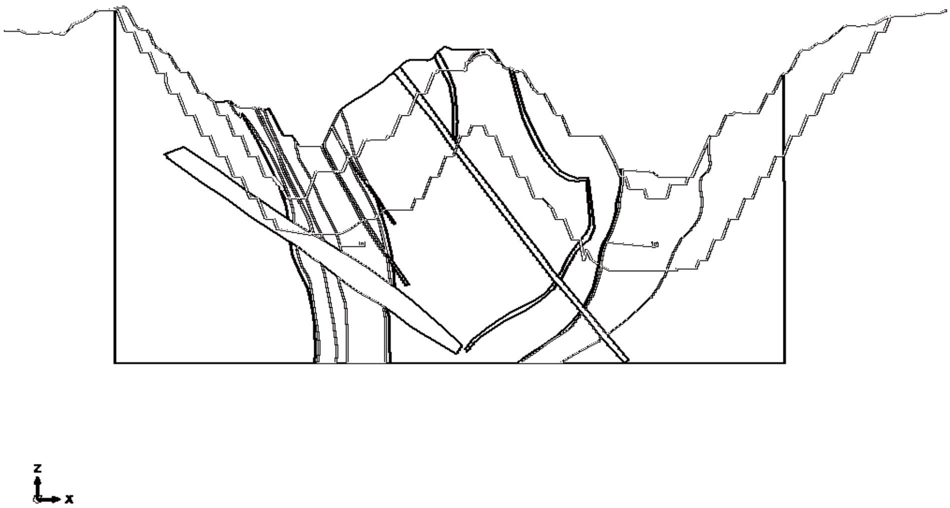
I forbindelse med masteroppgavearbeidet ble det gjort numerisk modellering i 2D for tre profiler i øst-vest-retning. Profilene valgt ut i samarbeid med Sydvaranger Gruve AS og profilenes plassering i forhold til dagbruddet er vist i figur 5. På figur 5 er profilet lengst til venstre det nordligste profilet.



Figur 5: Plasseringen av de tre profilene brukt i den numeriske modelleringen vist i forhold til dagbruddet.

Hvert profil ble modellert for bruddgeometrien ved modelleringstidspunktet og for endelig bruddgeometri. Det sørligste profilet er vist i figur 6. Som profilet viser er det planlagt en relativt stor utvidelse av bruddet.

Bergartene i Bjørnevatn er hovedsakelig ulike gneiser, i tillegg til jernmalmen. I tillegg finnes det noen yngre diabasganger i området. Slike bergarter er harde og sprø bergarter.



Figur 6: Profil i øst-vest-retning som viser geologi, bruddgeometri fra 2012 og planlagt bruddgeometri for endelig brudd. (Profilet er det sørligste profilet som ble brukt i den numeriske modelleringen)

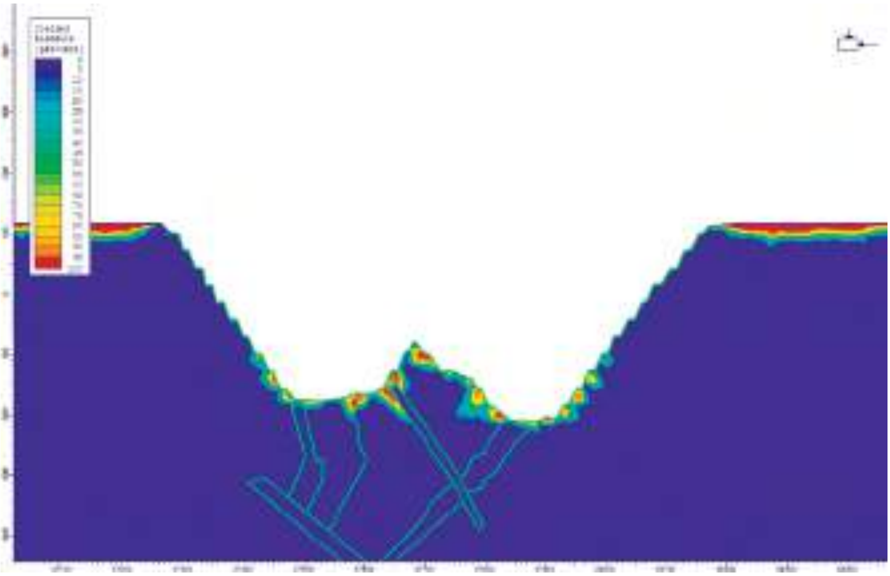
Programmet Phase^2 fra Rocscience ble brukt til den numeriske modelleringen. Phase^2 bruker endelig element-metoden (FEM). Kvaliteten på resultatene fra den numeriske modelleringen er avhengig av kvaliteten på inngangsparameterne. Inngangsparameterne i dette tilfelle er forholdet mellom vertikal- og horisontalspenninger, geometri og bergartsegenskaper.

Resultatene fra den numeriske analysen gav indikasjoner på endringer i spennings- og tøyningssituasjonen, deformasjoner og en indikasjon på materialenes styrkefaktor for de ulike trinnene av modelleringen.

Harde, sprø bergarter viser generelt elastisk oppførsel. Materialene ble derfor i utgangspunktet satt til å være elastiske i Phase2. For elastiske materialer gir Phase^2 en styrkefaktor for hvert element i modellen. Styrkefaktor er definert i Phase2 som forholdet mellom styrken til et material og indusert spenning i et gitt punkt. Styrken til et materiale er basert på styrkeparameterne til materialet. De induserte spenningene i et punkt er bestemt av den elastiske spenningsfordelingen beregnet i Phase2. En styrkefaktor som er større enn 1 indikerer at materialets styrke er større enn de induserte spenningene, mens en styrkefaktor som er lavere enn 1 indikerer at spenningene i materialet overgår materialets styrke.

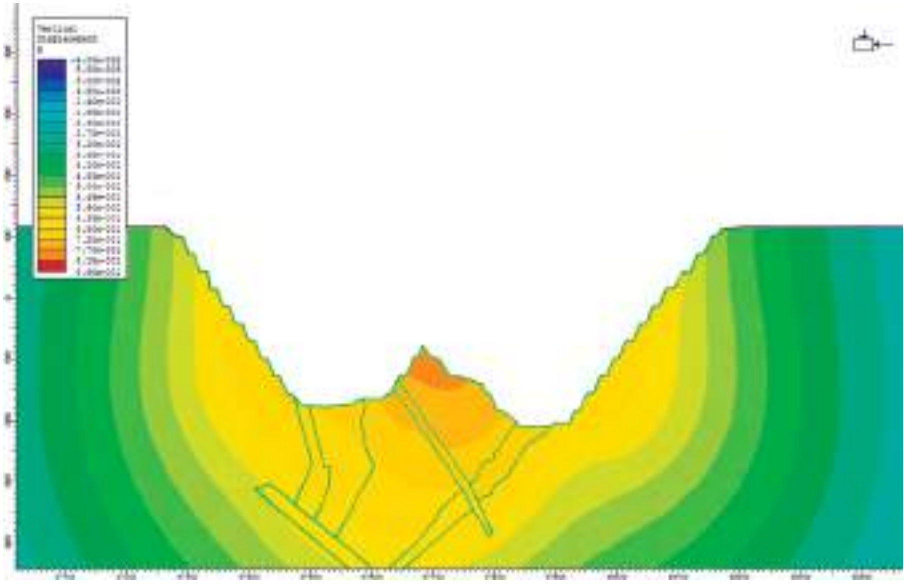
Etter å ha kjørt modellene med elastiske materialer viste resultatene at noen områder i hver av modellene fikk en resulterende styrkefaktor på under 1. Modellene ble derfor kjørt igjen med materialene satt til å være plastiske. Dette for å undersøke i hvor stor grad materialene hadde overskredet flytgrensen når det ble antatt plastisk deformasjon. Resultatet var at elementer med styrkefaktor lavere enn 1 i hovedsak tilsvarer elementer i flyt.

De områdene som modellen gir en styrkefaktor lavere enn 1 etter siste trinn av modelleringen er hovedsakelig områdene nær bunnen av dagbruddet i tillegg til områdene ved sadelen og overflateområdene rett ved siden av dagbruddet. Figur 7 viser elementer i flyt etter siste trinn av modelleringen for det midterste profilet. Trenden er den samme for de to andre profilene. Det hadde vært mulig å bekrefte deler av modellene ved å kartlegge i hvor stor avstand fra bruddet det ble observert brudd i bergartene i langs overflaten. Grunnet tidsforhold og det faktum at store deler av området er dekket av deponerte masser ble ikke enn slik kartlegging gjennomført.



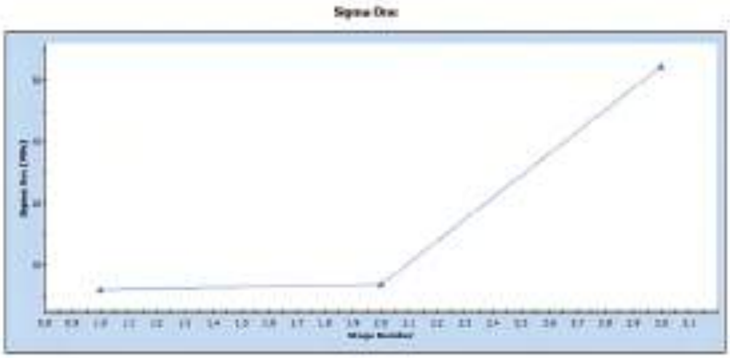
Figur 7: Elementer i flyt etter siste trinn av modelleringen

Generelt viser resultatene fra den numeriske modelleringen at det i områder nær overflaten vil kunne oppstå relativt store deformasjoner. Vertikalforskyvningen er stort sett større enn horisontalforskyvningen. Særlig er vertikalforskyvningen relativt stor i sadelområdet, som vist på figur 8.



Figur 8: Vertikal forskyvning etter siste trinn av modelleringen

Som nevnt tidligere er det høye horisontalspenninger i Bjørnevatn. De numeriske modellene viste økte spenningskonsentrasjoner nær bunnen av dagbruddet etter hvert som bruddet ble utvidet mot dypet. Figur 9 viser endringen i største hovedspenning for et punkt som i utgangspunktet lå over hundre meter under dagbruddet, men som etter siste modelleringstrinn lå rett ved foten av dagbruddsveggen. Grafen viser tydelig hvordan spenningen øker i størrelse når dagbruddet utvides.



Figur 9: Endringer i størrelse for største hovedspenning

Resultatene fra den numeriske modelleringen viste at det er fare for relativt store forskyvninger samt økte spenningskonsentrasjoner i skjæringsfoten ved en utvidelse av Bjørnevatnbruddet. Basert på den numeriske analysen bør det være gjennomførbart å utvide bruddet i Bjørnevatn som planlagt, så lenge visse forutsetninger er på plass. Det er blant annet viktig med god oppfølging av videre stabilitetsvurderinger.

SLUTTKOMMENTARER

Det må omfattende undersøkelser og analyser til for å kunne vurdere stabiliteten i et dagbrudd. Det er vanskelig å si noe basert på kun én type stabilitetsanalyse. I Bjørnevatn blir det i tiden fremover viktig med kontinuerlige vurderinger og god oppfølging. Hvis det blir gjennomført, bør det være mulig å utvide mot dypet.

Det kan være aktuelt med flere numeriske modeller ved videre drift i Bjørnevatn. I så tilfelle kan det være nyttig med nye spenningsmålinger. Resultatene fra slike målinger kan deretter inkorporeres i nye numeriske modeller. Nye numeriske modeller kan og ta hensyn til parametere som ikke ble vurdert i denne omgang. For eksempel kan informasjon om sprekkesett og grunnvannsforhold legges inn i en numerisk modell. Det kan og være aktuelt med en modellering i 3D.

Resultatene av masteroppgavearbeidet viser hvordan ulike analyser kan kombineres for å gi et mer komplett bilde av stabiliteten.

Til slutt vil jeg takke Sydvaranger Gruve AS for samarbeidet i forbindelse med masteroppgaven og for all hjelp med tilgang til data og informasjon om Bjørnevatnbruddet.

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UTSLAGSSALVE FOR KRAFTVERK I INDIA

Lake tap blasting for a hydropower project in India

Siv.ing Thomas Korssjøen Mathiesen, Norconsult AS

SAMMENDRAG

Norconsult har i perioden 2002 til 2012 vært engasjert som konsulent for detaljprosjektering og oppfølging av et dobbelt utslag under vann i Lake Koyna, Stage IVb.

Prosjektet ligger in den indiske delstaten Maharashtra, 180km sør-øst for Mumbai, i et område dominert av vulkanske bergarter; sub-horisontale lag av basalt, adskilt av vulkansk breksje, tuff og agglomerat. Bergmassen har generelt moderat oppsprekning og er av middels til god kvalitet. Berget i utslagsområdet er generelt overlagret av et 6-8m tykt lag med konsoliderte sedimenter.

Utslaget omfatter en 90m vertikal lukesjakt, 2 separate inntakstunneler, massegrop og 45° sjakter opp til utslagspluggene.

Utslagssalvene ble utført 25.˙ april 2012, ved bruk at NSP711 (Bonogel) og ikke-elektriske tennere. Før salvene ble forventete maksimalt trykk på lukene beregnet til 76.8 mVS (venstre inntak) and 77.2 mVS (høyre inntak). Målinger av trykk under utslagssalven viste at trykket var henholdsvis 73.2 og 69.5 mVS.

SUMMARY

Norconsult has been involved as a consultant for detail design and follow-up of a double lake tap in Lake Koyna, Stage IVb, from 2002 to 2012.

The project is located in the state of Maharashtra, about 180km south-east of Mumbai, in an area dominated by volcanic rock; sub-horizontal layers of basalt with alternating layers of pyroclastic sediments (tuff, breccia, agglomerate etc.). The rock mass is generally moderately jointed, of fair to good quality. The rock in the intake area is covered by a 6-8m thick layer of consolidated soil material.

The layout of the double lake tap comprise 90m vertical gate shafts, 2 separate intake tunnels, spoil traps and 45° shafts towards the intake plugs.

The final blasts were successfully performed 25th April 2012, using NSP711 (Bonogel) explosives, initiated by a nonelectric detonation system. Prior to the blasts, simulations suggest expected maximum pressure on the gate of 76.8 mWc (left) and 77.2 mWc (right). The measured pressure during/after blast was recorded to be 73.2 and 69.5 mWc respectively.

BACKGROUND

Lake Koyna (Shivajisagar Lake) is a large lake in the state of Maharashtra in India, about 180km south-east of Mumbai. After the construction of a Koyna dam (Figure 1), completed in 1963, at the natural river outlet near the village of Koyna Nagar, the lake stretches some 50 km north of the outlet. The energy project utilising the reservoir has been constructed in several stages, totalling to about 1960 MW installed capacity, making it the largest hydroelectric power plant in India. The outlets from the dam and from the underground powerstation serve as important steady sources of irrigation water throughout the dry season.

Figure 1 Koyna Dam.

Norconsult has been involved in the development of the energy project at Koyna for the two latest stages, IV and IVb, from 1994 to 2012. A further expansion stage V is currently under construction and there are also further plans for future expansions and irrigation projects. The owner of the project is the government of Maharashtra and the recent construction projects have been managed by the government controlled design and construction agency Koyna Construction. The contractor for the previous Stage IVa and the recent Stage IVb has been Patel Engineering. Norconsult was first engaged by this contractor in 1994 as consultant for detail design and later follow-up of the Lake Tap for Stage IVa, which was completed in 1999. This project was performed as a traditional lake tap, with a double intake. During the construction period of this project the the reservoir was lowered below the elevation of the lake tap, due to maintenance of the dam, allowing detailed investigation, dredging and other preparatory work for the lake tap to be performed at dry the lake bed prior to the piercing blast.

Norconsult was contacted by the same contractor again in 2002, as a consultant for the new lake tap project of Stage IVb, this time at a somewhat deeper location. The preliminary design for this project, dimensions and geometric layout, was identical with the previous project. In the stage IVb Norconsult was able to influence the design process somewhat, resulting in a slightly more favourable geometric layout; however, the overall constructability of the project remained somewhat more complicated and time-consuming than what would have been preferred.

The main responsibility of Norconsult was focused on the final 75m of tunnelling towards the reservoir and the final lake tap including; system for probe drilling and grouting, blast design, system for water and air filling and instrumentation/monitoring.

Figure 2 Location of project area.

Figure 3 Satellite photo showing the detailed location and layout of the project.

METHODS FOR SUBMERGED TUNNEL PIERCINGS (LAKE TAPS)

The traditional lake tap method, originating from Norwegian hydropower development, involves excavating a tunnel towards and under a lake, leaving a short rock plug to the lake bed. A final blast round prepared from the tunnel pierces the lake bed from below.

Although being a risky operation, with a definite chance of things not going exactly as planned, the upside, with regards to cost and time savings, leaves the lake tap option interesting compared to alternative methods for creating a submerged connection to the reservoir. Being a risky operation one of the most important principles in lake tap design is finding ways to reduce the probability of failure of all elements comprising the lake tap operation.

The method for lake tap into a tunnel system containing one form of a closure device, (gate or valve), can generally be divided into; closed and open systems. Figure 4 illustrates the two systems in a typical hydropower layout. In open lake tap systems the tunnel/rock plug is connected to the atmosphere, for example through a gate shaft, allowing the surge to even out the pressure after blasting. In closed systems the tunnel is isolated from the atmosphere by the gate. Both methods can be performed with full, partial or no water filling.

The filling of water in the tunnel prior to the blast will effectively slow down or eliminate the water flow into the tunnel, thereby limiting the sediment transport and ensuring that the rock debris settle in the intended spoil trap just inside the intake. This is usually necessary if a gate or valve is present relatively close to the intake. Further, the filling of water will limit surge of water through the tunnel/shaft and thereby reduce the maximum pressure on gates or valves in the tunnel system. If the intake tunnel inside the final plug is left dry, a rush water from the reservoir will fill up the tunnel once the final plug is blasted. The high velocity of the water flowing from the reservoir will transport rock debris that may damage any nearby gate structures.

If there are no structures that may be damaged by water pressure or sediment transport and the surge of water is acceptable, as may be the case for sewage water outlets or cooling water tunnels, leaving the intake tunnel dry may in some cases be chosen as a simple and inexpensive alternative. This method is often referred to as "*open dry system*". Equally, a closed system may also in some cases be performed without water filling, as the compression of the trapped air between the piercing and the gate will reduce the hydrodynamic shock and slow down the velocity of the water sufficiently so that the rock debris does not damage the gate. However, the latter system requires a significant distance between the piercing and the gate.

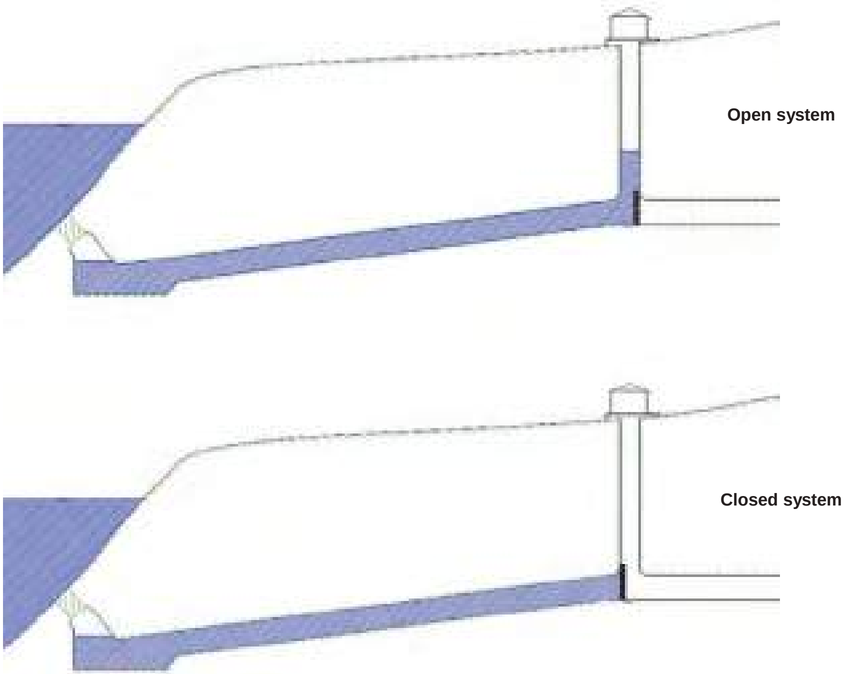

Figure 4 Principle layout of open (top) and closed (bottom) lake tap systems.

Water filling and compression of air pocket

In the case of water filled system it is crucial that no part of the explosives is in contact with the water in the tunnel during detonation. Even a very small charge will easily generate hydrodynamic shock waves that may very well damage gates or valves. It is therefore important to leave an air pocket towards the final rock plug, ensuring that the explosives are not in contact with the water in the tunnel. Further, the air pocket will also ensure that the explosives, detonators, and connections may be kept as dry as possible in order to minimize the risk of faulty ignition. An air pocket is enabled by utilising an inclined tunnel/shaft towards the rock plug, ref. Figure 4.

The water filled solution is generally considered to be a safer and more controlled way of performing a lake tap in cases where there are structures that may be damaged by pressure or sediment transport or where the surge of water is not acceptable. The method however, is more complicated and requires special attention to geometry and a practical design of systems for controlling and monitoring water level and pressure in the air pocket.

If there is a difference in the pressure of the water inside the tunnel and in the reservoir, a surge wave will result in a pressure build-up occurring after the rock plug is blasted. In order to allow lean and cost-optimal dimensions of the gate structures, it is generally desirable to keep the maximum pressure build-up as low as possible. This may be achieved by pre-compressing the air in the air pocket to an optimal level. Optimal pressure difference between the tunnel and the reservoir (before inclusion of explosive gasses) is often found to be in the order of 0.5 to 1 bar. However, this may vary depending on the geometry of the tunnel system, the amount of explosives and the size of the air pocket. In some cases it may also be advisable to deliberately increase the resulting pressure difference in order to obtain more energy from the inflowing water to assist in achieving the desired opening after the blast.

Pre-compression of air in the air pocket may be achieved by either allowing the air to be compressed as it is trapped in the air pocket during water filling, or by injecting air through a pipe into the air pocket. Although the first is usually a time efficient way of obtaining pressurised air at the face a system that allows for adjustment, being either addition or evacuation of air pressure, is generally advisable. The optimal volume and pressure in the air pocket may be found through hydrodynamic analyses of the system. Such analyses may be analytical, empirical, mathematical and/or numerical. The analyses must consider the geometry and dimensions of the tunnel system in relation to the volume and pressure of the air pocket and amount of gas developed by the explosives during the blast. Analyses depend on basic hydrodynamic theory; however, the complexity of the total lake tap system leaves significant uncertainty in many important factors of the analyses.

Design of system for monitoring

As mentioned above the importance of achieving the optimal volume and pressure in the air pocket necessitates a precise and reliable system for monitoring the water level in the tunnel, and the pressure in both the water and the air inside the air pocket. Further, it is prudent to have a system that monitors the actual pressure build up during the blast and inflow of debris and water, both for verification of the maximum load that the gate is exposed to and also for analyses of the surge wave that may indicate the propagation of the break through, thereby indicating the achieved dimensions of the opening toward the reservoir.

The system for monitoring does not have to be very complex or sophisticated but it important that the system functions flawless through the entire operation. Keeping in mind the important principle of reducing probability of failure, it is prudent to design the system with ample redundancy in case of failure in; instrumentation, cables or connections. Further, the installation of monitoring equipment must be carefully carried out, ensuring that the equipment or cables are not damage prior to the blast. Particularly, such damages may be caused by rock fall, which may very well occur during filling of water.

Blast design, charging, and detonation system

The main principles of a blast design for a lake tap is based on the fact that this is a very critical operation. It is important to achieve complete detonation, sufficient break and good fragmentation of the rock debris, thus allowing all rock debris to fall down and settle in the spoil trap (or flushed into the tunnel, for dry systems) leaving a sufficient hydraulic opening area for the operation of the tunnel. Once the final blast of a lake tap is initiated and completely or partially detonated, it is considered very difficult and/or dangerous to repair or improve a faulty or incomplete breakthrough to the reservoir. Unsuccessful lake taps do occur from time to time, for various reasons. Although it is usually possible to remedy such situations this is generally time consuming and usually involves a significant cost compared to the original design. It is therefore prudent to utilise a well thought through design of the lake tap system and pay special attention to close follow-up and monitoring of all elements in order to reduce the risk of failure.

Compared to normal tunnel blasting the following principles are usually used for blast design:

- High amount of explosives in a tight drill pattern; usually specific charge in the order or 2-3 times of normal tunnel blasting
- Higher number of detonators and 2 or more separate ignition systems - to minimise risk of incomplete ignition
- Shorter total ignition time - to ensure total ignition before blast fragmentation occur, and reduce the risk of charges falling down in the water prior to detonation

DESIGN AND EXECUTION OF LAKE TAP AT KOYNA STAGE IVb

Ground conditions

The rock mass in the project area comprises volcanic rock series of basalt with sub-horizontal layers of pyroclastic sediments (tuff, breccia, agglomerate etc.) between the lava flows. The basalt is competent with RQD mainly above 75%. The sediment layers are generally weaker rock types, but these are also moderately jointed and of fair to good quality. The thickness of the basalt layers generally range from 2 – 8 m.

Core drilling investigations was performed in order to ensure that the final rock plugs of the lake taps were located entirely within a layer of basalt of acceptable quality. However, the shaft between the plug and the main tunnel would pass through at least one layer of breccia and tuff.

Although the headrace tunnel had passed through a significant weakness zone with water ingress of about 3000 l/m, no weakness zones were expected between the intake gate structure and the lake taps.

The overburden material in the project area is comprised of consolidated reddish coloured sand, silt and clay, with increasing content of larger rock fragments towards depth. Along the shoreline of the lake the overburden could be studied in detail and studies showed that the material was rather evenly distributed over the entire project area.

Layout of the intake

The rock surface at the intakes for the Stage IVb was at elevation 602 m.a.s.l., however it was observed about 7 m thick soil overburden in the intake area. Due to the consolidated nature of the material it was decided to reduce the thickness by means of dredging. During this work it was observed that some parts of this overburden, particularly towards depth, was very competent and drilling and blasting was required to loosen the material. After dredging the overburden over the rock plugs were between 1.0 and 3 m.

The reservoir level varies from about elevation 636 up to 655 m.a.s.l., however, it was early established that the dimensions of the gate structure and gates would limit the acceptable level during the lake tap blast.

From the intake, there is a shaft at 45° angle, down to the intake tunnel leading to the gate structures in the bottom of a 90m tall vertical gate shaft. At the foot of the inclined shaft to the intake there is a 12 m deep and 45 m long spoil trap. The dimensions of the spoil trap was several times larger than necessary, allowing the debris from the final excavation and trimming of the rock plug to be dumped in the spoil trap leaving mucking of the debris not necessary at this point.

The intakes are 7.5m in diameter, circular, and the final rock plug lengths were designed to be 5.0 and 5.5m. The angle of the final drilling was shifted from the 45° of the shaft to 80°, corresponding more or less to the perpendicular angle to the rock surface at the lake bed.

The distance along the intake tunnels from the intake to the gates is 265m for the left intake and 289m for the right intake. For hydraulic reasons all parts of the tunnel system is fully concrete lined all the way up to about 15m from the intake. The rock mass conditions in the project area are fairly good and would have allowed a large part, if not all, of the tunnels to be left unlined.

A 3D model from the intakes to the gate structures is shown in Figure 5.

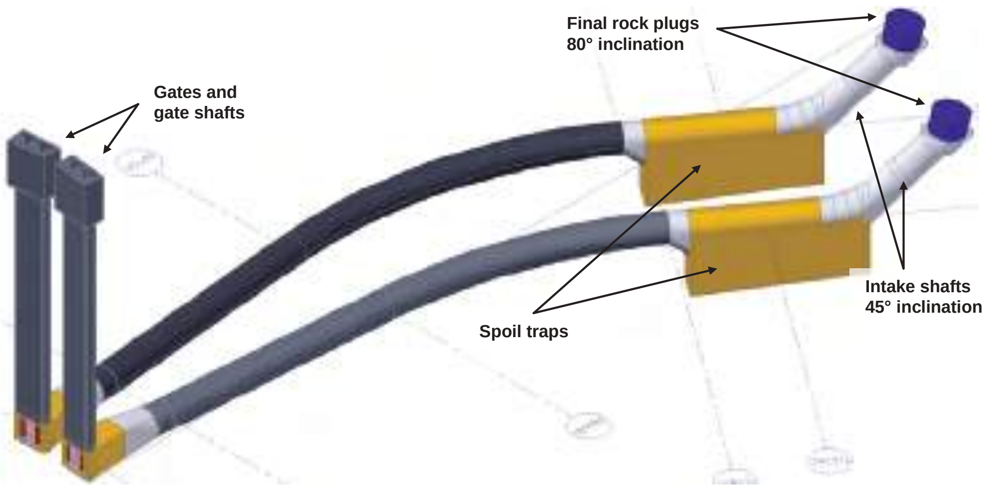

Figure 5 Geometric layout of the Koyna double lake tap.

Tunnelling towards the intake

Excavation of the last part of the tunnel towards the intake is regarded as sub-sea tunnelling and gradually also tunnelling with very low overburden. General principles to be followed were:

- Probe drilling in order to be well prepared for any adverse rock mass conditions of significant water leakage
- Probing all the way through to the reservoir at critical locations in order to verify the exact location of the tunnel in relation to the lakebed
- Careful blasting in shorter rounds as the face approaches the final rock plug
- System of probe drilling through the final face was performed in order to gather data for the final blast design