Reliability-based approach for robust geotechnical design

Approche fiabiliste pour un dimensionnement géotechnique robuste

Farrokh Nadim
Technical Director, Norwegian Geotechnical Institute, Norway, farrokh.nadim@ngi.no

ABSTRACT: Although geotechnical engineers have to deal with uncertainty and risk all the time, it is not common to talk about uncertainties and risk in the profession. Uncertainties are often addressed indirectly through the choice of safety factors and design criteria or wording in design guidelines. Reliability theory, which is used to calculate risk, provides the framework to account for uncertainties in a systematic manner. In this paper, notions of hazard, exposure, vulnerability, risk, risk management, acceptable risk, and reliability-based geotechnical design are addressed, and the State-of-Practice is exemplified with several case studies. The examples illustrate the quantification of uncertainty in geotechnical calculations, reliability-based foundation design, the assessment of hazard, vulnerability and risk, and the treatment (through mitigation measures) of risk under different design situations. The case studies include the settlement predictions for an embankment, the stability of underwater slopes subjected to earthquake hazard, the run-out distance for a quick clay slide, the reliability-based design of offshore piles, the breach of embankment dams and the evaluation of mitigation measures. Challenges and emerging issues like climate change, cascading events and the management of the risk posed by extreme events are discussed; and recent developments, such as stress testing and Bayesian networks, for dealing with these challenges are presented.

RÉSUMÉ: Bien que les ingénieurs géotechniques doivent faire face à l'incertitude et le risque en tout temps, il n'est pas commun pour eux de parler des incertitudes et des risques dans le dimensionnement des fondations. Les incertitudes sont souvent abordées indirectement par le biais d'un choix prudent des facteurs de sécurité et de critères de conception. La théorie de la fiabilité, qui est utilisée pour calculer les aléas et le risque, fournit un cadre pour tenir compte de ces incertitudes d'une manière systématique. L'article présente en premier lieu les notions de risque, vulnérabilité, exposition, aléas et gestion des risques, y compris l'acceptabilité du risque. La conception géotechnique basée sur la l'approche fiabiliste est ensuite traitée, et l'implémentation des méthodes est illustré par des études de cas issus de la pratique. Les exemples illustrent la quantification de l'incertitude dans les calculs géotechniques, la conception de la fiabilité d'une fondation, l'évaluation des risques et de la vulnérabilité, et le traitement (réduction) des risques dans différentes situations de conception. Les exemples portent sur la prévision du tassement sous un remblai, la stabilité des talus, les pentes sous-marines soumises aux secousses sismiques, l'étalonnage des facteurs de sécurité pour fondations en mer, la longueur de parcours des coulées argileuses, et l'analyse probabiliste des barrages. Défis et enjeux émergents tels que le changement climatique, les événements en cascade, "stress testing" et la gestion des risques posés par les événements extrêmes sont aussi abordés.

KEYWORDS: Uncertainty, risk, reliability, reliability-based design, acceptable risk, stress testing

1 INTRODUCTION

This paper is the third Suzanne Lacasse Lecture of ISSMGE's TC304. Its focus is on how uncertainties could be treated in geotechnical design. Geotechnical engineers have to deal with uncertainty in all aspects of their analysis and design. Even in relatively straightforward and routine projects, there are uncertainties in soil layering, in situ mechanical soil properties, pore pressures in the ground, geotechnical models that are used to make calculations, and external loads and load effects that act on a structure or system.

Uncertainty is closely related to risk. As a matter of fact, the ISO Guide 73:2009 Risk management – Vocabulary (https://www.iso.org/obp/ui/#iso:std:iso:guide:73:ed-1:v1:en), defines risk as "effect of uncertainty on objectives". Paradoxically, although geotechnical engineers talk about risk all the time, it is not common to acknowledge uncertainties in geotechnical engineering. Only a few university geotechnical engineering programs offer courses in reliability theory and risk assessment as part of their formal curriculum. Most geotechnical engineers, both those with years of experience and those freshly graduated from the university, have a deterministic mindset, believing that the risks can be managed just by being conservative.

Since the 80's, hazard and risk assessment of the geo-component of a system has gained increasing attention. The offshore, hydropower and mining industry were the pioneers in applying the tools of statistics, probability and risk assessment. Whitman (1996) offered examples of how probabilistic analysis can best be used in geotechnical engineering, and what types of projects the approach is appropriate for, and concluded that probabilistic methods are tools that can effectively supplement traditional methods for geotechnical engineering projects, provide better insight into the uncertainties and their effects and an improved basis for interaction between engineers and decision-makers.

The offshore industry has been a leader in explicitly considering risk in geotechnical design and decision-making. Due to the severe conditions that offshore installations are exposed to and the extreme conditions they are designed for, the offshore industry has had the opportunity to learn from experience to innovate and improve practice to better manage risk. Gilbert et al. (2015) presented case histories related to risk and reliability on the frontier of offshore geotechnics. Their case histories underscore the following:

1. Achieving an appropriate risk requires balancing risk and conservatism;
2. Managing risk requires understanding the loads as well as the capacities;
3. Maximizing the value of data used to make design decisions requires considering the potential the data have to affect the decisions; and
4. Developing effective geotechnical designs requires understanding how these designs fit into the larger systems they support.

In the 21st century, the awareness of the need for mitigation of impacts of natural hazards on the society has greatly increased. Environmental concerns and the society's expectation that the risk posed by geohazards be understood and managed have advanced the science behind hazard and vulnerability assessment. Nowadays, the notion of hazard and risk is a natural question in most aspects of the design foundation, and geotechnical engineers play a key role in the risk assessment for geohazards.

This paper argues that addressing the uncertainties in a quantitative or semi-quantitative manner would result in more robust and optimal geotechnical designs. This is demonstrated through a number of case studies covering a variety of problems. However, before presenting the case studies, a bit of background on risk assessment, the acceptable risk concept and reliability-based design is provided.

2 TERMINOLOGY

The terminology used in this paper is generally consistent with the recommendations of ISSMGE Glossary of Risk Assessment Terms (listed on TC304 web page: http://140.112.12.21/issmge/2004Glossary_Draft1.pdf):

Danger (Threat): Phenomenon that could lead to damage, described by geometry, mechanical and other characteristics. Its description involves no forecasting.

Hazard: Probability that a danger (threat) occurs within a given period of time.

Exposure: The circumstances of being exposed to a threat.

Risk: Measure of the probability and severity of an adverse effect to life, health, property or environment. Risk is defined as Hazard × Potential worth of loss.

Vulnerability: The degree of loss to a given element or set of elements within the area affected by a hazard, expressed on a scale of 0 (no loss) to 1 (total loss).

Although at first sight, the TC304 definition of "risk" seems to be different from that of ISO/Guide 73:2009(en), clarification Notes 3 and 4 of the latter state that:

"…..

Note 3: Risk is often characterized by reference to potential events and consequences, or a combination of these.

Note 4: Risk is often expressed in terms of a combination of the consequences of an event (including changes in circumstances) and the associated likelihood of occurrence."

In other words, the T304 definition of risk is one of the many possible ways to characterise and quantify risk.

3 RISK ASSESSMENT AND MANAGEMENT

The ISO Guide 73:2009 defines risk management as "coordinated activities to direct and control an organization with regard to risk". Its purpose is to reduce the risk to levels that are deemed tolerable or acceptable by the society. The management process is a systematic application of management policies, procedures and practices. A risk management framework comprises the following main tasks: (a) Danger or hazard identification; (b) Cause analysis of the dangers or hazards; (c) Consequence analysis, including vulnerability analysis; (d) Risk assessment combining hazard, consequence and uncertainty assessments; (e) Risk evaluation or is the risk acceptable or tolerable?; and (f) Risk treatment or what should be done?

Risk management integrates the recognition and assessment of risk with the development of appropriate treatment strategies (e.g. Fig. 1). Understanding the risk posed by natural events and human-induced activities requires an understanding of its constituent components, namely characteristics of the danger or threat, its temporal frequency, exposure and vulnerability of the elements at risk, and the value of the elements and assets at risk. The assessment systemizes the knowledge and uncertainties, i.e. the possible hazards and threats, their causes and consequences. This knowledge provides the basis for evaluating the significance of risk and for comparing options. Risk assessment is specifically valuable in detecting deficiencies in complex technical systems and in improving the safety performance, e.g. of storage facilities.

Risk communication means the exchange of risk-related knowledge and information among stakeholders. Despite the maturity of many of the methods, broad consensus has not been established on fundamental concepts and principles of risk management. The field suffers from a lack of clarity on key scientific pillars, e.g. what risk means and how risk is best described, making it difficult to communicate across disciplines and between risk analysts and stakeholders.

The ISO 31000 (2009) risk management process (Fig. 1) is an integrated process, with risk assessment, and risk treatment (or mitigation) in continuous communication and consultation, and under continuous monitoring and review. Due to the aleatory (inherent) and epistemic (lack of knowledge) uncertainties in hazard, vulnerability and exposure, risk management is effectively 'risk-informed' decision-making under uncertainty. Today's risk assessment addresses the uncertainties and uses tools to evaluate losses with probabilistic metrics, expected annual loss and probable maximum loss. Future-oriented quantitative risk assessment should include uncertainty assessments, consider technical feasibility, costs and benefits of risk-reduction measures and use this knowledge for the selection of the most appropriate risk treatment strategies.

Fell et al. (2005) made a comprehensive overview of the state-of-the-art in landslide risk assessment. Düzgün and Lacasse (2005) listed a large number of proposed risk formulations. The first step in any risk reduction process is a quantitative risk assessment. For landslides for example, one would (1) define scenarios for triggering the landslide and evaluate their probability of occurrence; (2) compute the run-out distance, volume and extent of the landslide for each scenario; (3) estimate the losses for all elements at risk for each scenario; and (4) estimate the risk.

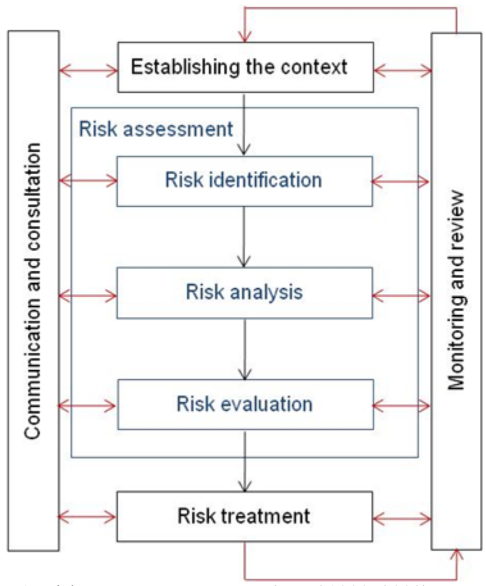

Figure 1. Risk management process (ISO 31000, 2009).

4 CONCEPT OF ACCEPTABLE/TOLERABLE RISK

A complex task in risk management is establishing risk acceptance criteria. The basic concept of acceptable risk is quite simple. In engineering applications, risk is commonly defined as a function of probability of occurrence of an undesirable event (hazard) and its potential consequences. When an event has a very low hazard and its potential consequences are small, e.g. Event 1 in Figure 2, the risk associated with that event is insignificant and acceptable. On the other hand, if an event occurs frequently and has the potential to inflict significant loss and damages, e.g. Event 2 on Figure 2, the risk associated with that event is unacceptably high. Conceptually, as depicted in Figure 2, somewhere in the 2-dimensional space of hazard-consequence, there is curve or a transition zone that separates the acceptable risk situations from the unacceptable ones. Complications start once one attempts to delineate the acceptable and unacceptable risk zones quantitatively in the hazard-consequence space.

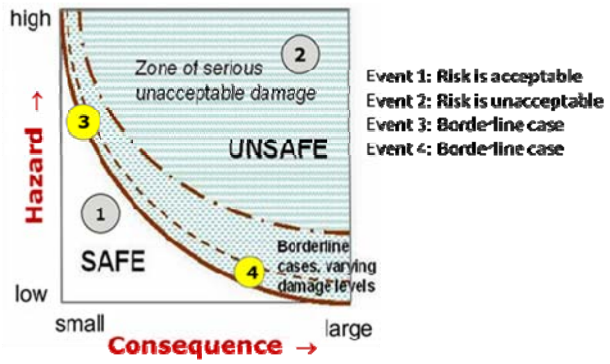

Figure 2. Schematic diagram of acceptable/tolerable risk concept.

Risk acceptability depends on factors such as voluntary vs. involuntary exposure, controllability vs. uncontrollability, familiarity vs. unfamiliarity, short vs. long-term effects, existence of alternatives, type and nature of consequences, gained benefits, media coverage, information availability, personal involvement, memory, and level of trust in regulatory bodies. Voluntary risk tends to be much higher than involuntary risk. If under personal control (e.g. driving a car), the risk is more acceptable than the risk controlled by other parties. For landslides, choosing to live close to a natural slope is a voluntary risk, while having a slope engineered by the authorities close to one's dwelling is an involuntary risk. Societies that experience geohazards frequently may have a different risk acceptance level than those experiencing them rarely.

The Geotechnical Engineering Office of Hong Kong, GEO, compared societal risks as described in a number of national codes and standards. Figure 3 shows the comparison. Although there are differences, the recommended risk level centres around 10^{-4}/year for ten fatalities.

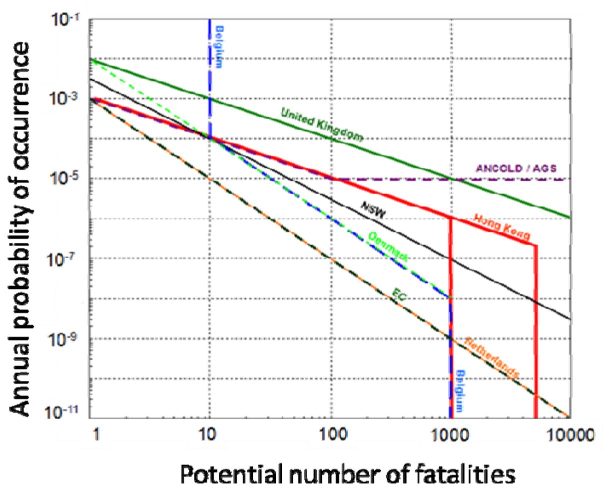

Figure 3. Acceptable Societal Risk criteria in different countries (Ho, K. Personal communication. Gov. of Hong Kong SAR, CEDD, GEO, Nov. 2009).

5 RELIABILITY-BASED GEOTECHNICAL DESIGN

Conventional geotechnical engineering design is based on the allowable stress design / working stress design approach, where the uncertainties in soil properties, loads and modelling are implicitly accounted for by a factor of safety. The factor of safety is defined as the ratio of the characteristic resistance to the characteristic load as calculated by an idealised model. This approach does not address the uncertainty in load and resistance in a consistent manner. The ambiguous definition of "characteristic" values allows the engineer to implicitly account for uncertainties by choosing conservative values of load (high) and resistance parameters (low) for example for a bearing capacity problem. The choices, however, are quite arbitrary. Designs with nominally the same factor of safety could have significantly different safety margins because of the uncertainties and how they are dealt with. Duncan (2000) pointed out that "Through regulation or tradition, the same value of safety factor is often applied to conditions that involve widely varying degrees of uncertainty. This is not logical." Lacasse and Nadim (2011) provided the example shown in Figure 4 for the analysis of a slope. The example clearly demonstrates that a low safety factor does not necessarily correspond to a high probability of failure and vice versa. Both the margin of safety (how far one is form failure) and probability of failure depend on the uncertainties in the analysis parameters.

Probability theory and reliability analyses provide a rational framework for dealing with uncertainties and making decisions under uncertainty. Depending on the level of sophistication, the analyses provide one or more of the following outputs:

- Probability of failure (or probability of unsatisfactory performance)
- Reliability index
- The most probable combination of parameters leading to failure
- Sensitivity of result to any change in parameters

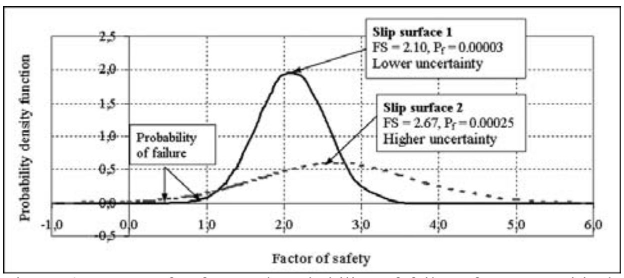

Figure 4. Factor of safety and probability of failure for two "critical" slip surfaces (Lacasse and Nadim, 2011).

In reliability-based design, the uncertainties in load, resistance and modelling are quantified and the design objective is to ensure that the probability of failure of the component or system under consideration is less than a target value. The reliability-based design approach is well-advanced in structural design practice, and is slowly gaining acceptance for foundation design of critical structures.

More commonly, the reliability-based design approach is "translated" into partial safety factors for geotechnical design such as the LRFD (load and resistance factor design) approach. The partial safety factors in the LRFD design can be calibrated such that a target safety level (i.e. a failure probability less than the target value) is ensured for "typical" levels of uncertainty encountered. However, in specific applications, the uncertainties may be quite different from those assumed in the calibration exercise and one must not forget that the LRDF approach is a user-friendly simplification of the full reliability-based design.

6 EXAMPLE APPLICATIONS

Several examples of application of the reliability approach to actual case studies are described to illustrate the added value of adopting the reliability approach. The cases studies included are:

- Settlement prediction of embankment on soft clay and settlement prediction for offshore structures
- Breach of a dam under operation
- Breach of a dam before construction
- Prioritising mitigation measures for a landslide dam
- Seismic stability of underwater slope
- Earthquake-triggered landslide management
- Partial safety factor for offshore foundation design
- Run-out of quick clay landslide

6.1 *Probabilistic settlement prediction with model and prediction updating*

In many geotechnical designs, it is important to quantify the uncertainties in predicted settlement in order to be prepared for appropriate action if the values predicted values turn out to be significantly higher than the acceptable thresholds.

In 2015, the Australian Research Council Centre of Excellence for Geotechnical Science and Engineering (CGSE) invited practice and academia to make predictions of the time-dependent settlement, pore pressure and horizontal displacement of the Ballina test embankment in New South Wales (NSW) in Australia. Around thirty teams made Class A prediction. A Class A prediction is a prediction without any knowledge of the observations (Lambe, 1973). The deterministic Class A predictions by almost all predictors underestimated the measured settlement after three years because of too high stiffness below the estuarine clay and too low shear deformation in the estuarine clay, and wrongful estimate of the permeability (or coefficient of consolidation). The predicted time-settlement curve was also rapid. CGSE requested NGI to prepare an estimate of the uncertainties in the prediction of settlement, pore pressure and horizontal displacement using probabilistic approaches. The Ballina test embankment provides the profession with a verification of how well one can predict the in situ behaviour. NGI combined two probabilistic approaches (Monte Carlo simulations and the first-order second moment approach) with the finite element program Plaxis 2D (www.plaxis.nl) to obtain the mean and variance of predicted settlement, horizontal displacement and pore pressure under the Ballina test embankment. The results of the Class A probabilistic prediction are shown in Figure 5. Although the statistical mean also underpredicted the settlement (it used the same mean of the parameters as the deterministic calculation, the actual observed settlement was within one standard deviation of the probabilistic prediction.

The results of the Class A and Class C predictions (a Class C prediction is a prediction after knowing the observations made; Lambe, 1973), where the possible range of predicted settlement, horizontal displacement and pore pressure under the CGSE Ballina test embankment are compared to the measurements, are presented in Liu et al. (2017). The Class C prediction, with a more detailed probabilistic model, resulted in a very good match of the measured behaviour with time.

Ronold (1989) did a probabilistic assessment of the consolidation settlement of a gravity base platform with a skirt foundation installed in a clayey soil deposit in the North Sea. He used the first-order reliability method (FORM) to estimate the probability of platform settlement exceeding 0.50 m during its design lifetime of 50 years. He then updated the calculations based on the observed platform settlement 10 years after its installation.

To do this, the Bayesian updating method was used. Bayes' theorem allows one to combine in a systematic way subjective judgment with observational data. Judgment is necessary in any geotechnical design, whether it is deterministic or probabilistic. In fact, Bayesian theory provides an ideal framework for updating the geotechnical design predictions based on field observations.

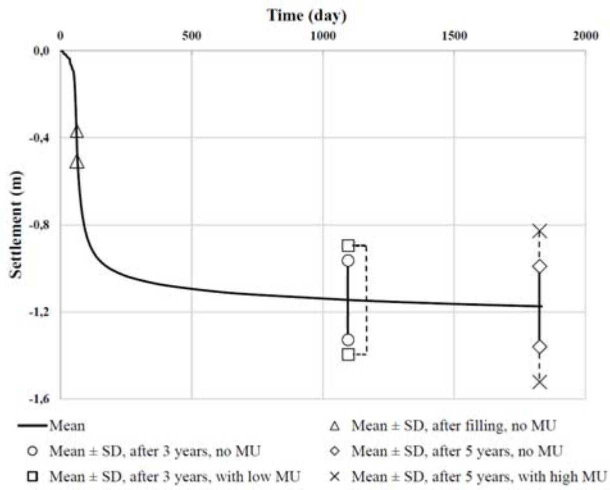

Figure 5. Predicted mean ± one standard deviation settlement at end of filling, after 3 years and after 5 years.

Tan and Nadim (1992) did a similar exercise for the Gullfaks A platform, a concrete gravity base structure that was installed in the northern North Sea in May 1986. Tan and Nadim used a Bayesian framework for updating the settlement predictions based on the measured settlements during the first few years after platform installation. The probabilistic consolidation analysis including model and prediction updating, is a powerful tool for decision-making in the offshore industry. The probabilistic estimates allow for taking early action if the observed settlement measurements imply an unacceptably high probability of exceeding a critical settlement during the lifetime of the platform.

Cassidy et al. (2015) presented "real-life" examples of the updating of the bearing capacity (load-displacement) curves of spud cans for offshore mobile units (jack-up platforms). This is an obvious real time application of great use, since jack-up penetration is often subject to accidents. Another very useful application of the Bayesian updating method for decision-making pertains to the evaluation of the feasibility of lifetime extension for offshore platforms.

6.2 *Risk analysis of an existing dam*

The concepts of probabilistic risk analyses for dams have been around for a long time (e.g. Whitman, 1984; Vick and Stewart, 1996). This example illustrates that the event tree analysis is a systematic application of engineering judgment. Its application does not require the prior existence of extensive statistics or the application of complex mathematics. The process provides meaningful and systematic estimates and outcomes on the basis of subjective probabilities. Lacasse et al. (2017a) presented one of the most recent examples of risk analysis for a rockfill dam in western Norway.

The Dravladalen Dam has a height of 29 m and reservoir of $58{\cdot}106$ m^3. A simplified cross-section of the dam is shown in Figure 6. The rockfill dam was built in the period 1971−1972. It is founded on rock and had a moraine core. The dam is 340m long, with crown at Elevation 962 m above sea level. The top of the dam has a width of 7.5 m. The dam was designed for a 1,000-year flood (Q_{1000}). Leakage was observed from the early stages of impoundment, but only small deformations were recorded. The "normal" leakage through the moraine core, based on laboratory leakage measurements, was about 3 to 8 l/s. In 1994, the recorded leakage was 11-13 l/s. In 2016, the leakage under full reservoir was on average 5 to 6 l/s, and the water was clear (no discernible fines).

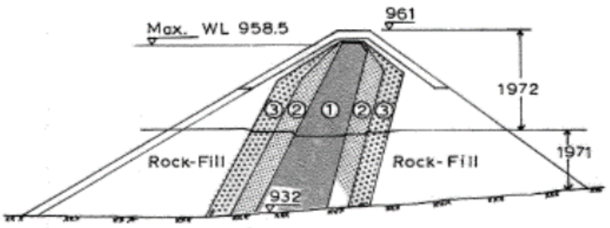

Figure 6. Cross-section of Dam Dravladalen. Zone 1: Core, Zone 2: Filter, Zone 3: Transition (NGI, 1996).

In 1996, a risk analysis of Dam Dravladalen led to the identification of an unforeseen mode of failure, which was revealed at the time to be the most critical failure mode. The failure mode was extreme flooding during late winter while ice and compacted drifted snow were blocking the spillway approach channel. Based on the results of the risk assessment in 1996, remediation measures were designed and implemented during the next 16 years. The rehabilitation of the dam included:

- New toe for the dam to increase drainage capacity.
- New slope protection downstream, with gentler slope.
- New crest for the dam.
- New shelter for the approach to the spillway tunnel.
- Additional downstream slope protection.
- New leakage monitoring system.
- Instrumentation of upstream slope and dam top.

In 2016, the dam owners wanted to see whether or not reliability analyses could demonstrate the effectiveness of the remediation measures on Dam Dravladalen. A new risk assessment was therefore done with two approaches: Event tree analysis and Bayesian network combined with Monte Carlo simulations.

An event tree analysis (ETA) is a form of "what if" analysis, where one looks at different triggers and possible mechanisms that could lead to failure, and follows the process from its initiation, through its continuation and progression until a potential breach. The event tree approach breaks down the potential outcome into a chain of events. It is a practical visual representation of several sequences of events that can lead to failure. The probabilities at each node of the tree can be obtained from statistical estimates based on past observations (actual data), engineering models based on physical processes (including parameter uncertainties), and expert judgment based on knowledge and evaluated experience. In his state-of-the-art and practice for risk analysis of embankments dams, Hartford (2008) identified the roles scientific inference and (engineering) judgement in the risk analysis process as an area of concern. He noted that scientists are sceptical to extensive use of judgement in risk analysis for dams, being concerned that "engineering judgment means that they just make up the numbers". However, Vick (2002) argued that: "The collective judgment of experts, structured within a process of debate, can yield as good an assessment of probabilities as mathematical analyses".

A Bayesian network (BN), also called belief network, is an emerging method for reasoning under conditions of uncertainty and for modelling uncertain domains. The method has been applied to a number of civil and environmental engineering problems. For example, avalanche risk assessment (Grêt-Regamey and Straub, 2006), design of early warning system for landslide hazard mitigation (Medina-Cetina and Nadim, 2008), rock slope failure assessment (Einstein et al. 2010), dam risk analysis (Smith, 2006), earthquake risk management (Bensi et al., 2011) and multi-hazard and multi-risk assessment (Liu et al., 2015). Each variable in the network is defined in a discrete and finite outcome space (discrete random variable) or as a continuous outcome space (continuous random variable). One important property of the Bayesian network is that the joint probability function of all random variables in the network can be factorized into conditional and unconditional probabilities in the network (Jensen, 2007).

For risk assessment of dams, both ETA and BN comprise eight steps (after Vick, 2002; Høeg, 1996):

1. Review of field performance and earlier case histories.
2. Dam site inspection and data review.
3. Failure mode screening.
4. Agreement on descriptors of probabilities.
5. Event tree construction and probability assessment.
6. Calculation of annual probability of breach
7. Evaluation of results.
8. Iteration and documentation.

The failure mode screening, event tree construction and probability assessment are best done by bringing together a number of "experts" with knowledge on the dam, the hazards and risk involved and knowledge on dam construction and behaviour in general. The format of a workshop is often useful to assess and discuss the estimates of probability. The 2016 ETA workshop for risk analysis of Dravladalen Dam assembled 18 specialists, including dam owners, engineers responsible for the dam operation, flood specialists, earthquake specialists, reliability specialists, consultants, authorities, academia (with dam expertise).

As mentioned earlier, in any ETA, engineering judgement needs to be translated into probabilities. In the analyses of Dravladalen Dam in 2016, the values in Tables 1 and 2 were used to assign probabilities to the different events in 2016. Table 1 lists the "traditional" guidelines (Høeg, 1996; Vick, 2002), whereas Table 2 is an adaptation of the IPCC (2012) recommendation, where a range of probabilities are used to reflect an uncertainty in the estimates. The values in Tables 1 and 2 were used to evaluate the probabilities for the different events in both the ETA and the BT analyses of Dravaldalen Dam in 1996.

Table 1. Estimate of probabilities and verbal description for the event tree analyses.

Probability	Verbal description
0.001	Virtually impossible, due to known physical conditions or process that can be described and specified with almost complete confidence.
0.01	Very unlikely, although the possibility cannot be ruled out on the basis of physical or other reasons.
0.10	Unlikely, but it could happen.
0.50	As likely as not, with no reason to believe that one possibility is more or less likely than the other.
0.90	Likely, but it may not happen.
0.99	Very likely, but not completely certain.
0.999	Virtually certain, due to know physical conditions or process that can be described and specified with almost complete confidence.

The following cases were analysed in 2016: 1) Internal erosion; 2) Flooding, with two seasons, a) during winter, with ice and hard-packed snow blocking the spillway tunnel, and b) during the summer, with glacier melting in the reservoir (including climate change); 3) Earthquake; and 4) Sabotage/terror event.

Comparison of the results of the event tree analyses in 1996 and 2016 showed that the estimated annual probability of failure for internal erosion and flooding modes of failure was reduced by one to two orders of magnitude because of the rehabilitation measures. The annual failure probability for all geotechnical and natural hazards scenarios was estimated to be $P_f = 1{\cdot}10^{-5}$/year in 2016 compared to $P_f = 4{\cdot}10^{-3}$/year in 1996.

Table 2. Estimate of probability ranges and verbal description for the event tree analyses.

Probability	Verbal description
~ 0.0 – 0.005	Virtually impossible, due to known physical conditions or process that can be described and specified with almost complete confidence.
0.005 – 0.02	Very unlikely, although the possibility cannot be ruled out on the basis of physical or other reasons.
0.02 – 0.33	Unlikely, but it could happen.
0.33 – 0.66	As likely as not, with no reason to believe that one possibility is more or less likely than the other.
0.66 – 0.98	Likely, but it may not happen.
0.98 – 0.995	Very likely, but not completely certain.
0.995 – ~ 1.0	Virtually certain, due to know physical conditions or process that can be described and specified with almost complete confidence.

The analyses with the Bayesian network (BN) were combined with Monte Carlo simulations using the ranges of values in Table 2. They gave essentially the same 'mean' annual probability of failure as those obtained with the ETA. However, the BN analyses also provided the distribution of the probabilities, with a mean value of P_f, maximum value and minimum value P_f, as illustrated in Figure 7 for the scenario of 'ice and hard-packed snow blocking the spillway tunnel'. Figure 7 gives the histogram of annual probabilities of failure and the best lognormal distribution fit, and gives the number (N) of Monte Carlo simulations done.

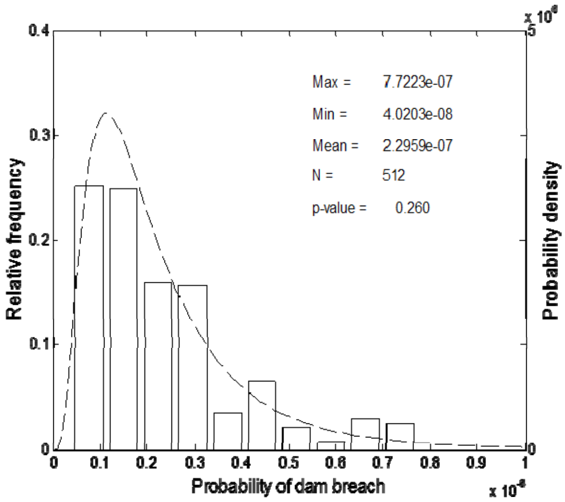

Figure 7. Annual probability of failure from Bayesian Network analysis combined with Monte Carlo simulations.

The application of reliability concepts can be useful for ensuring safe and cost-effective dam design. The annual probability of failure for Dam Dravladalen in 2016 was estimated significantly lower that what it was in 1996, demonstrating quantitatively the effectiveness of the remedial measures implemented in the period 1996-2012. The 1996 analyses identified a new failure scenario ('ice and hard-packed snow blocking spillway tunnel'), which had been overlooked in the deterministic design. This is discussed further in Section 8.3 of the paper.

6.3 *Risk analysis of a dam during design*

Lacasse and Nadim (2011) presented an example of the estimation of the annual probability of non-performance of a new tailings management facility at Roşia Montană in Romania (www.gabrielresources.com/prj-rosia.htm). The analyses established whether the dam provides acceptable safety against release of tailings and toxic water, and whether additional hazard reducing measures are needed. The project lies within the existing Roşia Montană mining district north-east of the town of Abrud in the Apuseni Mountains of Transylvania. The project aims at mitigating the consequences of the historic and future mining operation with the interception and containment of contaminated water currently entering the system, treatment of the contaminated waters and isolation and recovery of the waste rock piles within the project boundary. The operation of the project will generate tailings for approximately 17 years, producing tailings from the processing of approximately 215 Mt of ore. The proposed mining and processing operation requires the construction and operation of a Tailings Management Facility (TMF) in the valley. The TMF (Fig. 8) includes a 90-m high Starter Dam as a first stage of the Completed Dam, a Secondary Containment Dam (SCD), a tailings delivery system, a reclaim water system and a waste rock stockpile. The completed 180-m high TMF (called the Corna Dam) is designed as a depository for the treated tailings residue of a gold mine. The Corna Valley TMF site is to provide the required design storage capacity for the life of the mine, plus an additional contingency capacity.

To establish whether the dam provides acceptable safety against "uncontrolled" release of tailings and water during its life, an event tree approach was used to do the hazard analyses. The event tree hazard analyses considered the dam at different stages of its life and estimated the probability of non-performance. A non-satisfactory performance of the dam was defined as an uncontrolled release of tailings and water from the dam over a period of time. The analyses looked at critical scenarios, including all potential modes of non-performance for the dam under extreme triggers such as a rare, unusually strong earthquake and extreme rainfall in a 24-hour period.

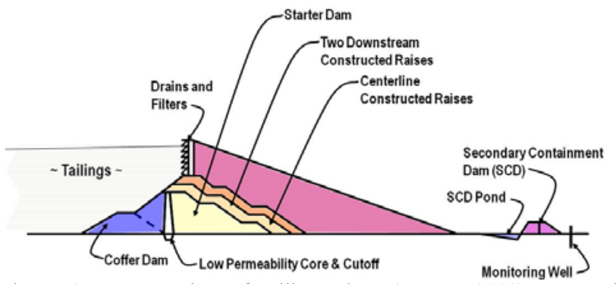

Figure 8. Cross-section of tailings dam (Corser, 2009). Personal communication. MWH Americas, Inc.).

Similar to the process described in the previous section, an event tree workshop was organised to develop the event trees and reach a consensus when quantifying the hazards. The analyses involved breaking down the complex system into its fundamental components, and determining the potential "failure" mechanisms leading to non-performance of the dam and the physical processes that could cause such mechanisms. The "non-performance modes" considered included:

- Foundation failure due to e.g. excess pore pressures or weak layer in foundation leading to cracking, instability and breach of the dam.
- Dam slope instability downstream or upstream, due to e.g. high construction pore pressures in core of Starter Dam, excessive pore pressures caused by static or earthquake loads, or instability due to inertia forces.

- Unravelling of downstream toe and slope due to e.g. overtopping or excessive leakage through or under the dam. This can be caused by a slide into the depository, dam crest settlement due to deformations of the Starter Dam, piping, internal erosion and sinkhole formation, or excessive deformations (slumping) of the top vertical part of the Completed Dam during earthquake shaking.
- Dam abutment failure followed by breach due to e.g. slide close to and/or under part of the dam.
- Liquefaction of the tailings.

The analyses looked into construction deficiencies, e.g. filter material segregation leading to uncontrolled internal erosion, inadequate drainage, very weak construction layers or zones in the embankment, inadequate types of material(s) in the embankment fill, or insufficient quality control and unforeseen construction schedule changes which often happen in mining operations. These conditions were integrated in the event trees as separate events during the course of the construction of the Starter Dam and Completed Dam.

The analyses that were prioritised through failure mode screening were grouped by triggering event and by the dam "Configuration" analysed, the Starter Dam, the Completed Dam or an intermediate construction stage. Event trees were developed for each trigger, with each non-performance mechanism looked at separately. In some cases, two non-performance mechanisms were considered successively.

The total probability of non-performance is the sum of all contributing probabilities to the non-performance for each of the dam configurations. The estimated probabilities were presented as a function of the release of tailings and water associated with the non-performance of the Dam. The highest estimated probabilities of non-performance for the completed dam were associated with earthquake shaking of the main dam and the static liquefaction of the tailings at time 9 to 12 years. The scenarios would result in some material damage and some contamination, but only in the vicinity downstream of the dam. For the Starter Dam, no plausible expected scenario led to a significant release of tailings and water because of the limited quantity of water available and the reserve capacity provided (2 PMP's). Essentially all material released could be contained by the Secondary Containment Dam.

The analyses showed that no sequence of plausible accidental events results in a probability of non-performance of the dam greater than once in a million years (or 10^{-6}/yr). For a lifetime of say 20 years, the probability of non-performance is $2{\cdot}10^{-5}$. The estimated probabilities of non-performance were lower than what is considered acceptable as design criteria for dams and other containment structures around the world.

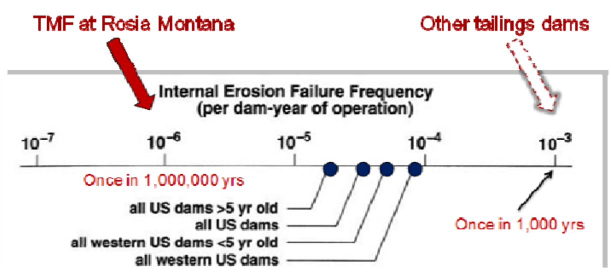

Figure 9. Annual probability of failure for different types of dams.

Figure 9 compares the annual probability of non-performance estimated the TMF at Roşia Montană with that of water retention dams in the USA and other tailings dams around the world. The factors that contribute to the low estimated probability of non-performance include the use of good quality rockfill for the downstream shoulder of the dam, gentle downstream slopes for both the Starter and the Completed Dam, dam capacity to store extreme precipitation and/or snowmelt events, spillway to release excess water in a controlled manner, the planned safety monitoring to warn of any early signs of unexpected performance, and the proposed preparedness to remediate any indication of unexpected behaviour. At the time there were no statistics for tailings dams. They are however know to fail more often than water-retaining dams. An estimate of the annual probability of failure of tailings dams is also shown in Figure 9.

6.4 *Design of risk mitigation measures for landslide dam*

This case study describes the risk mitigation measures that have been implemented and are being considered for the Usoi landslide dam and Lake Sarez in Tajikistan. The case study is presented in detail in Lacasse and Nadim (2011).

Lake Sarez is located in the Pamir Mountain Range in eastern Tajikistan. The lake was created in 1911 when an earthquake triggered a massive rock slide (volume: ~2 km^3) that blocked the Murghab river valley. A landslide dam, Usoi Dam, was formed by the rockslide which retains the lake. The dam at altitude 3200 meters has a height of over 550 meters, and is by far the largest dam, natural or man-made, in the world. Lake Sarez is about 60 km long, with maximum depth of about 550 m and is currently retaining 17 km^3 water. The lake has never overtopped the dam, but the current freeboard between the lake surface and the lowest point of the dam crest is only about 50 m. The lake level is currently increasing about 30 cm per year. If this natural dam were to fail, a worst-case scenario would be a catastrophic outburst flood endangering thousands of people in the Bartang, Panj, and Amu Darya valleys downstream.

There is a large active landslide on the right bank (Fig. 10), with observed movement rate of 15 mm/year. If this unstable slope should fail and slide into the lake, it would generate a surface wave large enough to overtop the dam and cause a severe flooding downstream. Experts who studied the hazards agree that the most probable scenario at Lake Sarez is failure of the right bank slope and overtopping of the dam.

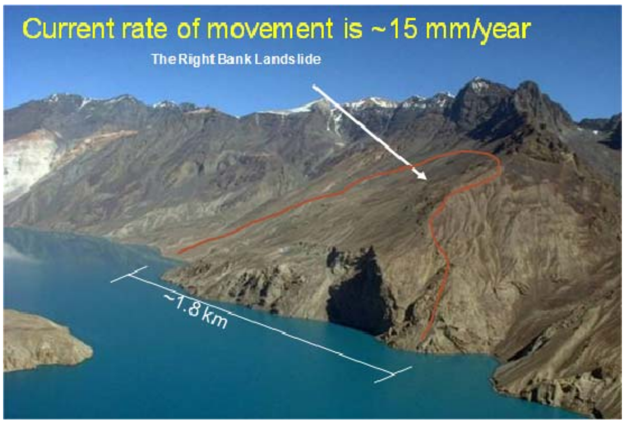

Figure 10. Active landslide on the right side of Lake Sarez.

In 2000, an international "Lake Sarez Risk Mitigation Project" was launched under the auspices of the World Bank to deal with the risk elements posed by Usoi dam and Lake Sarez. The objective of the project was to find long-term measures to minimize the hazard and to install an early warning system to alert the most vulnerable communities downstream. The early warning system for Lake Sarez has been in operation since 2005, with 9 remote monitoring units linked to a central data acquisition system at a local control centre near the dam (Stuckey, 2007). Data are transmitted via satellite to the main control centre in Dushanbe, Tajikistan's capital. Alerts and warning messages are sent from Dushanbe to 22 communities connected to the system. The local control centre is manned 24 hours per day, every day. The warning system has three alarm levels. Each level is based on monitored data and/or visual observations. Threshold values for triggering alarms include both maximum measured values and rate of change with time. Alarm states and emergency warning plans have been established. The main problem has been insufficient power in some of the remote villages. The system was turned over to the Ministry of Defence who now has responsibility for its operation. The plan is to keep the early warning system in operation until 2020 which is the target date for completion of the mitigation works.

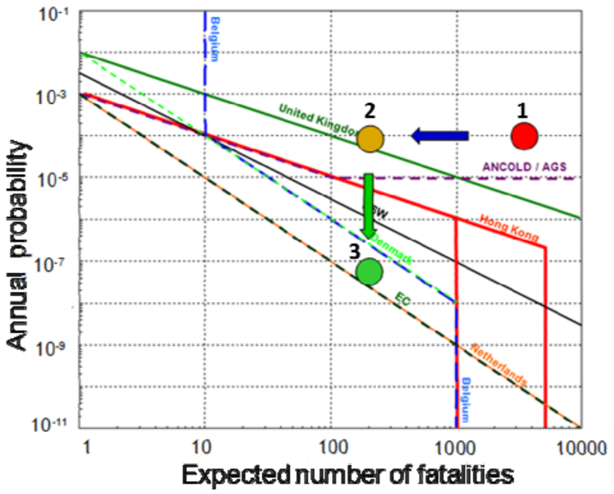

Figure 11. Annual probability of failure for different types of dams.

Figure 11 illustrates the effect of mitigation on the risk level. The acceptable risk criteria discussed in Section 4 are shown in background. The first estimate of the risk associated with the Sarez landslide dam, without mitigation, gave a risk level [P_f; number of fatalities] of [10^{-4}/yr; 5000 fatalities (circle 1)]. This risk was reduced to [10^{-4}/yr; 200 fatalities (circle 2)] with the installation of the early warning system, and would be reduced to [10^{-7}/yr; 200 fatalities (circle 3)] by in addition the lowering of the lake reservoir. A permanent lowering of the lake reservoir by about 120 m using a diversion tunnel around the landslide turned out to be the most cost-effective mitigation measure. The possibility of at the same time producing electrical power is being evaluated, but the transmission of power to potential users also presents a big challenge with the near unsurmountable mountainous terrain (Fig. 10).

6.5 *Probabilistic analysis of seismic stability of underwater slope*

The stability of submarine slopes under earthquake loading is a challenging issue in offshore geohazards studies. This is especially the case where seafloor installations such as platforms and pipelines are founded on a slope or within the potential run-out distance of a failed slope. Risk assessment for sea floor installations exposed to offshore geohazards requires an estimation of the occurrence probability of a hazardous event during a reference time period, for example the annual probability of the installation being impacted by a submarine mass gravity flow.

Nadim et al. (2014) described the procedures for the estimation of the temporal probability in two situations: 1) When there is a clear trigger for initiating the slide that would develop into a mass gravity flow, and 2) When there is evidence of slide activity, but no obvious trigger for slide initiation.

In the first situation, the assessment of temporal probability requires a probabilistic description of the frequency and intensity of the trigger(s) releasing the submarine slide, a probabilistic model for calculating the response of the slope to the trigger, and a probabilistic model for evaluating the runout of the released mass gravity flow. Using these models, the probability of a mass gravity flow affecting the seafloor installation(s) should then be computed for all relevant scenarios and return periods in order to derive the annual or lifetime probability. However, analysing all possible scenarios and return periods could be time-consuming and impractical. Nadim et al. (2014) presented a simplified procedure and demonstrated its application through a case study for earthquake-triggered slides.

Three scenarios of earthquake-induced slope instability should be assessed for submarine clay slopes (Biscontin et al., 2004):

1. Failure occurs during the earthquake: in this scenario, the excess pore pressures generated by the cyclic stresses degrade the shear strength so much that the slope is not able to carry the static shear stresses.
2. Post-earthquake failure due to increase in pore pressure at critical locations caused by seepage from deeper layers.
3. Post-earthquake failure due to creep.

Soils with strong strain-softening and high sensitivity are most susceptible to failure during earthquake shaking: excess pore pressure migration from deeper layers into critical areas, leading to instability, could occur over a time span of years or even decades in deep marine clay deposits. Post-earthquake creep-type failure is believed to be the most common mechanism for clay slopes.

Special cyclic direct simple shear tests (DSS) run on marine clay specimens in laboratory suggest that the earthquake-induced shear strains correlate well with the reduction of the pre-cyclic (static) undrained shear strength (Nadim et al., 2005). Figure 12 presents the results of such tests on marine clay from the Ormen Lange field in the North Sea (Nadim et al., 2005). The shear strain-shear strength ratio type of diagram in Figure 12 is in fact the original method of cyclic strength presentation introduced by Professor Harry Seed in the early 60's. In the diagram, γ_{cy} is the cyclic shear strain, N the number of cycles and γ_{tot} the total (cyclic + average) shear strain.

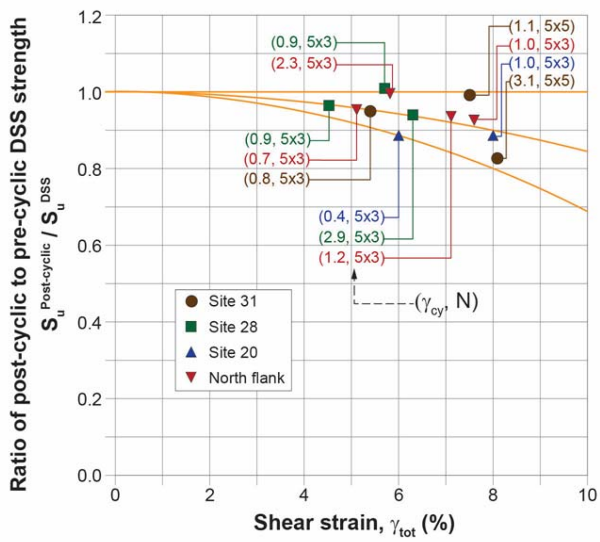

Figure 12. Effect of permanent strains during cyclic loading on post-cyclic undrained shear strength, special cyclic DSS-tests on clay (Nadim et al., 2005).

Nadim et al. (2014) calculated the annual probability of earthquake-induced slope failure using a procedure developed through a number of joint-industry research projects and offshore geohazards studies in the North Sea, the Caspian Sea, the Black Sea, offshore Indonesia, and the Gulf of Mexico. The approach accounts for the uncertainties in all steps of the assessment and utilizes the available information to come up with a rational estimate. The analysis has eleven steps:

1. Identify the critical slopes and establish the geometry and mechanical soil properties for the critical slopes in a probabilistic format.
2. Use Monte Carlo simulation, FORM or other technique to compute the cumulative distribution function (CDF) of the static safety factor for the slope, F_{FS}.
3. Update the CDF for static safety factor using the fact that the slope is standing today. This implies that the current factor of safety, although unknown, is greater than unity. The annual probability of failure becomes the question of the likelihood that the current factor of safety will fall below unity during a reference time of one year. Its probability distribution can be computed (from FORM analysis or Monte Carlo simulation), but is truncated to reflect that the slope is stable today. This is basically a Bayesian updating procedure where the a-priori information is that FS ≥ 1. The updated (or posterior) distribution of the factor of safety is

   $$P[FS<z \mid FS\geq 1] = [F_{FS}(z) - F_{FS}(1)]/[1 - F_{FS}(1)] \quad (1)$$

   Seismic slope failure occurs if the safety factor falls below unity as a result of the earthquake loading effects.
4. Do a probabilistic seismic hazard assessment and identify representative acceleration time histories for return periods of interest.
5. Establish the reduction in the post-earthquake undrained shear strength as a function of the maximum earthquake-induced shear strain from laboratory tests or literature survey (e.g. Fig. 12).
6. Perform dynamic response analyses for various combinations of dynamic soil properties and representative earthquake ground motions using the Monte Carlo simulation technique. The analyses should be done for at least two return periods, as discussed in Step 9.
7. Using Steps 5 and 6, establish the distribution function for the undrained shear strength reduction factor, reflecting the effects of earthquake loading.
8. From Steps 3 and 7, establish the CDF for post-earthquake static safety factor. The conditional probability of failure (given that the earthquake with the specified return period occurred) is the value of this CDF at FS equal to 1.
9. The annual failure probability is the sum (integral) of all conditional failure probabilities for a given return period, divided by that return period. The analyses should be done for at least two return periods, ideally above and below the return period that contributes most to the annual failure probability. Iteration might be necessary as this is not known beforehand.
10. With the result of Step 9, establish a model with load and resistance that matches the computed failure probabilities at the return periods of interest. The most usual load parameter is the input annual peak ground acceleration (PGA), with typically exponential or Pareto distribution. If PGA is used as the representative load parameter, the slope resistance needs to be specified as an acceleration parameter. A log-normal distribution for resistance is commonly assumed.
11. Estimate the probability that the resistance of the slope is less than the applied load (e.g. the annual PGA), which is the annual probability of earthquake-induced slope failure.

The example below exemplifies the application of this procedure to a slightly overconsolidated clay slope in a moderately seismic area. Figure 13 shows the results of the probabilistic seismic hazard study (Step 4). A linear relationship in the log-log plot of PGA vs. Annual Exceedance Probability implies a Pareto distribution for the annual PGA. The earthquake events with return periods between 1,000 and 10,000 years contribute most to the annual failure probability. The dynamic response analyses were, therefore, done for earthquake events with return periods of 3000 and 10000 years. Each of these events was represented by 4 sets of properly scaled acceleration time histories.

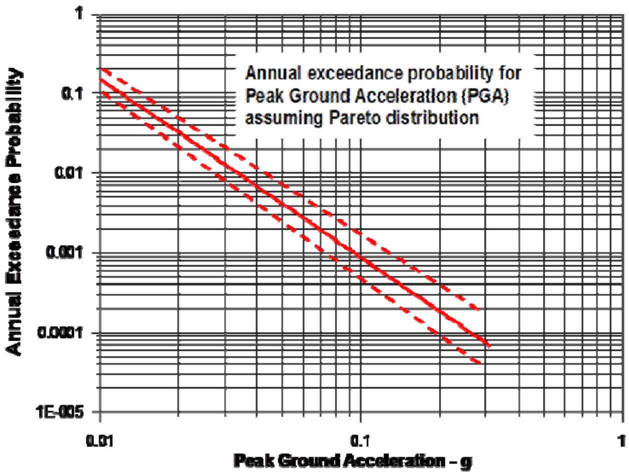

Figure 13. Calibrated (for return periods 1,000 to 10,000 yrs) Pareto distribution with $\mu = 0.0077$g and $\sigma = 0.0106$g for annual PGA (A_{max}). Dashed lines represent ± 1 standard deviation with respect to the best estimate (Nadim et al. 2014).

Figure 14 shows the histograms of the shear strength reduction factors obtained from the simulations and a fitted distribution function to the data. The computed and the updated CDFs for the static safety factor under undrained loading prior to the earthquake (Steps 2 and 3) using the FORM approximation are shown on Figure 15.

To estimate the annual probability of slope failure (Step 9), a simplified model similar to that suggested by Cornell (1996) was developed. The limit state function in Eq. 2 with the parameters in Table 5 and the FORM approximation were used to estimate the annual failure probability and reliability index (defined as $\beta_{annual} = \Phi^{-1}[1- P_{f,annual}]$):

Annual probability of failure: $P_{f,annual} = 3.7\cdot 10^{-4}$
Annual reliability index: $\beta_{annual} = 3.4$

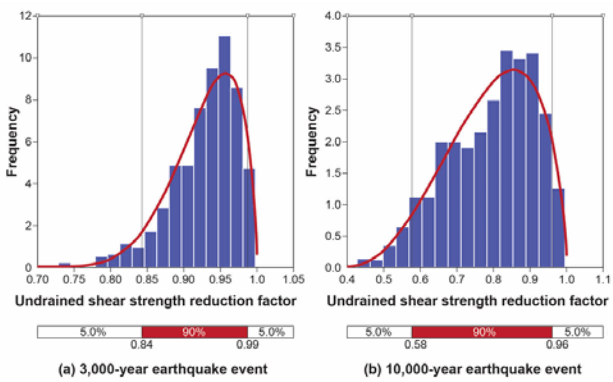

Figure 14. Histograms and best-fit distributions of earthquake-induced undrained shear strength reduction factors (Nadim et al. 2014).

It should be noted that the estimated annual probability above is for the initiation of a slide event that could potentially impact the critical facilities. The probability that the critical facilities will actually be impacted is less than this value and depends on the results of probabilistic mass gravity flow run-out analyses.

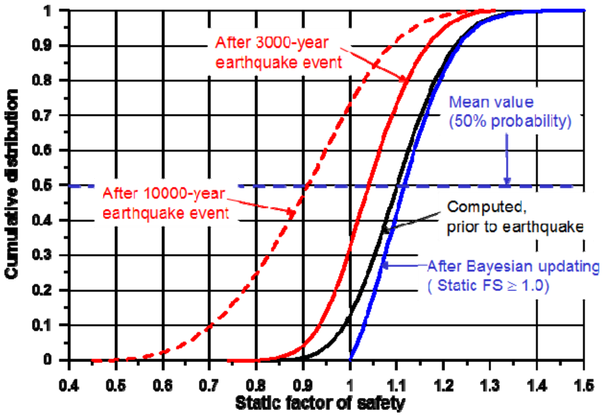

Figure 15. Results of probabilistic analyses of static undrained stability, prior to and after the 3000-year and 10000-year earthquake events (Nadim et al. 2014).

6.6 *Calibration of partial safety factors for offshore foundation design*

The example applications presented so far demonstrate the added value and potential advantages of reliability-based design (RBD) over traditional deterministic design in geotechnical engineering. Unfortunately, most practicing engineers are not familiar with the procedures of RBD, which typically involve one or more of the following methods: first-order second moment approximation (FOSM), first- and second-order reliability methods (FORM/SORM), Monte Carlo simulation (MCS) and event tree analysis (ETA). Simplified RBD methods (or semi-probabilistic methods) can be adopted to overcome this difficulty by producing design code formats that have a look and feel similar to the traditional geotechnical design codes (Ching and Phoon, 2012). The partial safety factors in the load and resistance factor design (LRFD) can be calibrated to achieve a target reliability level for some classes of geotechnical design problems. Hence, the LRFD approach may be considered a simplified RBD method.

Lacasse et al. (2016) presented an approach for the calibration of the load and resistance factors for offshore pile foundations. To evaluate the required resistance factor required for the axial capacity of tubular steel piles for offshore installations, the annual probability of failure was calculated for piles designed with the API and four newer CPT-based methods. The paper presented the calibration approach and illustrated the results with the design of three piled jackets.

The foundation of offshore installations must be designed to resist several combinations of static loads (weight of superstructure, buoyancy, etc.), P_{stat}, and environmental (dynamic) loads (wind, waves, earthquake, ship impact, etc.), P_{env}. Typically, the design equation in standards has the following format:

$$\gamma_{l\,stat} \cdot P_{stat} + \gamma_{l\,env} \cdot P_{env} < Q_{ult} / \gamma_m \qquad (2)$$

where $\gamma_{l\,stat}$ is the load factor on the characteristic static load, $\gamma_{l\,env}$ is the load factor on the characteristic dynamic load, γ_m is the resistance factor, and Q_{ult} is the characteristic foundation capacity under the applied loads. For storm loading, a return period for P_{env} of 100 years is typically used for checking the ultimate limit state of offshore structures in the North Sea.

There will always be a finite probability that the loads can cause damage or collapse of an offshore structure. Defining the level of finite probability of failure that is tolerable is a key challenge in the derivation of partial safety factors. Equation 2 should ensure that the annual probability of foundation failure is less than a target value, typically in the range of 10^{-4}/yr (NORSOK 2004; 2007) to $2.5{\times}10^{-4}$/yr (ISO 2007).

In the LRFD approach, the designer accounts for the uncertainties by introducing appropriate partial safety factors in design. The goal of the study by Lacasse et al. (2016) was to recommend the 'appropriate' resistance factor for the design of the piles for three offshore jackets such that the annual probability of foundation failure is not greater than 10^{-4}. This goal was achieved through a rigorous RBD approach.

Figure 16 shows the typical loads on a pile in an offshore jacket. In Figure 16, $P_{stat} = P_{static} + W'$ and $P_{env} = P_{ave} + P_{cyc} - P_{static}$. The analyses has six steps:

1. Statistical description of soil and load parameters,
2. Statistical analysis of the model uncertainty,
3. Deterministic analysis of axial pile capacity,
4. Probabilistic analysis of the axial pile capacity,
5. Calculation of the annual probability of failure,
6. Calibration of the required resistance factor for the target annual probability of failure of 10^{-4}.

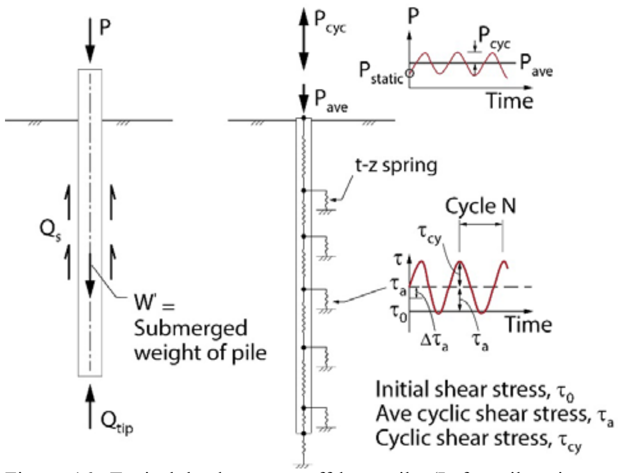

Figure 16. Typical loads on an offshore pile (Left: soil resistance; Right: Cyclic loading).

The calibration of the resistance factor was done for a target annual probability of failure of $P_f = 10^{-4}$ (annual β of 3.75). The calibration can be done with reference to both the axial pile capacity with the characteristic values ($Q_{ult\ char}$) and the mean axial pile capacity ($Q_{ult\ mean}$). The calibration has nine steps:

1. Start with deterministic pile design according to the code-specified load and resistance factors and assess the annual failure probability. Obtain the scaling factor required to shift the PDF from the calculated annual P_f to the target P_f (see Figure 17);
2. Find the ultimate axial pile capacity, $Q_{ult\ mean}$, for the target P_f with the scaling factor;
3. Find the load on the pile (static, P_{stat}, + environmental, P_{env}) (star means at the design point) for the target P_f;
4. Find the ultimate axial pile capacity at the design point, Q_{ult} , for the target P_f;
5. Calculate the required resistance factor for $Q_{ult\ mean}$ for P_{env} ;
6. Calculate the required resistance factor for $Q_{ult\ char}$ for P_{env} (design point);
7. Calculate the load factor, $\gamma_{l\,env}$, on P_{env} at the design point (relative to 100-yr characteristic load);
8. Calculate required resistance factor γ_m for $Q_{ult\ mean}$ for the prescribed load factor, $\gamma_{l\,env}$ ($\gamma_{l\,stat}$ is 1.0);
9. Calculate required resistance factor γ_m for $Q_{ult\ char}$ for the prescribed load factor, $\gamma_{l\,env}$ ($\gamma_{l\,stat}$ is 1.0).

Figure 17 illustrates the calibration in Steps 5 to 9 with PDFs for Q_{ult} and for P_{env}. The overlap of the two PDFs is a measure of the probability of failure. The PDF for the P_{env} was kept the same for the calibration with targets P_{f1} and P_{f2}.

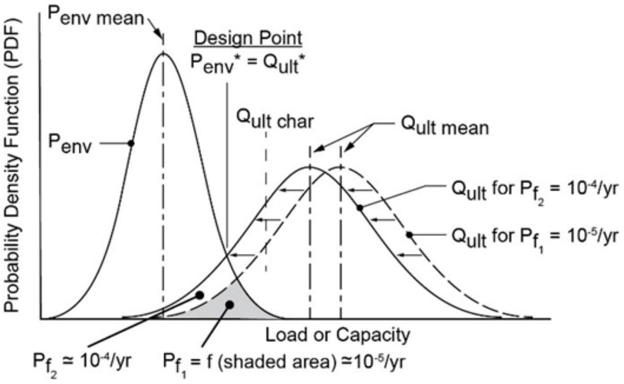

Figure 17. Illustration of the calibration (Lacasse et al 2013).

The resistance factor was obtained based on the axial pile capacity calculated with the characteristic values ($Q_{ult\ char}$). The calibrated resistance factor varied between 1.23 and 1.72 depending on the pile design method and type of soil profile. The calibrated factors reflect the varying influence of the uncertainty in the soil parameters and of the model uncertainties for the different methods. The axial pile capacity methods predicting higher pile capacity require a higher resistance factor to ensure that the probability of failure does not exceed 10^{-4}/yr. The calibrated resistance factor depends on the strength parameters used in the equilibrium equation to do the deterministic analyses, and should be used only with the strength parameters for which it was derived.

Little uncertainty is usually found in the loads induced by gravity (weight of the platform and foundation elements) and buoyancy: $\gamma_{l\ stat}$ was taken as 1.0.

The annual maximum storm-induced load on the foundation was taken to follow a Gumbel (Type I) extreme value distribution. The parameters of the Gumbel distribution were estimated from pairs of the extreme loads, corresponding to different return periods (10, 100, 1,000 and 10,000 yrs).

The results presented by Lacasse et al. (2016) showed that for the three jacket designs, the use of reliability concepts led to significant savings in required minimum pile penetration depths and offshore operation time, thus optimizing safety and costs. They also showed that the calculated annual probability of failure varied with the pile design method. The code-specified load and resistance factors do not always result in consistent annual failure probabilities for offshore foundations. It is possible to calibrate the resistance factors such that a more consistent reliability level is achieved.

Further work is warranted to quantify model uncertainty for pile design. The newer CPT-based methods for pile design need further validation with large scale tests, especially with diameters and loadings comparable to that used offshore. More case studies are needed to draw non-site-specific conclusions, on a variety of soil profiles.

6.7 *Earthquake-triggered landslide risk assessment*

The landslide risk assessment was done with the Bayesian network approach (BN). Nadim and Liu (2013a) provided a brief review of Bayesian networks. Figure 18 presents graphically a simple Bayesian network with five nodes and five arcs. The nodes are: Magnitude (M), Distance (D), Seismic severity (S), Landslide severity (L), and Building damage (B). These nodes are connected via the arcs: M-S, D-S, S-L, S-B and L-B. The user enters evidence, and the information propagates through the network. The probabilities in the network are updated when new information becomes available. The posterior probabilities and joint probabilities are calculated based on the Bayes' theorem (Ang and Tang, 2007).

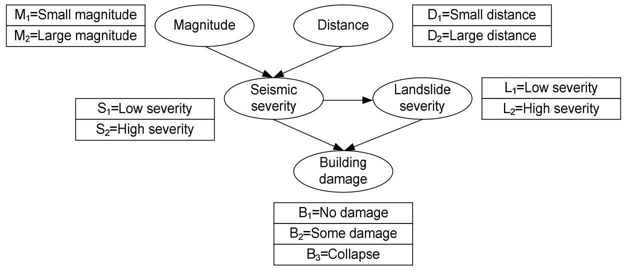

Figure 18. Simple Bayesian network (Nadim and Liu 2013a).

The network in Figure 19 estimates the risk to buildings under an earthquake-triggered landslide. One counts 11 nodes and 16 arcs. Each node has several discrete states. Management includes options of 'no action', 'active' and 'passive' countermeasures, and 'warning systems' (a form of passive measure). Active measures, such as retaining walls and drainage result in lower probability of failure and reduced risk. Passive countermeasures, such as rock fall nets or protective sheds, reduce the vulnerability.

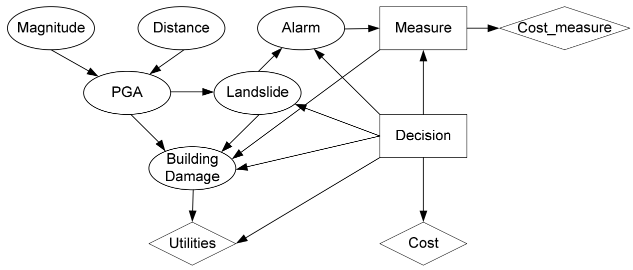

Figure 19. Decision making Bayesian network for earthquake-triggered landslide risk assessment (after Einstein et al 2010).

Figure 20 shows the distribution of the calculated distances to the seismic source taken as a line source. The annual probabilities as a function of the earthquake magnitude Mw (Fig. 21) used the recurrence relationship from Gutenberg and Richter (1994). The conditional probabilities of the peak ground acceleration (PGA), given the magnitude and distance to epicentre, were calculated with Ambraseys et al.'s (2005) equation and Monte Carlo simulations. The joint probabilities of the PGA inferred from the Bayesian network is given in Figure 22.

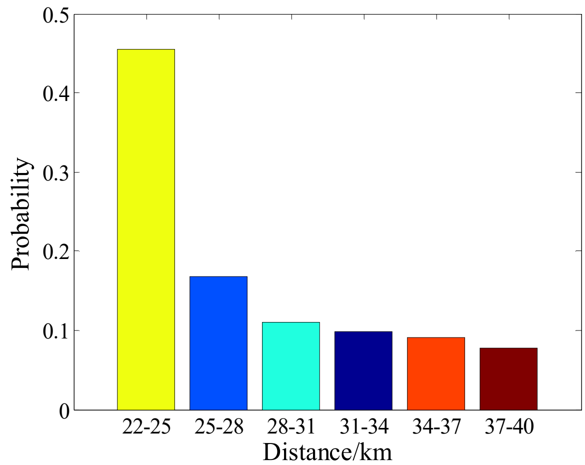

Figure 20. Discrete probabilities of distance to the seismic source (Nadim and Liu 2013a).

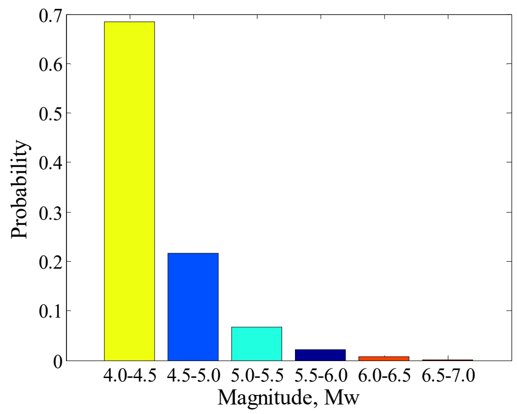

Figure 21. Discrete probabilities of earthquake magnitude (Nadim and Liu 2013a).

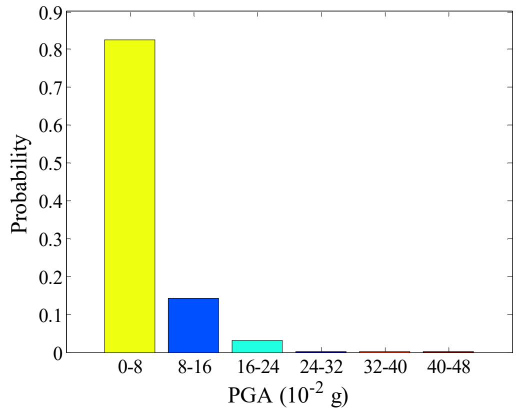

Figure 22. Discrete probabilities of peak ground acceleration (Nadim and Liu 2013a).

The approaches developed to assess the landslide hazard (i.e. the stability of the slopes during earthquake) fall into three analysis categories: (1) pseudo-static, (2) stress-deformation, and (3) permanent displacement. The dynamic slope performance was modelled with the permanent displacement analysis by Newmark (1965). The parameter used in the stability analyses are listed in Table 3.

Table 3. Soil and slope properties (Nadim and Liu 2013a).

Variable	Mean	Standard deviation
c'(kPa)	10	2
φ'(degree)	30	2
z(m)	2.5	0
α(degree)	35	0
γ(kN/m^3)	21.5	0
γ_w(kN/m^3)	10	0

c' is the effective cohesion;
φ' is the effective friction angle;
z is the depth of the failure surface;
α is the slope angle;
γ is the total unit weight of the soil;
γ_w is the unit weight of water

The calculated probabilities of slope failure for different ranges of PGA are listed in Table 4. As mentioned above, countermeasures made to landslide can reduce risk. The probability of slope failure when active actions were used are also listed Table 4.

For a building subjected to a multi-hazard situation involving additive load effects (e.g. earthquake and landslide), the damage was increased. For the other nodes, Nadim and Liu (2013a) adopted the Einstein et al (2010) probability approach and presented the results in tabular form. Table 5 gives an example for conditional probabilities of 'Building Damage'.

Table 4. Computed probability of failure (Nadim and Liu 2013a).

PGA (10^{-2}g)	0-8	8-16	16-24	24-32	32-40	40-48
$P_{f\,no\ action}$	0.124	0.256	0.305	0.328	0.339	0.346
$P_{f,\ active\ actions}$	0.025	0.03	0.035	0.04	0.045	0.05

Table 5. Conditional probabilities of 'Building Damage' for PGA = 0-0.08g (Nadim and Liu 2013a, after Einstein et al 2010).

Parent nodes	Measure	Passive		Active	
	Landslide	Yes	No	Yes	No
Building damage	No damage	0.4	0.1	0.52	0.1
	Some damage	0.3	0.1	0.43	0.1
	Collapse	0.3	0.8	0.05	0.8

Mitigation measures influence the outcome of multi-risk analyses. The results from the Bayesian network of the entire risk assessment and decision are shown in Figure 23 and compared to Einstein et al (2010).

Different mitigation measures result in different utilities. The warning system, showing the lowest (negative) utility, is the most optimal mitigation measure. The expected losses for the four mitigation options increase due to the cascade probability triggered by the earthquake. Neglecting the cascade effect could therefore underestimate the risks.

The parameters in the analysis, e.g. the costs, the probability of slope failure or the reliability of the warning system, can vary. Sensitivity analyses were therefore conducted to assess the effects of these variations on the results.

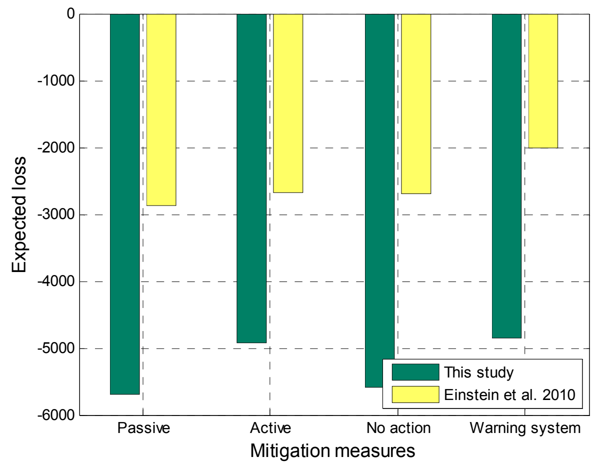

Figure 23. Losses for 'No action', 'Active measures', 'Passive measures' and 'Warning system' (after Nadim and Liu 2013a).

Figure 24 shows the effect of changing the probability of landslide occurrence. In this graph, the best mitigation measure is the one having the less negative utility. For low failure probabilities (P[landslide] < 0.15), no action is preferable, as expected; otherwise, active measures are preferred, except for probabilities between 0.15 to 0.25 where warnings system are slightly preferable to active measures or no action. This is only an example. The sensitivity of the decision to other factors needs to be similarly studied.

As a further application, one can assume that the average unit rebuilding cost for the "collapse" damage state is €200,000, and the average repair costs for the "yielding" damage state as 50% (€ 100,000) of the unit rebuilding cost (Nadim and Liu, 2013b). Figure 25 presents comparative risk curves with and without the cascade effect. The mean expected loss increases for the same return period of the hazard(s) when the cascade effects are included.

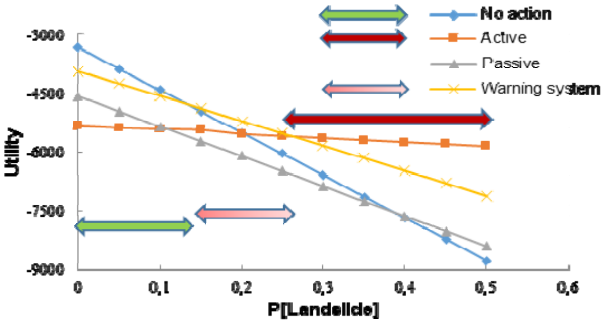

Figure 24. Sensitivity analysis of the risk as a function of the probability of slope failure for different mitigation action (horizontal arrows indicate range where mitigation measures are optimum (Nadim and Liu, 2013a; Lacasse et al., 2013b).

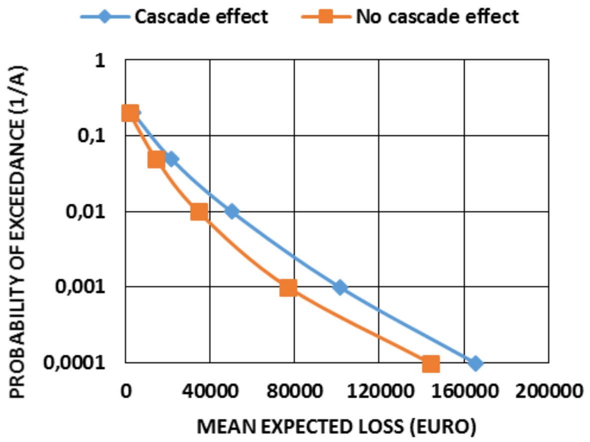

Figure 25. Risk curve with and without cascade effect (Nadim and Liu, 2013b).

6.8 *Run-out of Rissa quick clay landside*

The 1978 quick clay landslide at Rissa is the largest landslide to have struck Norway during the 20th century. Seven farms and five single family homes were taken by the landslide or had to be abandoned for safety reasons. Of the 40 people caught in the landslide, one person died.

A two-stage process of the landslide was described by Gregersen (1981) and was further summarized by L'Heureux et al. (2012b). An initial landslide was triggered by the excavation of clay for the extension of an existing barn and stockpiling the excavated material along the lakeshore. During the initial failure, 70 to 90 m of the shoreline slid out into the lake, including half of the recently placed earth-fill. The landslide edges were 5–6 m high and extended 15–25 m inland. The landslide developed retrogressively in the southwestern direction over the next 40 minutes. The sediments completely liquefied during the sliding and the debris literally poured into the lake like streaming water. At this stage, the landslide area took the shape of a long and narrow pit open towards the lake (Fig. 26). The length of the sliding area was 450 m, covering an area of 25–30,000 m^2 (at that stage, 6–8 % of the final slide area) (Gregersen, 1981).

The main landslide started almost immediately after the retrogressive sliding had reached the boundaries of Stage 1 (Fig. 26). At this point, large flakes of dry crust (150×200 m) started moving towards the lake, not through the existing gate opening, but in the direction of the terrain slope (see flakes A and B, Fig. 26). The velocity was initially moderate (flake A), about 10–20 km/h, but increased to 30–40 km/h (flake B). Houses and farms can be seen floating on top of the sliding masses on the videos of the Rissa landslide. A series of smaller and retrogressive slides followed over a short period of time. The sliding process propagated to the mountain side where it stopped. The main sliding stage lasted for approximately 5 minutes and covered 92–94% of the total landslide area (330,000 m2). The total volume of mobilized sediment was estimated to be between 5 and 6×10^6 m^3.

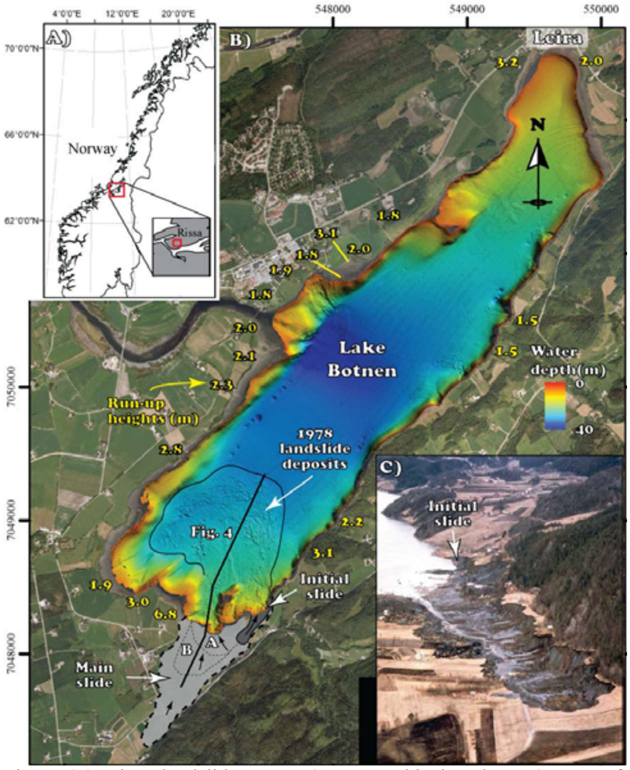

Figure 26. Rissa landslide area: A) Geographic location; B) Map of Lake Botnen with colour-coded bathymetry, outline of slide deposits and outline of the areas affected by the initial slide (dark grey), the two major flakes A and B and the subsequent retrogressive slide (light grey); C) Aerial view of the slide pit. (courtesy Aftenposten) (after L'Heureux et al., 2012b).

The observed run-out parameters of the Rissa landslide that were compared to the results of the simulations were as follows: the run-out distance under Stage 2, which was 1150 from the lakeshore; the maximum velocity over the flow domain, which was was 11 m/s, and the average deposit height, which was 6-8 m.

The probabilistic run-out analyses were done with a model developed at University of Oslo and NGI. The numerical model used is an extension of the Bing model (Imran et al. 2001) in Eulerian coordinates with two space dimensions. The Herschel-Bulkley rheology (Imran et al., 2001) was fully implemented to enable the dynamic computation of the depths of the plug and shear layer. The model is described in Lacasse et al. (2017b) and Lacasse (2017). The model was combined with Monte Carlo simulations for the probabilistic calculations.

The landslide movement was controlled by the combination of the parameters $\tau_{y,0}$, $\tau_{y,\infty}$ and Γ, as well as the initial bathymetric thickness and the bathymetry. Table 6 lists the statistics for the three random variables in the numerical analyses. The initial yield stress (undrained shear strength) of the Rissa clay near the sliding surface) was varied between 10 and 20 kPa (NGI, 2012) and had a CoV of 17%. The residual yield stress was varied in a range typical for quick clays, between 0.1 and 0.5 kPa (with a CoV of also 17%). A lognormal distribution was assumed for the three random parameters.

Table 6. Input parameters for numerical analyses

Random variable	Mean	Standard deviation	Distribution
Initial yield stress. $\tau_{y,0}$ (kPa)	15	2.5	Lognormal
Residual yield stress. $\tau_{y,\infty}$ (kPa)	0.3	0.05	Lognormal
Gamma, Γ	0.05	0.025	Lognormal

Stage 2 of the Rissa landslide was simulated, and the run-out distance, maximum velocity and deposit height were calculated with 1000 Monte Carlo simulations. Figures 27 to 29 show the histograms of run-out distance from the lakeshore, maximum velocity over the flow domain and average deposit height. The figures give the mean, minimum and maximum values, standard deviation (SD), coefficient of variation (COV) and number of simulations (N). The calculated mean values of run-out distance, maximum velocity and deposit height were 1346 m, 10 m/s and 3.1 m, respectively. The mean values[1] are to be compared to the observed run-out distance under Stage 2 (1150 m from the lakeshore, L'Heureux et al., 2012a), maximum velocity over the flow domain (11 m/s) and average deposit height (6-8 m).

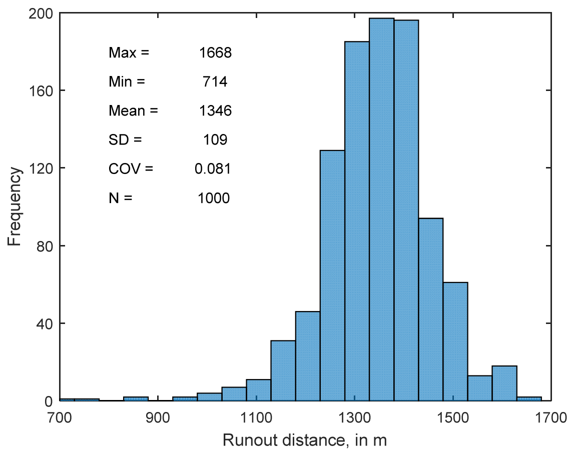

Figure 27. Statistics of run-out distance of Rissa landslide, 1000 Monte Carlo simulations.

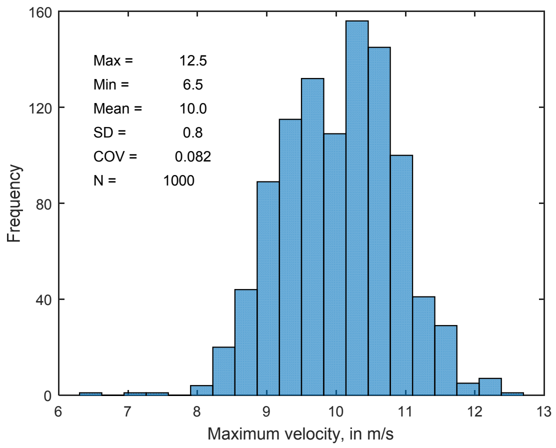

Figure 28. Statistics of maximum velocity of Rissa landslide, 1000 Monte Carlo simulations.

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[1] Since this is a Monte Carlo analysis with limited number of simulations, the most reliable results are the mean ± one standard deviation about the mean.

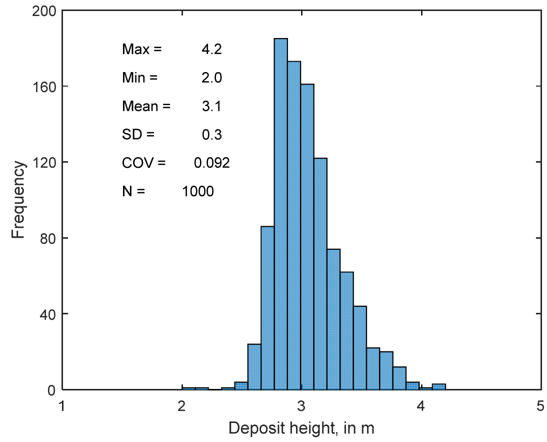

Figure 29. Statistics of maximum velocity of Rissa landslide, 1000 Monte Carlo simulations.

The mean calculated run-out distance overestimated the observed run-out distance by about one standard deviation. The calculated maximum velocity was close to the observed value and the deposit height was underestimated.

Figures 30 to 32 show that run-out distance decreases with increasing residual yield stress, maximum velocity decreases with increasing initial shear strength and parameter Γ, whereas deposit height increases with increasing residual yield stress.

The visco-plastic model used leads to longer run-out and thinner landslide deposits. There is a reverse correlation between predicted run-out distance and deposit height (Figs 30 and 32), so it is logical that the longer run-out distance resulted in a smaller deposit height.

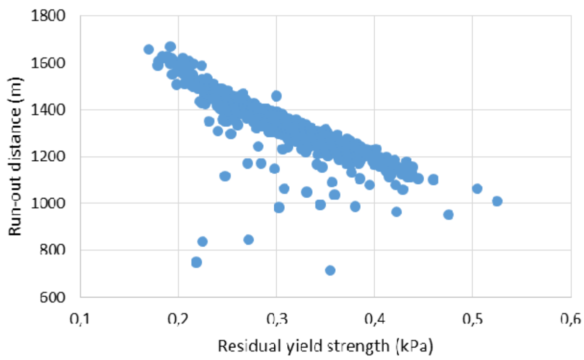

Figure 30. Run-out distance vs residual yield strength, Rissa.

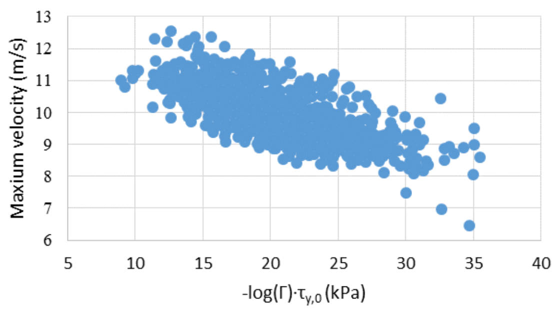

Figure 31. Maximum velocity vs parameter Γ, Rissa.

The overestimation of run-out distance comes from the fact that the model assumes that the entire material is quick clay. In reality, there is non-sensitive or less sensitive clay on top of the quick clay. Since the model is depth-averaged, the remoulded yield stress should probably have been selected with a higher value to account for the less sensitive material. This would then give a shorter run-out distance and higher deposit height. New simulations are needed to further document this effect. The results in L'Heureux et al. (2012a) with the Bing model also show a low deposit thickness for the same reason.

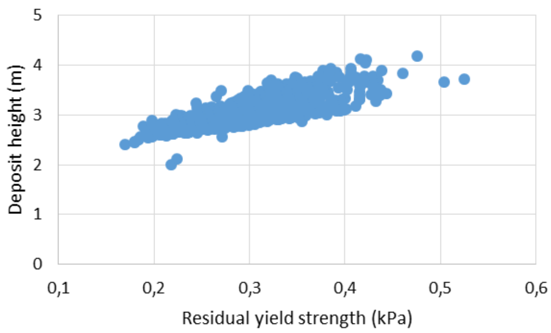

Figure 32. Deposit height vs residual yield strength, Rissa.

7 CHALLENGES AND EMERGING ISSUES

One of the basic challenges in the use of probability theory in geotechnics is that the classical frequentist statistics and sampling theory, which are the bases for derivation of probability distribution functions in most disciplines, are not of much use in geotechnical engineering. As Baecher (2017) pointed out in the Second Suzanne Lacasse Lecture, "Geotechnical engineers deal with uncertainties associated with limited knowledge. They have to assess the probabilities of unique situations. These uncertainties are not amenable to Frequentist thinking; they require Bayesian thinking. Bayesian thinking is that of judgment and belief. It leads to remarkably strong inferences from even sparse data."

An emerging issue is the realization of the importance of multiple risks and cascading events among the various actors in the disaster risk reduction community. A universally-accepted framework for joint analysis and quantification of all the human-induced and natural risks that can affect an engineering design does not exist. Risk assessment is still fragmented, where geo-risks are considered independently of each other and separately from industrial or human-induced risks. Cascading hazards could lead to disastrous consequences because the affected communities are often not prepared for them. For example, Zhang et al. (2014) and Zhang et al. (2016) did a systematic study of the cascading hazards that were triggered by the Wenchuan earthquake in 2008 and affected the town of Beichuan in northern Sichuan in the subsequent five years. Zhang et al. (2014) provided an overview of the disaster chain in the Beichuan town triggered by the Wenchuan earthquake and described seven episodes of hazards in the town and their severe consequences. The hazards included a strong earthquake, multiple large landslides, dam-breaching floods, large-scale debris flows, severe sedimentation, change of river course, and flooding/scouring. Zhang et al. (2014) discussed the interactions among these hazards and suggested a protocol for identifying the geohazards that could be triggered by a strong earthquake.

A multi-risk approach is required for sound environment management and land use planning, as well as for a competent emergency preparedness and response in regions that are exposed to multiple hazards. Multi-risk evaluation is a new field across a number of expertise areas, and with incomplete theory. The issues that need to be addressed fall into three classes: (a) Interaction and amplification of risks, including cascading effects (i.e. the recognition that multi-risk is more than a simple aggregation of single risks); (b) Dynamic vulnerability to multi-risks (i.e. how does the vulnerability change with time when one or more events hit., and how does the vulnerability change when different threats occur almost simultaneously); and (c) Application of multi-risk assessment (i.e. implementation in practice to mitigate risk more effectively).

Another emerging issue is that in many contexts, experts acting alone cannot choose the risk management strategy. A multi-disciplinary approach to risk management is heavily underlined by all UN organizations working in this field. A deciding factor on whether an extreme event turns into a disaster is the social vulnerability of the population at risk, i.e. the capacity to prepare for, respond to and recover from the extreme event. Policy-makers and affected parties are recognizing that traditional expert-based decision-making processes are insufficient in controversial risk contexts. The approach tends to de-emphasise the affected interests in favour of "objective" analyses, and the decisions lack popular acceptance. The approaches can slight the local and anecdotal knowledge of those most familiar with the situation and can, in the worst case, produce irrelevant or unworkable outcomes. Conflicting values, interests and expert evidence characterise many risk decision processes. The decisions become more complex with long time horizons and uncertainties on climate and demographic and other global changes. Risk communication and stakeholder involvement have been widely acknowledged for supporting decisions on controversial environmental risk events. The value of a participatory approach is multiple. The decision-makers are those who need to manage the threats. The stakeholders are the most knowledgeable on the decision-making process, and on the concerns (or lack thereof) for hazards. When stakeholders are contributors to the development of participatory documents, the value of the document increases because the stakeholders have ownership of the written work, making it more likely that guidelines become more effective.

Many factors complicate the risk picture. Urbanisation and changes in demography are increasing the exposure of vulnerable population. Climate change is altering the geographic distribution, frequency and intensity of hydro-meteorological hazards and threatens to undermine the resilience of poorer countries and their citizens to absorb loss and recover from disaster impacts. The earthquake-tsunami-nuclear contamination chain of events in Japan is a telling example of cascading hazards and multi-risk: the best solution for earthquake-resistant design (low/soft buildings) may be a less preferable solution for tsunamis (high/rigid buildings). The sea walls at Fukushima gave a false sense of security. The population would have been better prepared if told to run to evacuation routes as soon as the shaking started. The 2008 China earthquake is a second recent example of cascading hazards where an earthquake caused large rock falls, landslides and landslide dams ("quake dams") that could lead to disastrous dam breach. Storage facilities for hazardous materials, e.g. underground spaces and tailings dams for nuclear waste, contaminated materials, industrial waste and wastewater) are often in the chain of cascading events. To reduce the hazard and risk, improved design, improved models, reduced uncertainty and improved risk management are required, in addition to improved risk assessment and management.

It is therefore increasingly evident that the conventional approach for risk assessment and management may be inadequate when dealing with rare, extreme events. The next section of this paper (Section 8) presents the idea of stress testing as a complement to traditional risk assessment for managing the risk posed by extreme events.

8 STRESS TESTING APPROACH

Conventional strategies for managing the risk posed by natural and/or man-made hazards rely increasingly on quantitative risk assessment (QRA). The conventional approach for risk assessment and management may be inadequate when dealing with rare, extreme events. Nadim (2016) proposed stress testing as a complement to traditional risk assessment for managing the risk posed by extreme events. Stress testing is a procedure to determine the stability of a system or entity. It involves testing the said system or entity to beyond its normal operational capacity, often to a breaking point, in order to observe its performance/reaction to a pre-defined internal or external pressure or force. Stress testing is a rapidly evolving field in engineering risk management. The new research initiatives in Europe and elsewhere are likely to provide the tools and methodologies needed for stress testing of critical infrastructure in the near future (Nadim, 2016).

Conventional strategies for managing the risk posed by natural and/or man-made hazards rely increasingly on quantitative risk assessment (QRA). The QRA methods estimate the various components of risk, namely hazard, vulnerability and exposure of elements of risk, and utility or value of the elements at risk. The methods do this quantitatively and provide a value estimate of the expected loss. Within the QRA framework, engineers and natural hazard specialists try to understand the geophysical processes that would lead to ‘extreme’ natural events. They focus on the uncertainties and the unpredictable nature of rare events and attempt to estimate the extremely small annual occurrence probability (hazard) associated with these events.

However, the conventional approach for risk assessment and management may be inadequate when one needs to deal with rare, extreme events. One of the challenges in the management of risk associated with extreme events is that the mechanism triggering an extreme event may be different from those triggering the more frequent events. Furthermore, for hydro-meteorological hazards, a central concern is that climate change has introduced substantial non-stationarity into risk management decisions. Non-stationarity is the realization that past experiences may no longer be a reliable predictor of the future character and frequency of events; it applies both to hazards and to the response of human systems to same. As climate change is expected to change the frequency, magnitude, and other characteristics of extreme events, some of which will be associated with extreme impacts, risk management strategies must accommodate a shifting distribution of the latter.

As discussed in the example on the calibration of partial safety factors for offshore foundation design, the engineering design of modern critical infrastructure is often done deterministically using partial safety factors on load and resistance parameters. The partial safety factors defined in the codes and standards are calibrated to achieve a target safety level through a probabilistic, reliability-based approach. When the engineering structure is exposed to loads induced by natural hazard events, the load induced by a "design event" with a long return period is defined and the "characteristic design load". This is illustrated in Figure 33.

This conventional design approach implicitly accepts that there is a "residual" risk, which could be "neglected" because the probability of that risk being realized is extremely small. This residual or neglected risk could be due to "extreme events", which have a longer return period than the return period for the design load, or they could be due to the uncertainty in the prediction models and lack of knowledge of the mechanisms at work. The latter have a shorter return period than that specified in the design basis, but have a greater intensity than the design event because of the imperfections in our prediction and calculation models. Both types of events pose a risk to the integrity and performance of the system, a risk which is implicitly accepted and knowingly neglected in conventional engineering design. Nevertheless, these events can occur, and when they do, they are referred to as extreme events. Therefore, *the conventional engineering design is not suitable for dealing with the risks posed by extreme events.*

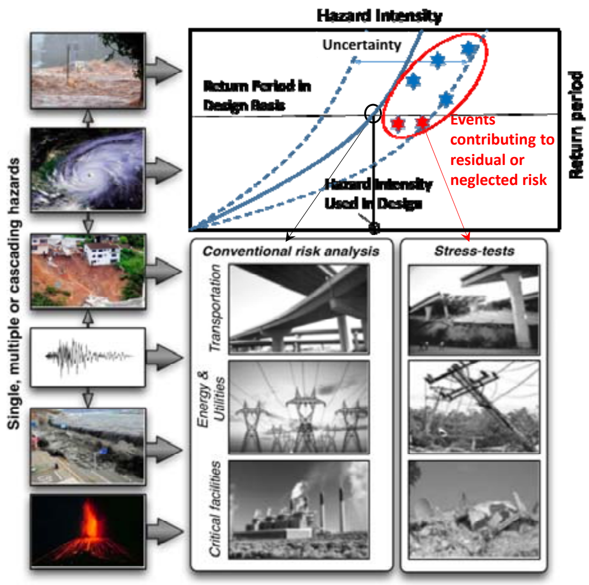

Figure 33. Residual or neglected risks in conventional reliability-based design approach (Nadim, 2016).

8.1 *Stress testing for extreme events*

Stress testing is a procedure to determine how a given institution, system, etc., would perform under greater than usual stresses or pressures. It involves a simulation test of the said system or entity to beyond its normal operational capacity, often to a breaking point, in order to observe its performance/reaction to a pre-defined internal or external pressure or force. Stress tests have been used for many years in air traffic safety, in particular for airplanes and helicopters. In recent years, stress testing has often been associated with methodologies to assess the vulnerability of a financial system or specific components of it, such as banks. A number of analytical tools have been developed in this area and have been frequently used since the late 1990’s (e.g., Borio et al., 2012). However, stress testing has also been criticized in the field of forecasting due to the use of historical data taken from periods of time that are considered to be insufficient for reliable predictions.

More recently, stress testing has been applied to the comprehensive safety and risk assessment of Nuclear Power Plants, in particular in the aftermath of the 2011 Fukushima Dai-ichi accident. Present knowledge has already led nuclear regulators, experts and operators throughout the world to important conclusions regarding the improvement of nuclear power plant safety. In particular, the accident highlighted three areas of potential weakness in present safety approaches:

- Inadequacy of safety margins in the case of extreme external events (especially natural hazards).
- Lack of robustness with respect to events that exceed the design basis.
- Ineffectiveness of current emergency management under highly unfavourable conditions.

These issues were the focus of the stress tests imposed on all nuclear power plants in Europe in 2011 and 2012 (WENRA, 2011). Whereas nuclear facilities represent one important asset subjected to potential damage from natural hazards, there are other critical infrastructures (CI) that may be subjected to a similar risk of catastrophic impacts that will require future attention. According to the European Union Council Directive 2008/114/EC, CI refers to an asset, system or part thereof located in Member States that is essential for the maintenance of vital societal functions, the health, safety, security, and economic and social well-being of people. The disruption or destruction of the CI would have a significant impact in a Member State as a result of the failure to maintain is vital societal functions. The CI sectors include: (I) Energy (electricity, oil, gas) and (II) Transport (road, rail, air, inland waterways, ocean and short-sea ship-ping and ports). Infrastructures must meet the following criteria to be judged as being critical: (1) the definition above, (2) within sectors as identified above and (3) have a potential impact according to the following criteria: (a) casualty criterion, (b) economic effect criterion and (c) public effect criterion (Bouchon et al., 2008).

The premise behind promoting stress testing for CIs is that a CI is designed to withstand the impact of natural hazards according to a set of regulations specified by codes and standards or by an owner/stakeholder. These regulations are often set through probabilistic evaluations with the objective of reducing risk to an acceptable. This evaluated risk will be in accordance with what society will tolerate in terms of loss of life, environmental damages and the loss of assets through the definition of acceptance criteria that are incorporated into regulations. As mentioned earlier, the design rules that result from such regulations implicitly accept that there is a residual risk associated with rare, extreme events that is neglected because of the (objectively calculated or perceived) very low probability of occurrence. However, the Fukushima accident showed that, as a consequence of the neglect of extreme events, a system that is very robust as long as events remain within its design basis can abruptly shift to complete failure when that threshold is passed. Stress tests can help detect such “cliff-edge effects” and introduce some robustness in the system without any change in the acceptable level of risk.

Most risk evaluations are based on probability estimates using historical data and consequence models that try to estimate the impact of unwanted future hazard situations. For natural hazards, historical data may in some cases be sparse or highly uncertain. Similarly, simplified models of highly complex situations may yield forecasts containing significant uncertainty. Both situations may therefore neglect risks that should be introduced into the evaluations.

A stress test is an examination of the safety of a system under those particularly unfavourable scenarios that fall outside the design basis specified by the regulatory regime, by the operational institution or by the stakeholders. The stress test can test the system to assess its response to scenarios expected to be in the residual and neglected risk areas (Fig. 33). In this respect, stress testing is not a substitute for conventional risk or safety assessments, but it provides additional valuable insight for extreme situations. What stress tests and conventional risk or safety assessments have in common is that they both rely on a description of the system of interest, which helps to associate the state of the system and a set of consequences under any given or potential scenario.

The characteristics of the critical systems in the two main industries where stress tests have been applied in recent years are quite different. Financial systems are open and extremely dynamic, with risks coming from the many inter-action points that make up a large network consisting of numerous inter-financial institutional linkages. Consequently, the modelling and assumptions are quite fragile and uncertainties are difficult to quantify. These systems therefore require a stochastic evaluation of any impacts. On the other hand, nuclear power plants are closed systems with simple criteria for what needs to be avoided. Identifying the causality of incidents is relatively straightforward in closed systems.

Most non-nuclear critical infrastructures fall somewhere in between the systems of these two main industries and may be characterized as semi-open systems. Hence, while on the one hand, causality is strong with respect to the physical impacts of natural hazards and their geographical distribution, multiple failures and potential cascading processes can sometimes occur in stochastic patterns.

The stress test methodologies developed for evaluation of the performance of CI subjected to extreme events should be able to handle both causal and stochastic impacts.

8.2 *A generic framework for stress testing*

The aim of stress testing for extreme events should be the identification of critical parameters and methodologies for hazard and risk assessment for low probability, high consequence events. From a decision-making perspective, the two key factors of stress testing are the definition of the acceptable states of the system and the severity of the scenarios under which the system is required to remain within the system boundaries.

High standards in either of the two key factors are likely to lead to important changes to the system’s design basis. Highest standards are likely to be very costly to implement. Lower standards, on the other hand, are unlikely to lead to a significant enhancement of the system’s robustness or a satisfactory reduction of risk. There is a need to strike a balance between the two extremes. French nuclear regulators, for instance, ask operators to identify a set of core components and subsystems that are necessary to safeguard the critical functions of power plants. New safety measures will have to ensure that the integrity of the "hard core" is maintained under significantly more severe conditions than before, while damage to the rest of the system will be deemed tolerable.

In the financial sector, stress testing defines a scenario and uses a specific algorithm to model expected impact on a portfolio's return should the scenario occur. There are three types of scenarios:

- Extreme event: hypothesize the portfolio's return given a catastrophic event, often the recurrence of a historical event. Current positions and risk exposures are combined with the historical factor returns.
- Risk factor shock: shock any factor in the risk model by a user-specified amount. The factor exposures remain unchanged, while the covariance matrix is used to adjust the factor returns based on their correlation with the shocked factor.
- External factor shock: instead of a risk factor, shock any index, macro-economic series (e.g., oil prices, property prices), or custom series (e.g., exchange rates). Using regression analysis, new factor returns are estimated as a result of the shock.

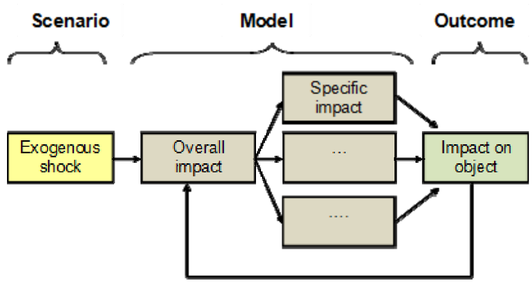

Figure 34. Generic model for macro-economic stress tests (adapted from Borio et al. (2012).

A generic model for stress tests based on available methods for macro-economic stress tests is given in Figure 34. In this approach, scenarios with exogenous shocks are the subject (input) to models that predict the outcome for specific objects at risk. The object-specific impact is then fed back into the model to investigate the sensitivity of feedback processes.

A stress test is more system-oriented than conventional hazard and risk assessments, focusing on impacts on specific objects from a generic (top-down view) perspective. Traditional risk analysis tends to focus on specific objects (bottom-up perspective) rather than assessing cascading impacts. With this new perspective, the stress test methodology should comprise the components explained below.

Scenario analysis: One of the key aspects in stress testing is the ability to test the critical infrastructure for low probability and high consequence shocks and to make the test robust enough to yield reliable results. A stress test is not better than the collective competence of the persons involved in its design. A well-designed stress test should include robust methods for scenario construction, hazard identification, stakeholder involvement, and treatment of uncertainty and expert judgment to embrace the full complexity of the problem.

Model: The models to be used in stress tests should advance from risk analysis to object-oriented methodologies. The stress tests should be based on scenarios that physically quantify the loading from natural hazards, identify the system weaknesses and system interactions by considering spatial, scalar and thematic factors. This approach may require more complex models than the usual event tree cause-and-effect model used in more traditional risk analysis.

Outcome: The final outcome of the stress test should inform the critical infrastructure designers and stakeholders of the weaknesses of the CI under consideration, and measures to improve the performance of the CI. In other words, the outcome should contribute to improving CI's, and hence society's resilience against extreme natural hazard events.

The stress tests recommended by the West European Nuclear Regulators Association (WENRA, 2011) for the nuclear industry are mainly designed to complement existing safety analyses, whether deterministic or probabilistic. Their starting point is to identify the weaknesses of existing analyses: uncertainties in parameters, limitations in modelling approaches, assumptions which may not provide the optimum options for the most critical variables, etc. Conservative assumptions are then adopted in order to cover the most important weaknesses.

The approach recommended by WENRA basically asks for documentation outlining the satisfactory response of a plant affected by a Design Base Earthquake (DBE) and Design Base Flood (DBF), and consideration of whether an event more severe than the DBE or DBF is physically possible. This approach is relatively standard. If an event is found to be more severe than the DBE or DBF, any "cliff edge" effect in the response of a building or equipment leading to failure should be identified. The approach is almost fully deterministic with little regard to uncertainties. The WENRA (2011) methodology for stress tests could be made more robust by addressing these shortcomings.

In summary, borrowing from the terminology of stress testing for traffic, financial sector and nuclear plants, a well-designed stress test should include the following elements:

- Consider extreme event scenarios.
- Identify the "cliff edge" possibilities in the system in question that could produce risk shocks.
- Identify the conditions that could produce an external factor shock, e.g., human and organisational factors, or climate and/or demographic changes, etc.
- Identify the tipping points of the system, i.e. the threshold which differentiates the potential situation from the condition where the system status will change irreversibly and cannot be saved anymore, but where the consequences still need to be minimized. For stress testing of CIs, this may require structural (failure) analyses for some systems.
- Provide measures to improve the robust/resilience of the system; May conduct further stress testing after taking measures.

Zhang et al. (2017) recognized the usefulness of stress testing to manage the risks of low-frequency high-consequence hazards. They suggested a framework for coping with the landslide risks under extreme rainstorms. They saw stress testing as a targeted reassessment of safety margins of a system under extreme events. Their framework, developed for the slope safety system in Hong Kong, consists of five steps (Fig. 35):

1. Identifying future critical rainstorm scenarios under the changing climate;
2. Evaluating the response of the slope safety system to the critical rainstorm scenarios;
3. Assessing the risks posed by the multi-hazard processes;
4. Evaluating the bottlenecks of the slope safety system; and
5. Proposing strategies for improving system performance.

A preliminary stress test was performed to evaluate the response of Hong Kong Island to four storms corresponding to 29%, 44%, 65% and 85% of the 24-hour probable maximum precipitation. The testing showed where the extreme storms can pose high risks, and therefore indicating where new safety and preventive policies and strategies for multi-hazard risk management are required.

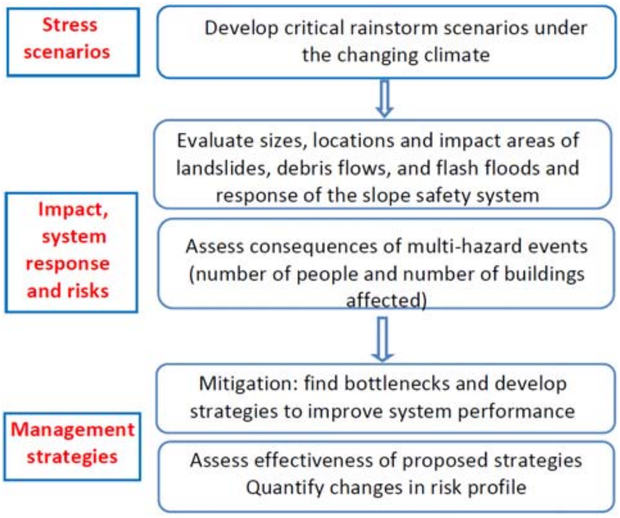

Figure 35. Proposed stress-testing framework for evaluating the Hong Kong slope safety system (Zhang et al., 2017).

8.3 *Stress testing of rockfill dam in western Norway*

Stress testing of critical infrastructure for extreme natural hazards events is a relatively new area of research in engineering risk management, and not many examples are found in the literature. However, the process of risk assessment for dams and tunnels in a workshop format where event trees for possible failure scenarios are constructed comes very close to ideas being forwarded in stress testing, namely identifying the weak points in a complex system and making the system more robust through simple remediation measures. The example provided in this section is based on one such study in Norway. The dam is the same Dravladalen Dam presented in the example in Section 6.2.

Although Dravladalen Dam was the smallest of the three dams analysed, the annual failure probability estimated for this dam was anomalously high. This was mainly because of an uncertain potential for compaction of deep, drifted snow and accumulation of ice within the spillway approach channel that could restrict spillway discharge even at small flood inflows in winter and spring. The risk analysis workshop for Dravladalen Dam in 1996 highlighted this as a weakness of the dam system requiring additional study and important mitigation measures.

In stress testing terminology, this system weakness was identified when the scenario of an extreme flood in wintertime was considered. The dam owner (Statkraft) decided to build a concrete shelter for the approach channel to the spillway tunnel in order to address this problem. This shelter was built as part of the remediation measures undertaken by the dam owner during the past fifteen years (Section 6.2).

As the photographs in Figure 36 illustrate, the concrete shelter has effectively eliminated the mode of failure that was most critical for the dam. A new risk assessment that was done for the dam in January 2016 showed that the annual failure probability for this failure mode was reduced by two orders of magnitude because of the implementation of this simple mitigation measure (Lacasse et al., 2017a).

Figure 36. Approach to spillway tunnel of Dravladalen Dam in March 2000 (above) and in March 2015 (winter with heavy snow downfall (below) (Photos: Statkraft AS).

In a conventional risk assessment, the dam owner could have argued that the probability of occurrence of an extreme flood event in winter is so low that there is no need for implementation of risk reducing measures.

In a stress testing framework, on the other hand, the focus is on identifying the weak points in the system and improving the performance under extreme events, without focusing too much on the probability of occurrence of those events. With this goal in mind, the critical infrastructure owner(s) are more willing to invest in risk reducing measures to make their CI more robust against extreme events or unexpected sequence of events that may lead to failure.

9 CONCLUDING REMARKS

Even in simple and routine geotechnical design problems, a geotechnical engineer must deal with the uncertainties in mechanical soil parameters, external loads and load effects, and most importantly in many cases calculation models. These uncertainties are often addressed indirectly through conservative choice of safety factors and wording in design guidelines.

The example case studies presented in this paper demonstrate that reliability theory and reliability-based design provide the framework to account for the uncertainties in a systematic manner and make the design more robust while avoiding excessive conservatism. There is added value in combining deterministic with probabilistic analyses, and one simply gets more insight in the problems that may occur and their consequences.

Uncertainty is closely related to risk. For a geotechnical engineer, the task of risk analysis involves assessing hazard, vulnerability and risk in the context of safety and/or potential economic loss. This is a very complex task and subject to major uncertainties when engineering judgment has to be translated into numbers.

In geotechnical analyses, we "run the risk" of using the numbers in a complex analysis, for example an event tree analysis, to subconsciously arrive at the answers that we want to achieve. Unbiased estimates, when data are lacking, are difficult to achieve, even for the most experienced professional.

It is essential that the assessment models for hazard and risk become more transparent and easier to perceive and use, without reducing the complexity of the models required for the assessment. There is today an increasing focus on hazard- and risk-informed decision-making. Therefore integrating deterministic and probabilistic analyses in a complementary manner, has been recommended since the late 80's. Such integration will enable the end-user (with or without scientific background) to concentrate on the analysis results rather than the more complex underlying information.

Focus needs to remain on "safety". The complementary deterministic-probabilistic approach brings together the best of our profession and the required engineering judgment from the geo-practitioners and the risk analysis proponents.

New challenges such as climate changes impact, rapid urbanisation, multiple risks and cascading events, and managing the risk posed by extreme events are pushing the limits of what conventional reliability-based design and quantitative risk assessment could handle. New, innovative calculation methods and ideas for dealing with these problems are evolving. The geotechnical engineering profession must consider adopting the new approaches to be able to deal efficiently and convincingly with the modern challenges faced by the society.

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