Risk and Reliability in Geotechnical Engineering

Suzanne Lacasse
Norwegian Geotechnical Institute, Oslo, Norway

Farrokh Nadim
Norwegian Geotechnical Institute, Oslo, Norway

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Suzanne Lacasse and Farrokh Nadim
Norwegian Geotechnical Institute
Oslo, Norway

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ABSTRACT

Statistics, reliability analyses and risk estimates can be very useful decision-making tools in geotechnical problems. Yet the methods are little used in practice. The offshore and mining industry are at the forefront for the use of these approaches, having encouraged their use and sponsored research that has enabled the methods to be well-documented and of proven usefulness in the study of alternatives for design and decision-making in face of uncertainties. The paper presents a few case studies in different areas of geotechnical engineering and discusses the results that would have been obtained without the use of the risk approach. Special emphasis is given to dams and offshore structures, both piled and shallow foundations. The authors take a look at the reasons why the methods are not used to a greater extent in practice and make recommendations as to when and how one should uses such methods.

KEYWORDS

Risk, reliability, uncertainty, foundations, failure, dams, offshore foundations, soil parameters, probabilistic analysis

INTRODUCTION

This invited paper presents the role of reliability- and risk-based approaches in solving geotechnical design problems. It discusses existing geotechnical applications, available reliability tools and the benefits of risk and reliability estimates when used in conjunction with deterministic analyses. Case studies with dam design, piled and shallow foundations, earthquake analysis, slope stability, rock mechanics and mining engineering, are presented as examples of the application of such line of thought.

Reliability analyses are needed because geotechnics is not an exact science. Predictions of foundation behaviour cannot be made with certainty due to spatial variation of soil and load properties, limited site exploration, limited calculation models, and uncertainties in the soil parameters. Reliability-based analyses enable one to map and evaluate the uncertainties that enter in the formulation of a geotechnical problem. If a deterministic model for the analysis of a geotechnical problem exists, a probabilistic analysis model can always be easily established with the tools available today. That one finds difficult the quantifying of the uncertainties is not a good reason to avoid defining the uncertainties or establishing their significance in design.

It is increasingly important today to adopt rational, consistent, and "documentable" design approaches that inform of and account for the uncertainties in the analysis parameters. Only reliability analyses can provide the designer with insight in the inherent risk level of a design.

The paper aims at establishing that reliability-based approaches are a necessary and useful complement to the conventional (deterministic) analyses: they are not a replacement, but an addition to conventional analyses that provides important information on the effects of uncertainty on the response.

RISK, RELIABILITY AND STATISTICS

It is fair to say that other areas of civil engineering, such as structural and hydrodynamics analysis, lie ahead of geotechnical practice in the area of reliability. Geotechnical engineers are learning and gaining benefits from the experience of these related fields: mathematical solutions to complex approximation and iteration problems already exist, the significance of different reliability aspects have already been established, advances of research in reliability engineering and the advent of powerful personal computers bring the exploitation of the available tools within everyone's reach, the language barrier between probabilists and geotechnical engineers is decreasing. The modern engineer will also experience increasing demands for multidisciplinary expertise.

Risk analysis is about prediction events that have not yet happened. Usually the analysis is broken down into its constituent parts. No matter what type of analysis technique is adopted, the actual components to be analysed will be the same as for a deterministic (conventional) analysis. Such analyses however do not eliminate the risk of an «oddball» (Dr. R.B. Peck) or of human error

The term risk implies a combination (the product) of the probability of an event occurring and the consequences of the event should it occur. Probability of failure is a measure of risk only if all failure modes result in the same consequences. This is not generally valid as different occurrences may lead to different failures and different time sequences, and therefore different consequences.

The purpose of a risk analysis is to provide a tool to support the decision-making process. Risk-based analysis pulls together a set of relevant scenarios with the corresponding probabilities of occurrence and consequences. The associated probabilities are in fact only a quantification of one's uncertainty.

Risk or reliability analyses have been developed, for example, for a panoply of geotechnical problems, for example:

- Bearing capacity (single and several failure modes)
- Settlement (total settlement and settlement versus time)
- Spring stiffness for soil-structure interaction analysis
- Safety of dams and slope stability
- Axial pile capacity
- Skirt penetration resistance for offshore structures
- Soil resistance to pile driving
- Site response under earthquake loading

Among the first case studies where probabilistic concepts were used in geotechnical engineering, noteworthy contributions include those presented by Folayan et al. (1970), who updated settlement predictions on the basis of observations, and Høeg and Murarka (1974), who calculated the probability of failure of a retaining wall. Just the same the concepts have been less widely used in geotechnical engineering than expected, given the variability of soil and rock.

A committee on «reliability methods for risk mitigation in geotechnical engineering» examined the reasons why risk and reliability methods are not more widely used and whether there was potential for wider use (report to the National Research Council (NRC, 1995). Some of the conclusions reached by the committee are reiterated herein.

Figure 1 illustrates schematically how the reliability analysis of a geotechnical problem is done. Compared to a deterministic analysis, the input parameters are defined over a range of probable values rather than punctual values.

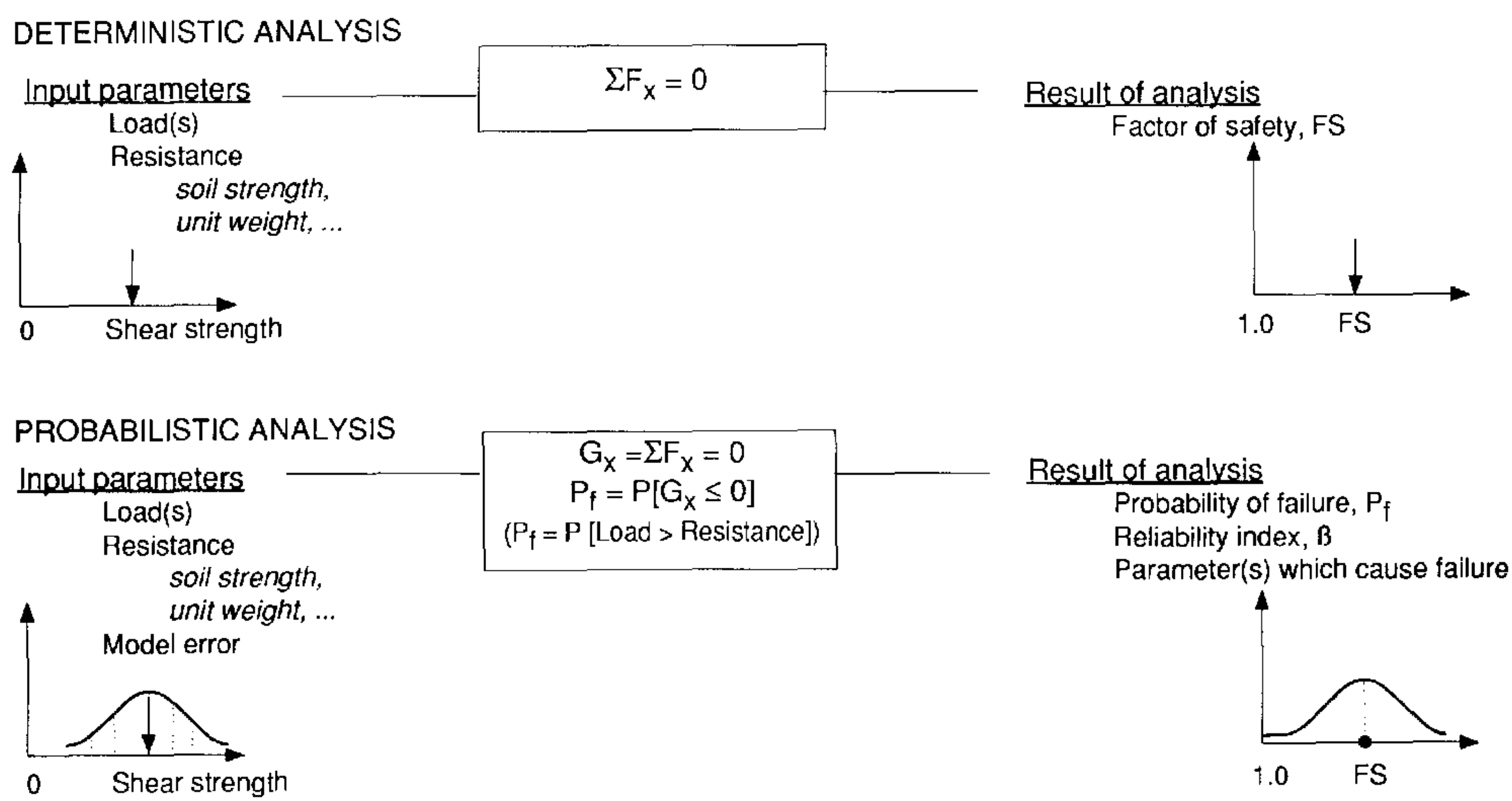


Fig. 1 Comparison of deterministic and probabilistic analyses

The equilibrium function describing for example failure is defined by a "limit state function" which has the same form as the deterministic equation. The probability of failure, i.e. where the load can exceed the resistance, is calculated. Instead of a point estimate of factor of safety, the distribution of the resistance is compared with the distribution of the load. The probability of failure is the probability that the distribution describing the load and the distribution describing the resistance intersect.

Analysis tools

As long as there exists a deterministic model to analyse a geotechnical problem, a probabilistic analysis can always be established using the tools described in the next section. These tools are ready-made software that are easily linked with the software describing the deterministic geotechnical solution. Probabilistic analyses provide the following results:

- Probability of failure (probability of non-performance)
- Reliability index, or where is the most probable response relative to failure
- Sensitivity of result to any change in parameters

One probabilistic analysis will give the same insight as a large number of parametric analyses with all of the uncertain parameters that are part of the formulated solution. As input, the user must supply (1) the equation defining failure and (2) the mean and distribution function (often normal or lognormal) for each parameter in the analysis. Except for distribution function, the required input is the same as for deterministic analyses.

The following methods can be used to quantify the effect of uncertainties in a geotechnical response:

FORM: First-Order Reliability Method. Probably the best practical method today, it approximates the limit state function by a first-order function. The method works well over a wide range of probabilities and is simple to implement when one has an explicit limit state formulation. FORM accounts for the probability distribution of all uncertain variables.

SORM: Second-Order Reliability Method As FORM, but the limit state function is approximated by a second-order function. The results of the SORM analyses have for all geotechnical problems modelled so far given probabilities of failure very close to the values obtained with FORM.

SORM+: SORM with sampling around solution point. Improved SORM, with a search around the solution for an even more critical point. The results of the SORM+ analyses have also been found to be close to the results obtained with FORM.

FOSM: First-Order Second Moment approximation. The most feasible approach for complex formulations where the performance mechanism cannot be formulated explicitly. The FOSM approximates mean and variance but cannot account for the probability distribution of the uncertain variables. The solution method is used, for example, for the probabilistic analysis of finite element models.

Monte-Carlo simulation (MCS). Repeated simulation of problem solution with randomly selected values of variables. The method applies to all problems but can require a large number of simulations. It can be made more efficient with Latin Hypercube sampling (LHS), which is a Monte-Carlo simulation optimised by "organised" sampling. It reduces considerably the number of simulations required for a reliable distribution of the response.

Bayesian updating. Bayesian updating is a method to relate predicted behaviour with observations, for example updating of factor of safety or settlement prediction on basis of pore pressure and settlement records; updating of pile capacity on basis of pile driving records and/or instrumentation results; updating of bearing capacity on basis of preload test.

Uncertainties in soil parameters

To obtain the statistics of soil parameters, traditional procedures can be used or stochastic interpolation (geostatistics) can be implemented when a lot of data exist. The techniques provide unbiased estimates of mean and variance.

It can be useful to establish data banks for different types of parameters or geographical locations, or to review the literature and compare one's values to values used by others. These estimates can be biased by the beliefs of the designer. The probabilistic analysis will, however, single out the importance of the hypotheses on the results.

Defining probability distribution function may often appear as a problem. However, most geological processes follow a normal or lognormal law. One may choose to use a bounded uniform distribution if one expects all values within a range to be equally probable.

In probabilistic analysis, a "model uncertainty" needs to be defined by a mean or bias and a coefficient of variation (a normal distribution is often assumed). Model uncertainty is difficult to assess. It should be evaluated on the basis of literature, comparisons of relevant model tests with deterministic calculations, «expert» opinions, if available and relevant case studies of "prototypes", if they exist. Model uncertainty is best included in a reliability analysis in one of three ways: (1) as a global factor on the limit state function, (2) as a factor on each parameter of the analysis, or (3) as a factor on components of the analysis, for example on skin friction in each layer and end bearing in the case of axial capacity of a pile.

And yes, considerable reflection and engineering judgement have to be used to establish the values of the model uncertainty, but do not all geotechnical analyses require a dose of good engineering judgement? Including model uncertainty is nevertheless one step forward compared to ignoring the uncertainties that come for example from the calculation model or the way of recovering soil samples.

These were a few basics on the approaches to account for the uncertainties in a geotechnical problem. Lacasse and Nadim (1996b) presents several examples of uncertainties in characterising soil properties, and Lacasse and Lamballerie (1995) give examples of the statistical treatment of cone penetration test results.

Cases studies are now used to illustrate that reliability studies provide useful additional information. Risk and reliability considerations permit the engineer to be «probably right», whereas with deterministic analyses alone, the engineer risks to be «exactly wrong».

CASE STUDIES

Soil investigations

Soil investigations, in the way they are planned, represent a form of risk-based decision.

In general, the complexity level of a soil characterisation is based on the level of risk of a project.

Figure 2 illustrates this with three soil investigations, each depending on the importance of the construction. (The figure is modified from an oral contribution by P.K. Robertson at the 14th ICSMFE in Hamburg in September 1997).

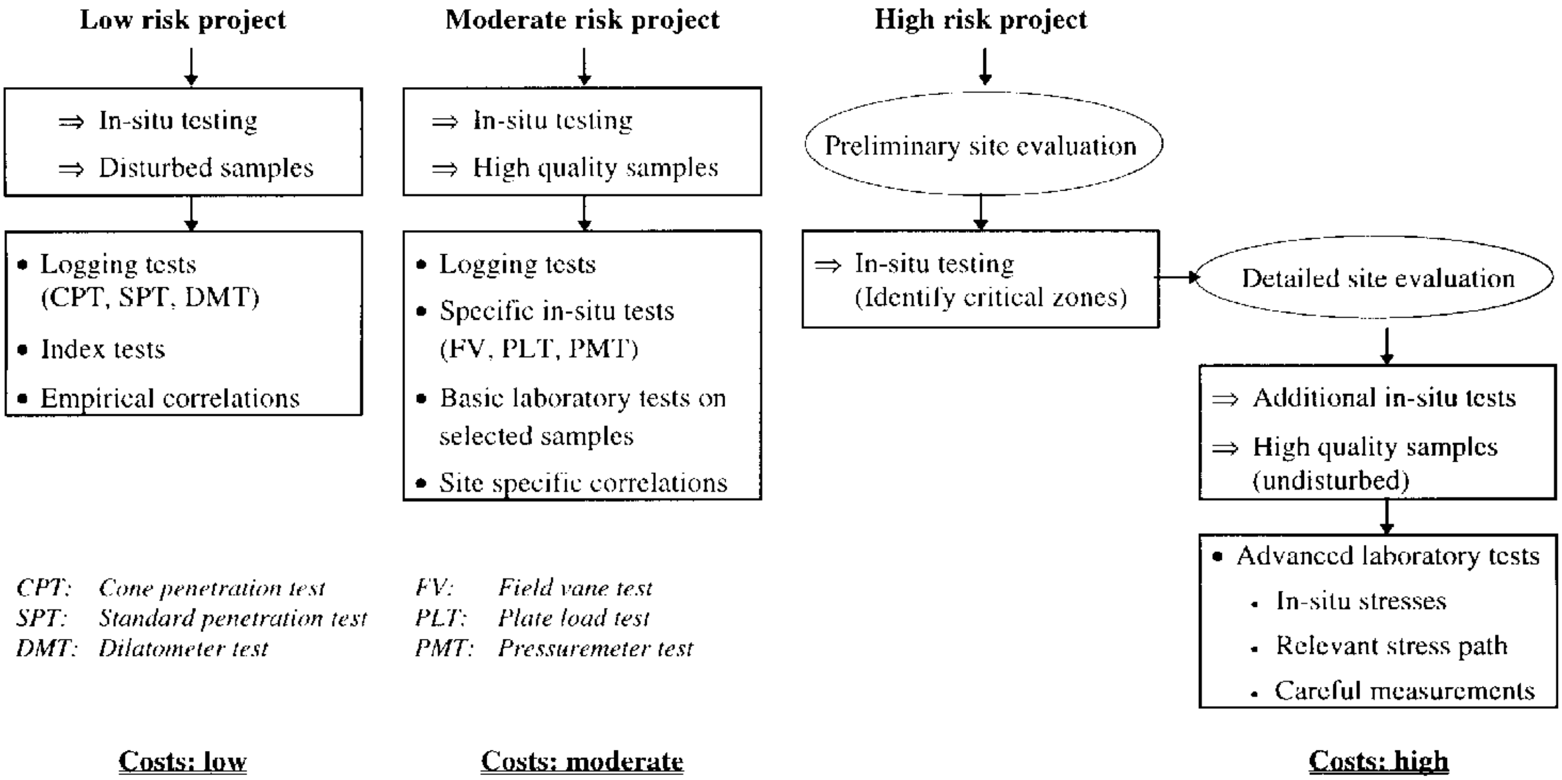


Fig. 2 Risk based soil investigations

For small to medium sized foundations:

- a low risk project involves few hazards and has limited consequences. Relevant experience exists to assist in design. Simple in situ and laboratory testing would be selected.
- in a moderate risk project, there is concern for some hazards and the consequences of non-performance are moderate. Limited experience exists to assist in design. Specific in situ tests and good quality soil samples are required.
- a high risk project involves frequent hazards, and has moderate to high consequences. High quality in situ and laboratory tests are required.

The decision-making process for selecting the appropriate soil investigation methods is risk-based, albeit sub-consciously, as it involves consideration of consequences and costs.

Uncertainty analysis and procedures can help optimise site investigations. The uncertainty in a geotechnical calculation is often related to the possible presence of an "anomaly", for example boulders, soft clay pockets or even drainage layer. Probability approaches can be used to establish the costeffectiveness of additional site investigation to detect such "anomalies". Figure 3 presents an example where the procedure developed by Tang (1987) was used.

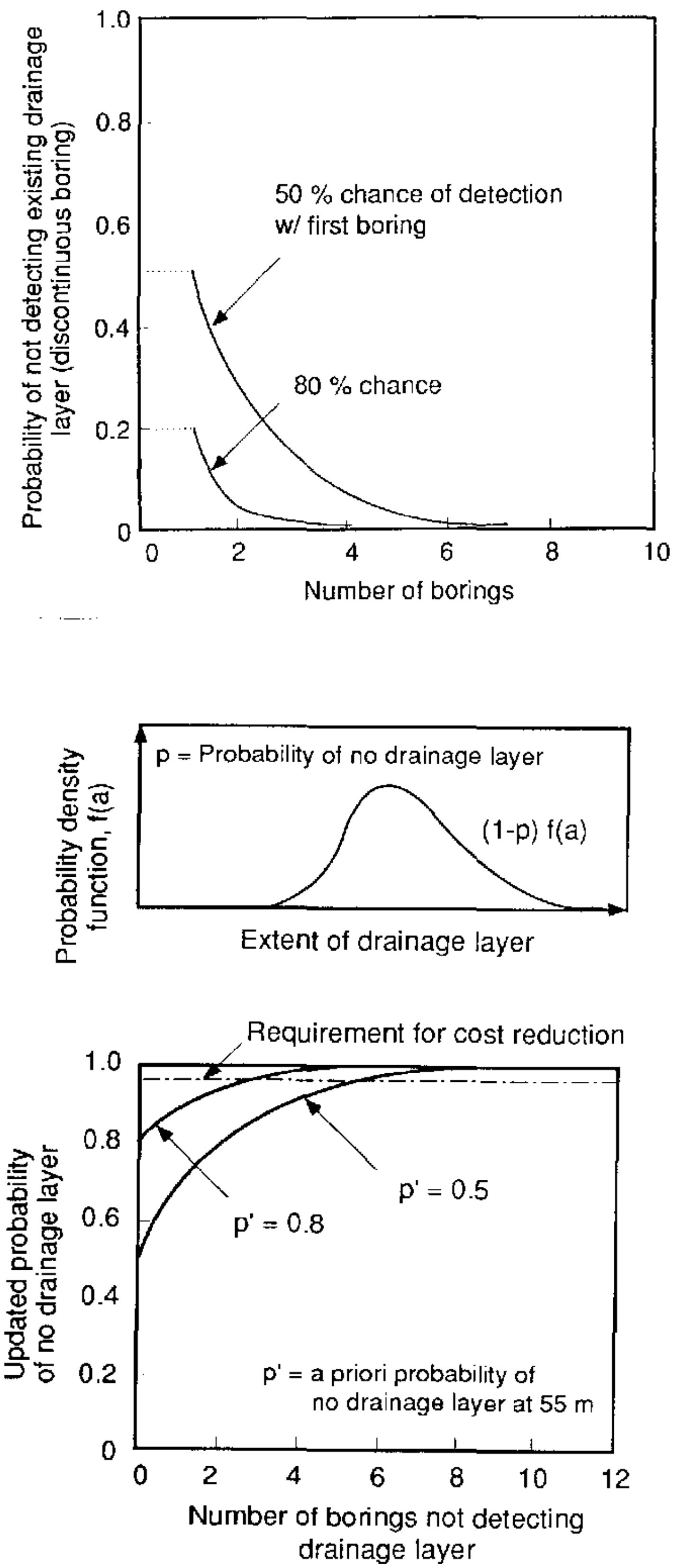


Fig. 3 Cost reduction with increased number of borings to detect a drainage layer

In this application, having no drainage layer present at a depth of 55 m was determinant on the resulting lifetime settlement. A settlement of less than 50 cm would mean a reduction in costs. If the probability of no drainage layer at a depth of 55 m was less than 2%, the settlement would not exceed 50 cm. With drainage layer detectability for each boring of 50 % or 80% and distribution of drainage layer extent as shown on Fig. 3, one would need 3 to 6 borings to enable the required cost reduction.

Dams

The concept of probabilistic risk analysis for dams was summarised by Whitman (1984). Important contributions can be found in the proceedings of an international conference on the safety of dams (edited by Serafim, 1984). The status of risk assessment for dams was made by Høeg (1996) and more extensively at an international workshop on risk-based dam safety evaluation in Trondheim, Norway (NNCOLD, 1997).

Vick (1997) summarised risk analysis practice in different countries, based on a survey of 11 countries (Australia, Austria, Canada, France, Germany, Holland, Norway, Sweden, Switzerland, United Kingdom and the United States of America):

1) Risk analyses for dams focus on safety and reliability of existing dams. The analyses are run to establish a diagnosis or set priorities among possible failure modes, to act as support in decision-making on issues related to dam safety modifications, and, to a slightly less extent, to establish budgeting priorities.

2) The analysis tools used are in order of frequency qualitative methods, event trees, Bayesian approaches with probabilistic characterisation of judgement, and reliability approaches with probabilistic characterisation of parameter uncertainty.

3) The results of the analyses are generally done on a case-by-case basis with no formal criteria, although nearly half of the countries used «f-n» curves (curves of probability of failure versus potential number of fatalities) as criteria, such as those summarised in Fig. 4.

There is a wide diversity in many aspects of reliability-based analysis of dams and its implementation. What is important is that there is a growing awareness that the uncertainties need to be evaluated and accounted for.

Canada (especially BC-Hydro) has taken a pioneering role in seeking to achieve greater consistency in the risk of their dams (e.g. Vick and Stewart, 1996). In Canada and Norway, a simplified probabilistic risk analysis has been applied in the re-evaluation and re-certification of rockfill dams, and to set priority on remedial measures. The analysis is effectively a systematic application of engineering judgement. The procedure consists of six steps (see Høeg, 1996 or Vick and Stewart, 1996 for more details):

1) Dam site inspection, including review of documents.

2) Failure mode screening, defining all failure modes, eliminating those that physically not possible.

3) Construction of event tree, listing failure sequences (events), and the interrelationship among events.

4) Probability assessment of reach event, often based on subjective beliefs, sometimes on observations and experience.

5) Evaluation of results: the failure probability for an outcome is evaluated from the product of the probability of each event occurring along any one branch of the event tree; the realism in whether a given combination of events (failure mode) has higher probability of failure than another is also considered.

6) Iteration: with the results of the first analysis, identify unlikely failure modes and the dam's vulnerability and strengths and include failure modes that were overlooked. This iteration can be repeated, as needed.

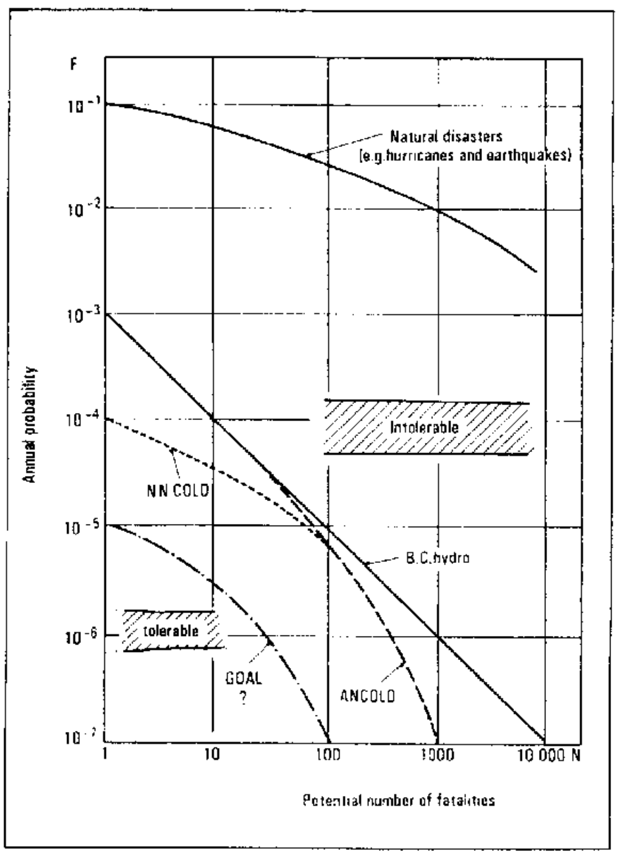


Fig. 4 Example of «f-n» curves for dams (Høeg, 1996)

To achieve consistency in the evaluation of the probabilities from one dam to another, conventions have been established to anchor the probabilities (Reagan et al., 1989). An example of descriptors of uncertainty used in the dam profession goes as follows (Vick, 1996):

Verbal description of uncertainty	Event probability
Virtually impossible	0.01
Very unlikely	0.10
Completely uncertain	0.50
Very likely	0.90
Virtually certain	0.99

Virtually impossible describes an event due to known physical conditions or processes that can be described and specified with almost complete confidence.

Very unlikely is as stated, although the possibility cannot be ruled out on the basis of physical or other reasons.

Completely uncertain is used when there is no reason to believe that one outcome is any more or less likely than the other to occur.

Very likely describes an event that is not completely certain.

Virtually certain is an event due to known physical conditions or processes that can be described and specified with almost complete confidence.

The example of Viddalsvatn dam completed in 1971 in Norway is a case where the risk of failure due to internal erosion was evaluated. Viddalsvatn dam, owned by Oslo Energi, is a rockfill dam with moraine core located in mid-Norway, south of Ålesund, east of Bergen. It has height of 80 m, length of 425 m and reservoir volume of 200 x 10^6 m^3. The dam, its performance and the risk analysis done, are described in detail by Johansen et al (1997). During the first years, Viddalsvatn dam experienced several incidents of briefly increased seepage, caused by internal erosion. Sinkholes developed on the crest of the dam following these. Automatic seepage monitoring is now carried out continuously. Grouting of the core has stopped further leakage and internal erosion.

The risk analysis followed the six-step procedure described above (Johansen et al., 1997). Steps 3 to 6 in the procedure was conducted in a workshop format with a group of specialists on the different aspects of the dam design. Potential failure sequences were broken down into individual events in either logical or temporal progression to form an event tree. Field observations or subjective field experience were considered. Hydrologic (100-yr. and 1000-yr. flood), earthquake and normal loading conditions were considered.

Figure 5 presents the event tree for failure by internal erosion under normal loading. The initial event (E_1) is the occurrence of a leakage incident. The branches in the event tree account for the possibility that such events might be progressive in nature (earlier such events of internal erosion have been self-healing in Norwegian dams). The branches present the possibility for seepage initiating toe unravelling of the rockfill, progressing to dam breach or «failure». Failure was defined here as the uncontrolled release of the reservoir. Additional branches and intervening events represent the leakage detection and reservoir drawdown response capabilities of the dam. Viddalsvatn dam has a set of branches propagating from event E_4 to account for possible failure due to breach of the dam from renewed sinkhole formation on its crest. This results in branches representing reservoir level (WL), sinkhole depth and the detection devices built in the dam. Events E_3 and E_4 are «dummy» branches in the event tree introduced to label the two failure modes. These «dummies» do not act as components in the probability assessment. The probabilities calculated show that the sinkhole leading to an overtopping failure mode is one order of magnitude less likely than the toe unravelling mode of failure.

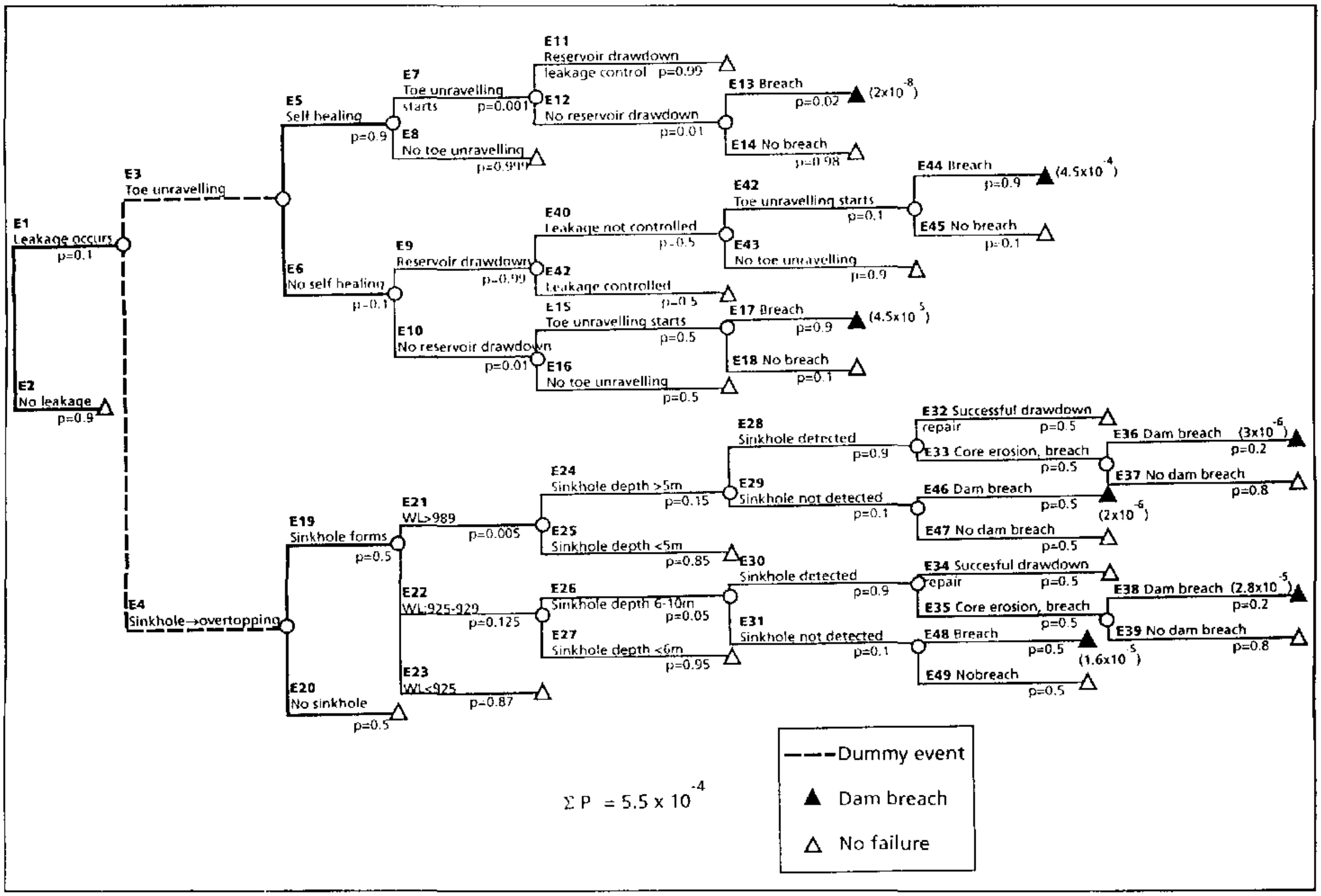


Fig. 5 Internal erosion event tree for Viddalsvatn dam in Norway

Each event in the event tree was associated with a probability of occurrence. The probabilities were obtained by first assigning a verbal descriptor as given above. The sum of the probabilities at any node is always unity, if all possible events have been included. The estimates relied heavily on engineering judgement. Observations were however also used. The possibility of progressive erosion was set as quite high (event E_6, p=0.1) because of the possibility of damage due to previous incidents that were observed. For the event of toe unravelling after self-healing leakage (event E_7), the probability is believed to be virtually impossible (p=0.001) because of the calculated rockfill discharge capacity and the dam's observed performance during three earlier such incidents. However should internal erosion be progressive, toe unravelling should be considered more likely (event E_{42}, p=0.1 and E_{15}, p=0.5), depending on the success of leakage control by reservoir drawdown (events E_9 and E_{10}).

Each outcome in the event tree ends up as dam breach or no dam breach. The annual probability of dam breach by internal erosion is then the summation of the probabilities of each outcome leading to dam breach, or $P = 5.5 \times 10^{-4}$.

Some component events were treated statistically, for example the 100-yr. and 1000-yr. flood were based on historic data, and the earthquake frequency and response spectrum were based on the Norwegian database for earthquakes. The events represent the expected response of the dam following an initiating event.

The calculation with an event tree for each of the loading cases (an event tree was built for each of the flood, earthquake and normal loadings) resulted in the following annual probabilities of failure:

Loading	Annual probability of failure
Flood	1.2×10^{-6}
Earthquake	1.1×10^{-5}
Normal (internal erosion)	5.5×10^{-4}

The total annual probability of failure for all modes is the sum of the three components, or 5.6×10^{-4}. The results represent a relative order of magnitude for the different scenarios. They should not be interpreted to be an accurate probability.

In practice, the results of the analysis proved even more useful when done on several dams and compared, as was done in Johansen et al. (1996).

Offshore structures

The Norwegian offshore industry has been at the forefront in applying reliability-based analysis to assist in decision-making. This, associated to the fact that all types of foundations are very costly and often heavily instrumented, has contributed to the documentation of case studies where reliability concepts have been used.

The offshore structure case study selected for this paper presents the deterministic and probabilistic analyses of an offshore pile foundation at two times in its lifetime:

1) In 1975, before platform installation, when limited information and limited methods of interpretation of the soil data were available

2) In 1993, after a reinterpretation of the available data using the geotechnical improvements attained in the interim additional and more advanced laboratory tests, a re-analysis of the loads, and an analysis of the installation records

The re-analysis in 1993 was prompted because the environmental loads had been revised, the structure had been hit by a ship by accident, and the operators hoped to increase the loads on deck. The structure is a steel jacket installed in 110 m of water in the North Sea in 1976. The jacket rests on four pile groups, one at each corner. Each pile group consists of six piles (Fig. 6). The piles in the groups are 60" diameter tubulars, with wall thickness of 3 and 2.5".

Geotechnical investigations. The soil profile consists of mainly stiff to hard clay layers, with thinner layers of very dense sand in between. In 1975, two soil borings were done at the jacket location. The two borings indicated somewhat comparable soil profiles, although the horizon and the thickness of the sand layers differed. Based on the information obtained from the borings, the soil characteristics in Fig. 7 were derived from the standard "strength index" types of tests in common use at the time: (torvane, pocket penetrometer, unconfined compression test, unconsolidated undrained (UU) test), and an interpretation of the results based on the judgement and experience of the geotechnical consultant at the time. The friction angle of the dense sand was based on the results of consolidated drained triaxial compression tests on recompacted specimens. The friction angle for the specimens compacted to the highest density possible was measured as 38-40 degrees.

The profiles selected in 1975 showed a wide variation in the soil parameters, with considerably higher shear strength below 20 m if one believed the one boring. The position of the second sand layer differed for the two borings. No advanced laboratory tests enabled one to estimate more appropriate values for the soil parameters. There is no doubt that sampling disturbance must have contributed to the scatter in the results.

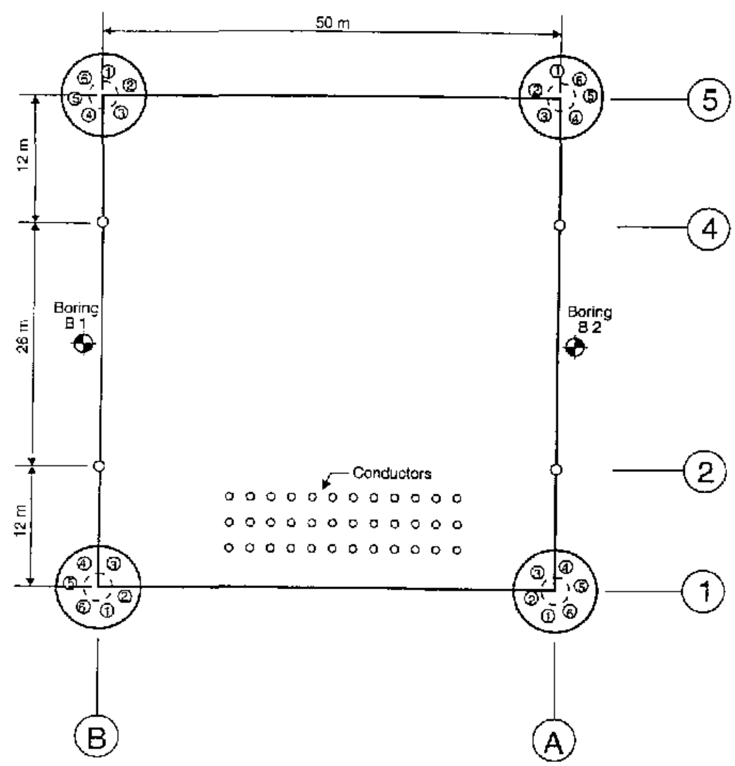


Fig. 6 Offshore structure - Pile foundation layout

During pile installation, records were made of the blow count during driving. The individual blow counts for the piles in Leg B1 are given in Fig. 8. No instrumentation of the driving operation was done. The installed pile lengths were between 36 and 45 m. The pile driving records were evaluated by a consultant in 1993 and used to adjust the soil stratigraphy (Fig.

9). In 1993, new samples were also taken and more advanced strength tests were run, including consolidated undrained triaxial compression tests.

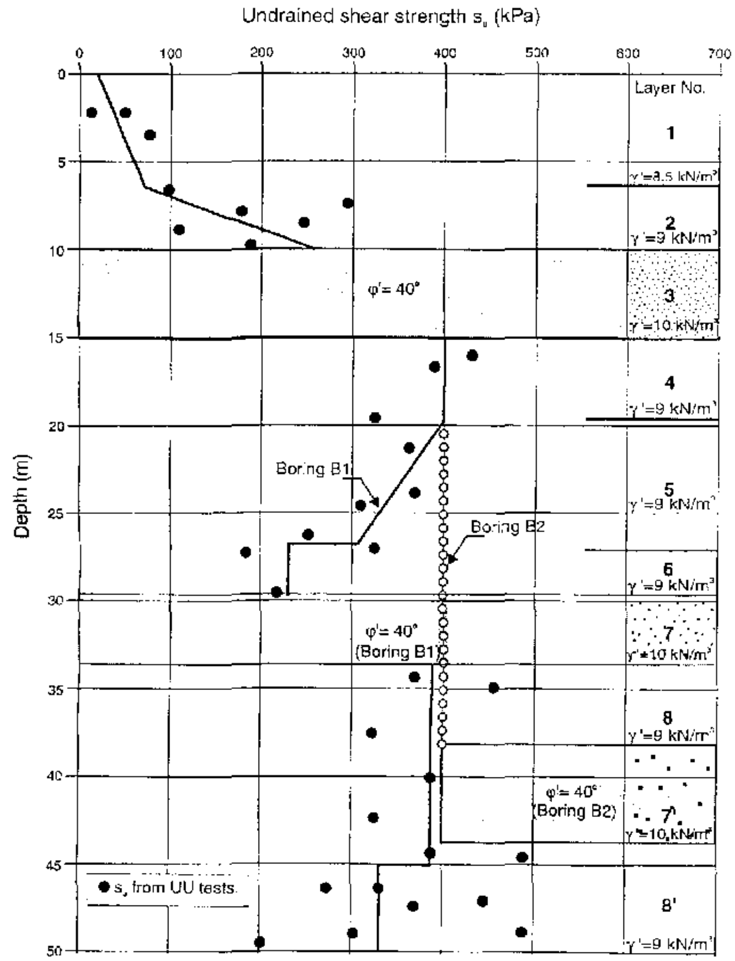


Fig. 7 Offshore structure - Soil profile for 1975 analyses

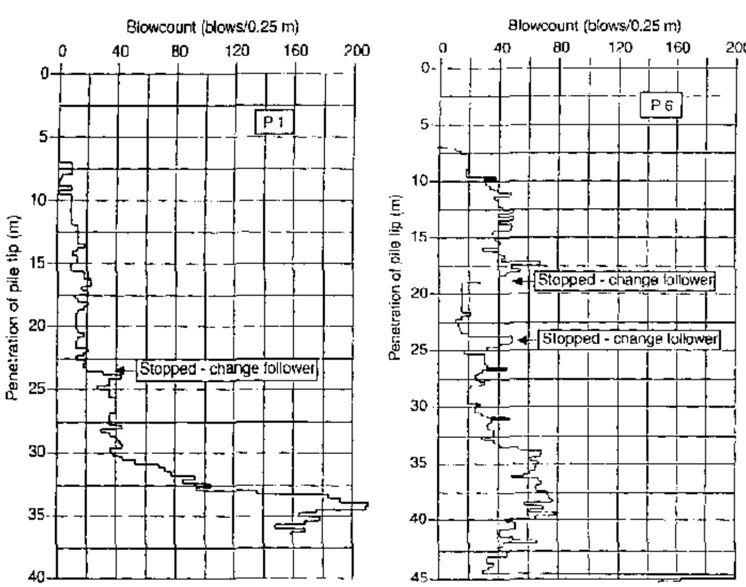


Fig. 8 Offshore structure - Pile driving records, leg A5

The result of this "educated" adjustment on the basis of the pile driving, and a re-evaluation of the borings and laboratory test results using normalised soil characteristics, new soil samples and the running of more advanced laboratory tests (direct simple shear tests, consolidated undrained triaxial tests) led to the adjusted soil shear strengths in the stiff to firm clay shown in Fig. 10, where a much narrower range of soil strengths is suggested. The full curve represents pessimistic values, the dotted line the best estimate values. No re-evaluation of the friction angle in sand was done, although ideally, this should have been done.

Analysis. The deterministic analyses were done with the API RP2A recommended practice in use at the time of the analysis. The design requirement at both times in the platform lifetime was a factor of safety of 1.50 under extreme loading and 2.0 under operation loading. The axial pile capacity is a summation of the skin friction on the pile shaft and end bearing on the pile tip.

The probabilistic analyses were done with first-order reliability method (FORM), where the deterministic axial pile capacity model was formulated in terms of random variables in each layer. In this paper, only the results of the analysis of the capacity of most loaded pile are considered.

Geotechnical parameters in model. Table 1 gives the uncertainties associated with the soil parameters in two of the more important soil layers. The coefficients of variation reflect uncertainties in the laboratory test results, possible measurement errors, spatial variability and the uncertainty in degradation due to cyclic loading. Cyclic degradation is important for an overconsolidated clay subjected to a fairly high ratio of cyclic loading. The effect of cyclic loading is expected to be minor for the dense sand. Very little data were available for the different soil parameters. The mean and coefficients of variation were obtained as follows:

Submerged unit weight, γ': No measurements were available. The mean value and coefficient of variation were based on experience acquired for similar soils where many measurements have been taken. For stiff clays, the mean submerged unit weight is 8.5 to 9.0 kN/m³ (as stiffness increases); for very dense sand, the mean submerged unit weight is normally 10 kN/m³. A coefficient of variation of 5% is a common value for scatter in submerged unit weight.

Depth, z: The layer thickness can vary. Since only two borings are available, the values used in the analysis are uncertain. The mean layer thickness is based on the measured values from the site investigations. The coefficient of variation of 10% is based on engineering judgement. The position and thickness of Layer 7 were quite uncertain in 1975. For this reason the coefficient of variation was increased for this layer from 10 to 20%.

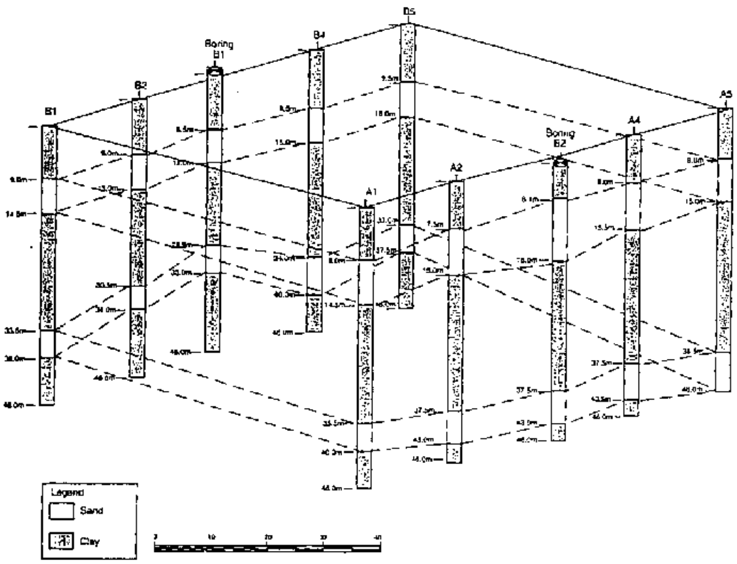


Fig. 9 Offshore structure - Stratigraphy inferred from pile driving and re-analysis of soil data

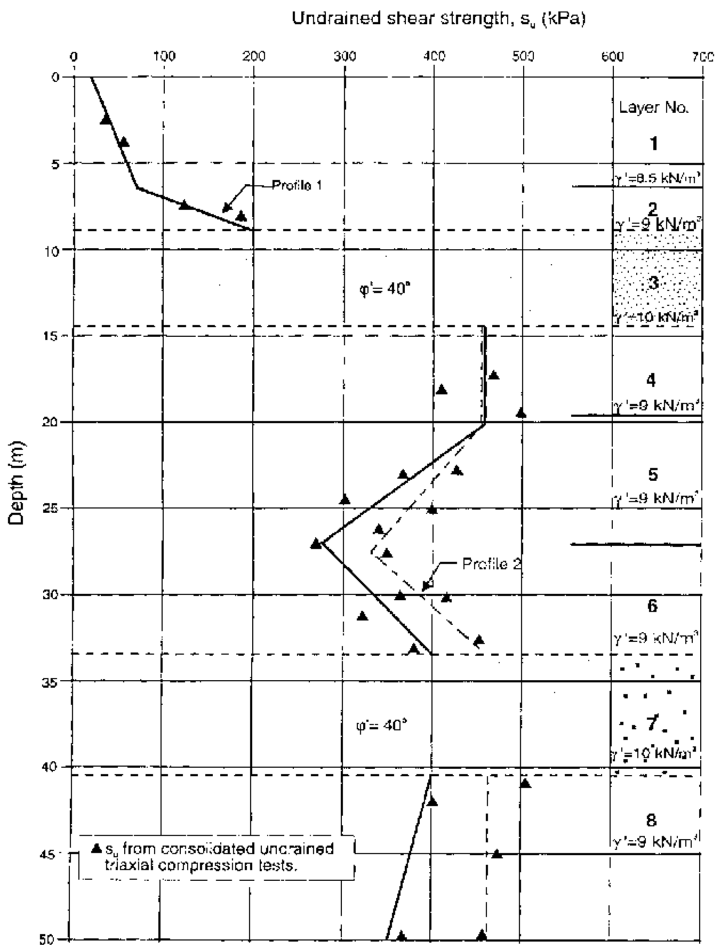


Fig. 10 Offshore structure - Soil profile for 1993 analyses

Undrained shear strength in stiff clay, s_u : In 1975, the undrained shear strength was based on punctual measurements from index strength tests, known to give a relatively poor estimate of the undrained shear strength. The data points are shown in Fig. 7, which explain the high coefficient of variation. In 1993, the undrained shear strength profile was based on:

(1) results of consolidated-undrained triaxial compression tests at effective stresses relevant for the in situ values

(2) a recalculation of the soil shear strength based on the normalised strength ratio for similar clays within the same geographical area and with similar geological history.

This led to new soil strength profiles in 1993 and Coefficients of variation of 10 or 15%.

Friction angles, ϕ' and δ, and coefficient of earth pressure, K, in very dense sand: Very little information was available for the very dense sand layers. A friction angle, ϕ', of 40° (and soil friction angle δ of 35°) is typical for a very dense sand. In 1975 there was little known about this angle and the coefficient of variation was set to 15%. In 1993, considerable research contributed to reducing this coefficient of variation to about 5%. Lacasse and Goulois (1989) collated the opinion of 40 international experts who suggested that the uncertainty about the mean is quite small. For the coefficient of earth pressure, K, values are undocumented, but based on engineering judgement, experience and the results of the Lacasse and Goulois (1989) study.

Pile capacity parameter in clay, α: The prediction of the axial capacity of a pile in clay is done with the friction parameter, α, times the undrained shear strength. The mean value is based on the API RP2 A guideline. The coefficient of variation is based on engineering judgement and the experience gathered for piles in stiff clay.

Pile capacity parameter in sand, f_{lim}: The mean value of f_{lim} is specified by the API RP2 A design guidelines. The decrease in the coefficient of variation of f_{lim} from 25 to 15% between 1975 and 1993 reflects the understanding acquired over the year on pile friction in sand and the results of the expert opinion pooling summarised in Lacasse and Goulois (1989).

Table 1 Examples of uncertainty in soil parameters in Layers 5, 7 and 8 - 1975 and 1993 analyses

		Coefficient of variation		
Layer	Variable	1975-analyses	1993-analyses	PDF
5	γ'	5 %	5%	N
	z	10 %	10%	N
	s_u	25 %	15%	LN
	α	10 %	10%	LN
7	γ'	5 %	5%	N
	z	20 %	10%	N
	K	15 %	10%	N
	δ	15 %	5%	N

	f_{lim}	25 %	15%	N
8	γ'	5 %	5%	N
	z	10 %	10%	N
	s_u	25 %	10%	LN
	α	10 %	10%	LN
	N_c	15 %	15%	N

Notation:

γ' = submerged unit weight z = depth to bottom of layer
s_u = undrained shear strength α = skin friction factor
N_c = bearing capacity factor K= coefficient of earth pressure
δ = soil-pile friction = ϕ'-5° ϕ' = friction angle (of sand)
f_{lim}=limiting skin friction(sand) N/LN=normal/lognormal PDF
PDF= probability distribution function

Loads in model. The characteristic load used for deterministic foundation design of fixed offshore structure on the Norwegian Continental Shelf is defined as the load with an annual occurrence probability of 1% (i.e. the maximum load associated with the 100-year storm).

The extreme axial load on the most loaded pile is the sum of a permanent component resisting the submerged platform weight and a transient (cyclic) component resisting the storm, current and wind-induced forces. The key parameters entering the load calculations are the environmental characteristics, the platform weight, and the model used for estimating the response of the platform to the environmental loads.

The main environmental parameters for the foundation loads of the platform under consideration were the significant wave height (H_s) and the spectral peak period corresponding to the significant wave height ($T_p|H_s$).

Data on storm characteristics were gathered during almost two decades of platform operation. A 100-year value for the significant wave height of 13.5-14.5m was expected for the area of the North Sea around the site, so a storm threshold of H_s = 7m was used in the calculations. A total of 130 events exceeding this threshold were observed during the time period summer 1975-summer 1992. A truncated Weibull distribution was used for the significant wave height and a lognormal distribution (conditional on H_s) was adopted for the spectral peak period. To quantify the uncertainty in the significant wave height for the 100-year event, the fitted Weibull model parameters were treated as random variables (Haver and Gudmestad (1992).

The procedure used for estimating the foundation loads in the design phase was deterministic. To make a comparison, similar types of distributions were assumed for the environmental parameters in 1975, but the site specific data were not used in fitting the distribution parameters. Rather, the distributions were chosen to be representative of the general area of northern North Sea, which meant that there was a larger dispersion in the parameters.

To obtain the unconditional distribution of the 100-year axial load on the most loaded pile (Leg A5), the probability of exceeding a given load level was estimated using the FORM and SORM approaches. The load level was varied and the results were plotted on the Gumbel scale as shown in Fig. 11. As seen, a Gumbel distribution with mean of 20 MN and coefficient of variation of 10% provides a good fit to the extreme axial pile load based on the 1993 information, whereas with the information available in 1975, the same load has a mean of 19 MN and a coefficient of variation of 15%.

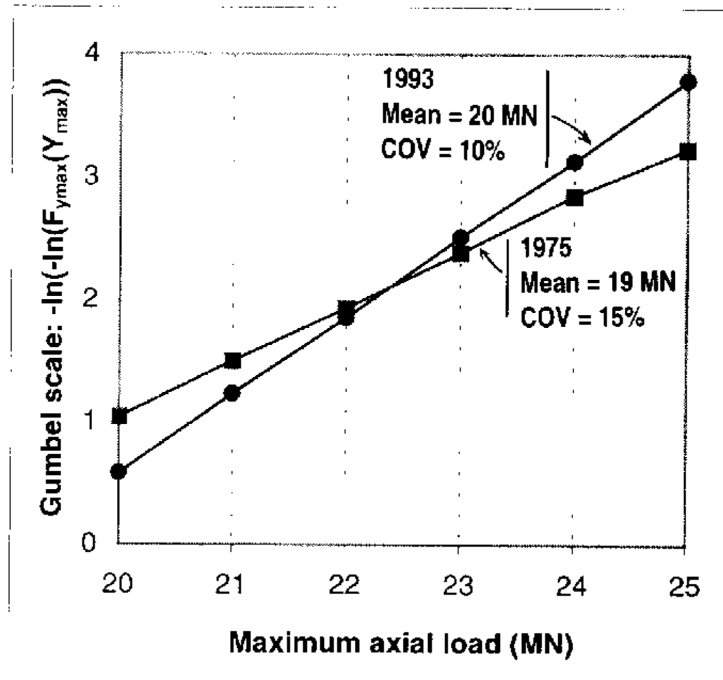


Fig. 11 Assumed distribution of the 100-yr extreme axial load on Pile P2 in Leg A5

In both situations, the significant wave height was the dominant random variable contributing about 80% of the uncertainty in axial pile load. Model uncertainty was the next most important parameter. The contribution of other random variables such as the spectral peak period, submerged platform weight, and wind characteristics was negligible. The cyclic component (due to design storm) represented about 40%, and the static component (due to submerged weight) represented the remaining 60% of the extreme axial load in 1975. In 1993, the cyclic component represented about 35% of the revised axial load. The reduction of uncertainty in the extreme axial pile load reflects the change in knowledge with increased research, almost two decades of site-specific wave data, and the increased proportion of the gravity load on the total axial load.

Model uncertainty in axial pile capacity calculation. In the probabilistic pile capacity analysis, a variable describing the uncertainty in side friction calculation in each layer was used. An independent model uncertainty variable in each layer is required because the soil type can vary from one layer to the other and different resistance mechanisms need then to be considered. In the bottom layer, two model uncertainty variables should be considered: the first applying to the side friction calculation and the second to the end bearing calculation. The duality of model uncertainty in the last layer is important because side friction and end bearing are two different resistance mechanisms which are modelled by different equations. The model uncertainty variables were taken as normally distributed.

In a dense to very dense sand, the uncertainties due the calculation model can be very large, and the bias is believed to show a lot of conservatism in the API RP2A method. The uncertainties are believed to be far greater for piles in sand than for piles in clay. The model uncertainty values used in the analyses were based on the study by Lacasse and Goulois (1989) for sand, and on several NGI research projects for clay (Lacasse and Nadim, 1996a).

The API RP2A (1993) model for side friction is believed to predict quite well the side friction in softer clays. The bias is probably 1.00 for both normally and overconsolidated clays (Lacasse and Nadim, 1996a). These values were evaluated from back-calculations of model tests and comparisons of several methods of analyses. The coefficient of variation was taken as 0.15 to reflect the lack of knowledge for pile driven in clays with high undrained shear strength and high (unknown) overconsolidation ratios.

On the other hand for piles in sand, the bias in the side friction model increases as the density of the sand increases. A bias of 1.00 is expected in loose to medium sand. For dense and very dense sands, the bias is higher, based on recent, and still unpublished, prototype-size pile loading tests. A bias of 1.10, with coefficient of variation of 0.15, was used in the analyses.

For end bearing in very dense sand, the existing calculation model is generally believed to be conservative (Lacasse and Goulois, 1988). For this reason, the mean of the model uncertainty was taken as 1.20, with a coefficient of variation of 0.15 to reflect the lack of good reference pile load tests with comparable pile size as used offshore.

Results of analyses. The results of the analyses are summarised in Table 2. In 1975, only deterministic calculations were carried out. The 1975-probabilistic calculations were run in 1994.

The values of P_f and β in Table 2 are conditional values given the 100-year storm occurs. They should not be confused with the annual failure probability and reliability index.

Figure 12 illustrates schematically the results of the reliability analysis of the most loaded pile for the offshore jacket used in this example.

Table 2 Results of 1975 and 1993 deterministic and probabilistic analyses - Pile P2 in Leg A5 (penetration depth = 40.8 m)

Soil Profile	Deterministic factor of safety	Reliability index β	Probability of failure, P_f
1975	1.73	2.06	$2.0 \cdot 10^{-2}$
1993	1.39	2.41	$0.8 \cdot 10^{-2}$

The newer deterministic analysis gave a low safety factor (FS), a situation of major concern since the safety factor was below the minimum required factor of safety under extreme loads of 1.50. However the added information reduced the uncertainty in both soil and load parameters. The pile with a safety factor of 1.39 is nominally safer than the pile was believed to be in 1975 where the safety factor was 1.73. The probabilistic analyses showed that the pile, although with a lower safety factor, had higher safety margin than perceived at the time of design. The lower uncertainty in the parameters in the newer analysis caused a reduction in the probability of failure (P_f) by a factor of 2.5.

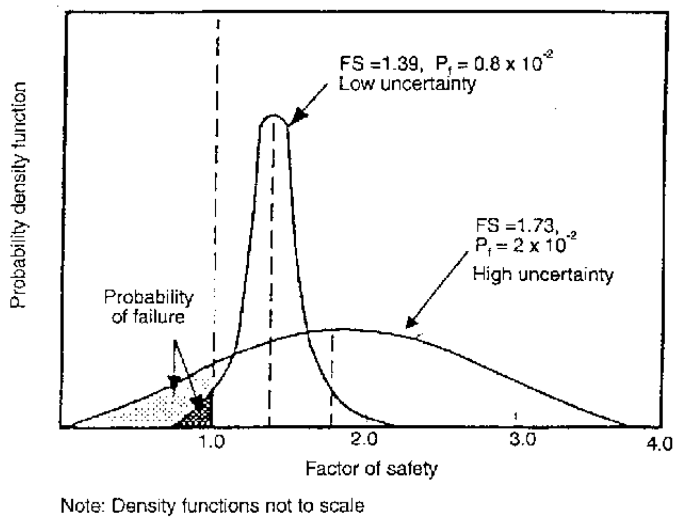


Fig. 12 Illustration of safety factor and probability of failure for most loaded pile in case study of offshore jacket

The factor of safety is therefore not a sufficient indicator of safety margin because the uncertainties in the analysis parameters affect probability of failure, but these uncertainties do not intervene in the deterministic calculation of safety factor.

As for deterministic calculations, the essential components of reliability estimates in geotechnics are (1) a clear understanding of the physical aspects of the geotechnical behaviour to model and (2) the experience and engineering judgement that enter into all decisions at any level, whether for parameter selection, choice of most realistic analysis model, or decision-making on the viability of a concept. As illustrated in the case study, the most important contribution of reliability concepts to geotechnical engineering is increasing the engineer's awareness of the existing uncertainties and their consequences.

Shallow foundations

Case study 1. The first case study calculates the limiting equilibrium analysis of gravity platform (offshore, but the approach is the same on land) installed on a uniform soft plastic clay. As for a deterministic analysis, the approach took into account the different stress conditions along the potential slip surface since the probabilistic formulation is exactly the same as the deterministic one. The potential slip surfaces (Fig. 13) were analysed individually and as a system with all potential failure surface included.

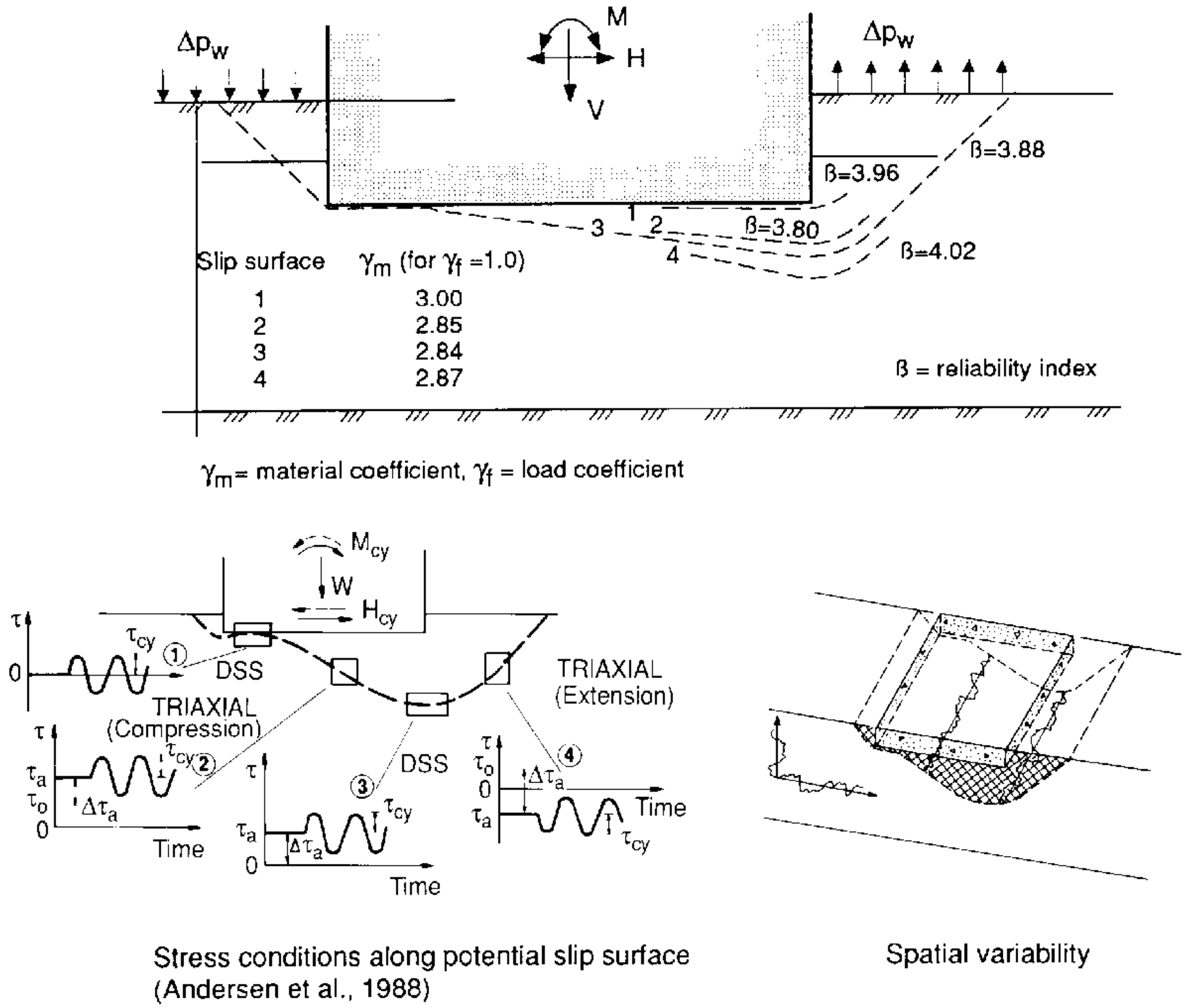


Fig. 13 Results of probabilistic analysis of bearing capacity of shallow foundation

Spatial variability, which can reduce the uncertainty in the soil properties such as undrained shear strength of the clay, was included (Vanmarcke, 1977; 1984). The coefficient of variation of the extreme environmental loads was taken as 15 %, the horizontal load and moment were taken as perfectly correlated. The uncertainty in the soil parameters at the soft clay site was very low because of the exceptional homogeneity of the deposit.

The reliability analyses indicated the following:

- The critical slip surface based on the highest probability of failure was different from the critical slip surface based on the results of deterministic analyses. This is seen repeatedly for different soil profiles and illustrates well that the uncertainty in the analysis parameters plays an important role on the margin of safety. The discrepancy is due to the different uncertainties in the triaxial compression and triaxial extension strengths used in the equilibrium analysis.

- Based on the results of analyses of gravity structures on both soft and stiff clay, model uncertainty and moment were very significant uncertain variables. For the soft clay, this was partly due to the homogeneity of the site.

- First-order, second-order and improved second-order approximations gave same probability of failure. The simpler first-order approximation is therefore sufficient.

- Changing the probability distribution of the soil parameters from normal to lognormal had only a modest effect on the computed probability of failure.

- The reliability analysis including all failure surfaces resulted in a probability of failure equal to that of the most critical failure surface. (The same conclusion was true with different failure modes). The most critical slip surfaces were essentially perfectly correlated.

Case study 2. Probabilistic stability analyses were done using the «mobilised friction angle» approach (an effective stress approach) and the «available shear strength» approach (based on the undrained shear strength of the soil). The two approaches define factor of safety with two different formulations:

- the ratio between the undrained shear strength and the shear stress mobilised for equilibrium

- the ratio between the tangent of the characteristic friction angle and the tangent of the friction angle being mobilised at equilibrium.

Both analysis methods are often allowed in code of practice.

Shallow foundations on two soil types were considered: a contractant soil (loose sand, normally consolidated clay, path DE on Fig. 14) and a dilatant soil (dense sand, heavily over-consolidated clay, path DG on Fig. 14). The «true» safety margin for the foundations for both soils is independent of the method of analysis.

Table 3 presents the results of the calculations. Depending on soil type, the computed nominal probability of failure differed appreciably for the two approaches. The probabilistic and deterministic results showed significant differences, especially for the dilatant soil, as the uncertainties in the soil properties interacted differently in each approach

For the «mobilised friction angle» approach, uncertainties in friction angle, cohesion, pore pressure parameter and submerged unit weight were considered. For the «available shear strength» approach, uncertainties in undrained shear strength and submerged unit weight were included. To «calibrate» the two analysis methods, a model uncertainty factor would have to be included.

This case study documents again how wrong an impression a safety factor alone can give of the actual safety margin against failure.

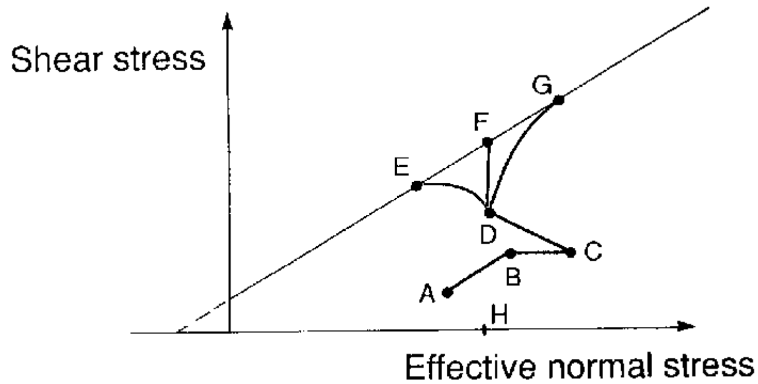


Fig. 14 Effective stress paths for contractant and dilatant soil

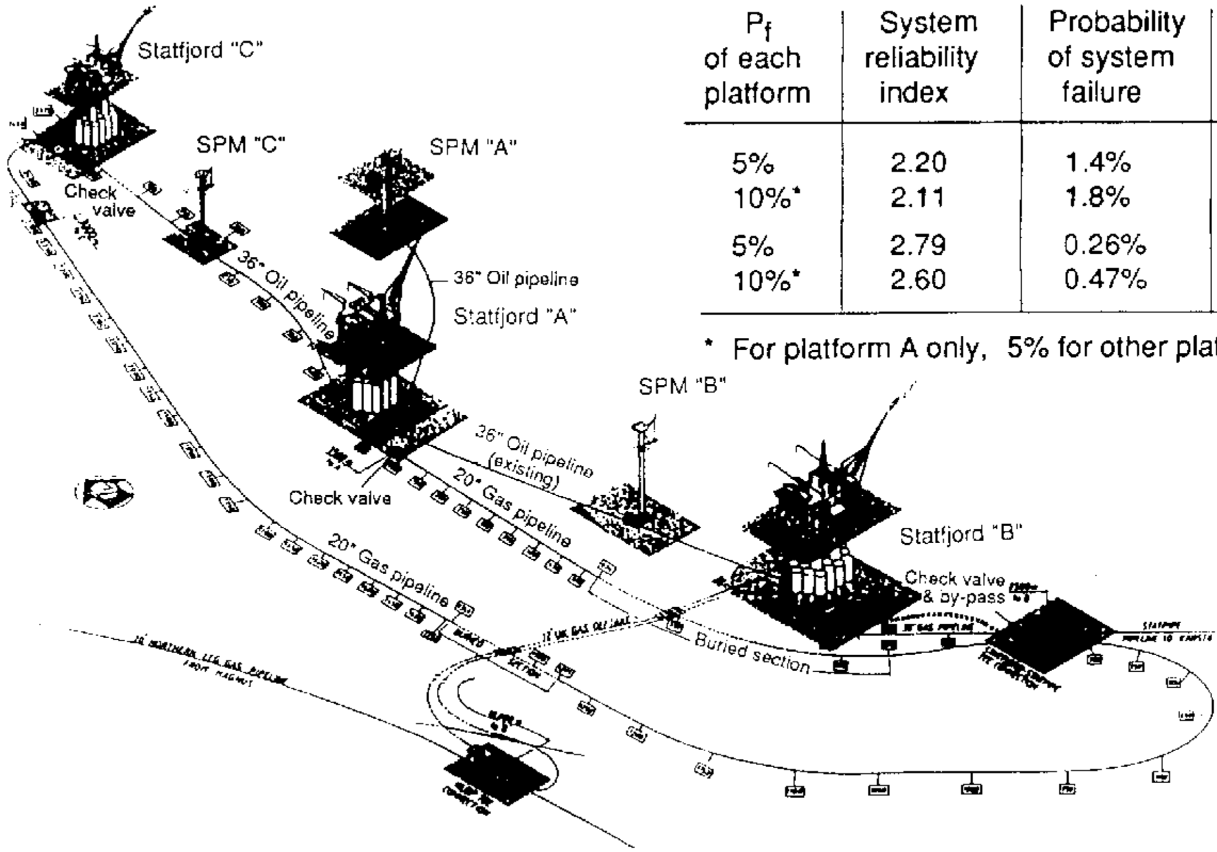
Table 3 Results of stability analyses with two approaches (Nadim et al., 1994)

Soil type	Analysis method	Factor of safety	Probability of failure
Contractant	Mobilised friction angle	1.9	1.7×10^{-5}
	Available shear strength	1.4	2.5×10^{-3}
Dilatant	Mobilised friction angle	1.4	6.7×10^{-3}
	Available shear strength	1.5	2.3×10^{-6}

Earthquake response

Figure 15 presents an application of the seismic reliability analysis of a group of offshore platforms that answers the following question: given that a strong earthquake with 10^{-4} annual occurrence probability takes place at the Statfjord oil field, what are the chances that oil production must be stopped completely? The seismic reliability was evaluated by considering the possible failure modes of the platform network, the correlation between the failure modes, the seismic reliability of each platform and the spatial variation of the earthquake peak ground acceleration.

A typical gravity platform designed on the basis of the Norwegian Petroleum Directorate guidelines has an implied probability of failure of 5 % under the 10^{-4}/year earthquake. Analyses were done with 5 % probability of failure for each platform taken individually. The effect of increasing the failure probability of the Statfjord A platform to 10 % was also considered. As listed on Fig. 14, the reliability of the system was much greater than the reliability of each platform. Accounting for the spatial variation of the earthquake loading parameters reduced the probability of failure by a factor of about 5 (Nadim and Gudmestad, 1994).



P_f of each platform	System reliability index	Probability of system failure	Earthquake motion
5%	2.20	1.4%	No spatial
10%*	2.11	1.8%	variation
5%	2.79	0.26%	Spatially
10%*	2.60	0.47%	variable

* For platform A only, 5% for other platforms

Fig. 15 Platform and pipeline layout at Statfjord field and probability of system failure (Nadim and Gudmestad, 1994)

Other applications

Other application include risk assessment for slopes and landslides, rock mechanics and mining problems.

Within slopes and landslides, a recent proceedings of a Workshop organised by the International Union of Geological Science (Cruden and Fell, 1997) presents the state of the art. and the interested reader can find a complete summary of the approaches used, criteria (or lack of criteria) for tolerable risk and recommendations for further work. Many of the concepts follow the lines of the risk assessments for dams. The authors expose which analysis they feel confident with, and which ones need to be developed in more detail.

Christian (1996) presented cases studies of slopes showing that the ones with highest safety factors are not necessarily the safest. He concluded that a simple prescription of a factor of safety is not realistic and may lead to either too conservative or not conservative enough designs. The reliability approach provided a framework that provided more useful information than the safety of factor alone. Christian pointed out though that doing the calculation brings up «troublesome» questions, and the engineer needs to put some thought in selecting the critical parameters and estimating the uncertainties. Malone (1996) also described risk management of slope safety in Hong Kong. His co-workers have published a number of case studies (e.g. Hardingham et al, 1996).

Recent contributions present reliability analysis of reinforced embankments. The reliability calculations are done on a spreadsheet and give the same results as the more complex first and second order analyses (Low and Tang, 1997).

Probabilistic modelling is also possible for fluid transport in porous media, for example contaminant transport (Woodbury, 1997; Sitar, N., Private communication, University of California at Berkeley, 1996; Dimakis, P., Private communication, University of Oslo, 1996).

Einstein (1996) gave three detailed examples of the application of risk-based methods for rock engineering; slope design, flow through fractured media and tunnelling.

The decision-making reported by Einstein, when uncertainties were present, was based on risk analysis. The process is illustrated in Fig. 16. It involves collection of information and establishing relations between parameters and performance, both deterministically and probabilistically. The decision for a specific design alternative was made on the basis of comparing predicted and required performance.

Einstein (1996) concluded from three case studies that it is necessary to include the uncertainties in the analyses in order to facilitate and validate the decision-making that would ensure that the performance would satisfy the prescribed criteria. The information obtained from the uncertainty and risk analysis was of a major contributor for decision-making.

As for other geotechnical problems, collection of information, determining uncertainties, modelling the performance and determining the consequences were the key elements of the assessment. These four steps are none others than the steps usually carried out by the geotechnical engineer or rock mechanics specialist (and not by the statistician or probabilist).

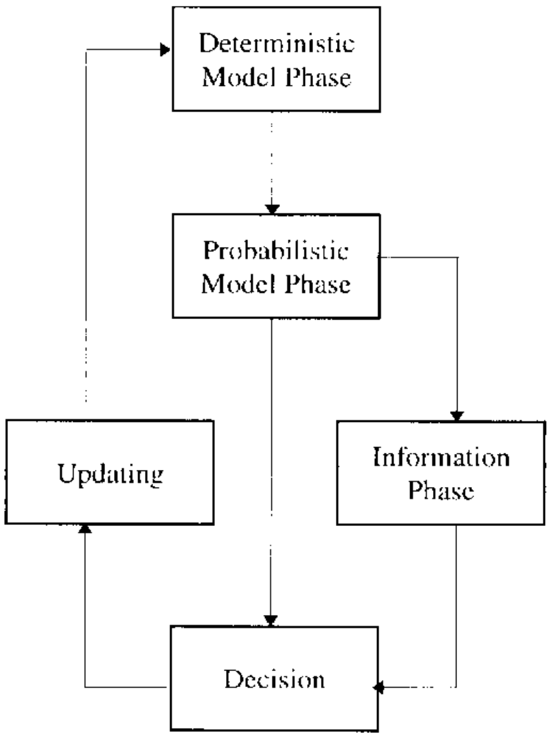


Fig. 16 Decision-making under uncertainty (Einstein, 1996)

WHY ARE THE METHODS NOT USED MORE?

There has been, and there still is, reluctance to use probabilistic and risk analyses in different areas of geotechnical engineering. The tools exist, they are generally easy to apply, and the engineer recognises that there are uncertainties, and even that they can be very large. Then, why are those tools not used more frequently?

There may be several reasons for this: distrust of new terms, which, as some wrongly believe, could mean complex mathematics; the belief (wrong again) that running such analyses will bring large added costs; the unfounded fear that such analyses will replace existing methods or even sound engineering judgement; the impression that the procedures require a large effort of information collection and modelling; the belief (wrong here too) that the good old «sound engineering judgement» is excluded from the procedure and the ensuing decision-making process.

The probabilistic community needs to take some of the blame for the slow ingress of the approaches in practice, because at the start, the concepts were not explained such that the practising engineer could assimilate them easily. Some vulgarisation has been done since, and the application of the methods has greatly simplified over the years.

Most published risk analyses focus on the result and how it was used. Information on how it was actually done is generally sparse and the intermediate steps not documented. Risk analysis is straightforward, it is the engineering behind it and after the results are obtained that is the important part, and often little is said about it.

To refute some of the myths associated with reliability-related analysis methods, the authors maintain the following:

- One does not need to be a mathematician to use reliability methods, just like one does not need to know how to program in the C++ or FORTRAN language to use Windows-based or finite element programs. What one does need to understand is the output of the calculation, to be able to evaluate the reasonableness of the results. That necessity is not particular to reliability approaches, it is a requirement for any geotechnical calculation.

- Reliability and risk analyses will never replace the traditional (deterministic) analyses. The deterministic analysis is an integral part, in fact the basis, of the probabilistic/risk assessment. The probabilistic or risk analyses cannot be carried out without its deterministic counterpart.

- Are reliability analyses more expensive than deterministic analyses? One needs an initial investment in time to code the probabilistic approach in a program or a spreadsheet; once that is done, running the analysis for the same geotechnical problem requires little engineering time. (Programs for many typical geotechnical problems have for example been developed at NGI).

- One may argue that time is required to evaluate the uncertainties in the soil and load parameters entering the analysis. The authors maintain that an evaluation of the uncertainties should be done in any design: in a traditional analysis, the designer needs to know or estimate the consequence of the assumptions he made and how good the assumptions are. Neglecting to assess the uncertainties in a design analysis where failure to perform can result in damage or even more severe consequences is simply not good engineering and not responsible. Reliability approaches enable the engineer to systematise the uncertainties and the treatment of these, at the same time as they provide the effect of the uncertainties on the predicted performance.

- Judgement, as illustrated by the case studies in this paper, is not excluded from risk and reliability analyses: judgement can be formally included, and one can even examine the effects of this additional uncertainty on the results obtained.

Establishing acceptable risk levels

Establishing the basis for acceptable risk criteria is difficult and controversial. Society requires now, with increasing frequency, that analyses be done to determine the level of risk imposed on the public (as opposed to voluntary risks, like driving one's own car).

Risk statistics for persons voluntarily or involuntarily involved to hazards range from 1 x 10^{-5} death per year for air travel, 3 x 10^{-4} for road accidents to 2 x 10^{-3} for parachuting Californians accept to live in Parkfield or the San Francisco Peninsula where there is a 90 % and 20 % respectively probability of a major earthquake on the San Andreas Fault occurring between year 1988 and 2018.

Figure 17 presents a compilation of probabilistic risk for projects vs economic and human losses resulting from failure which can be used as a guideline (modified from Whitman, 1984). Figure 18 shows the risk criteria chart proposed by B.C. Hydro for dams, and Fig. 19 illustrates the risk evaluation guideline proposed by USBR (quoted by Whitman, 1997). None of these represent «official» risk criteria, they are really the starting point of a discussion which some day needs to be finalised.

The proposed guidelines have in common that they are essentially based on engineering judgement and experience, and suggest somewhat similar bounds of acceptable and unacceptable levels of annual probability of occurrence. The engineers, because of their understanding of both technical and safety issues, are the ones who can and need to establish the «acceptable risk», based on the design standards and degree of belief in our methods.

ADVANTAGES OF RELIABILITY METHODS

What is not generally recognised is that the concepts and the approaches may be used for different purposes and at different levels, for example (modified from Høeg, 1996):

- during design, to place the main design efforts where the uncertainties and the consequences of these on design and costs are greatest;

- during operation of major or critical engineered facilities, to enable the engineer to have at hand a number of action scenarios depending on the observed response of the structure;

- when selecting among different remedial actions, which are always influenced by time, financial and logistics constraints;

- to relate risk levels of a construction to other tolerable risk levels.

Establishing risk levels represents the most complex aspect of the risk analysis, but the benefits of the other three aspects are often overlooked.

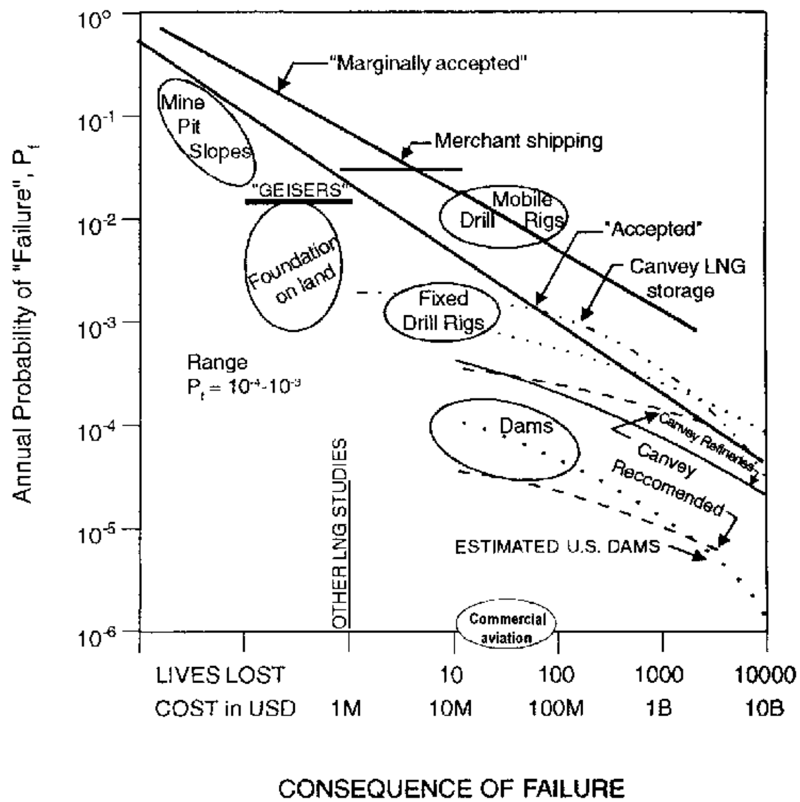


Fig. 17 Probabilistic risk for projects vs economic and human losses (modified from Whitman, 1984).

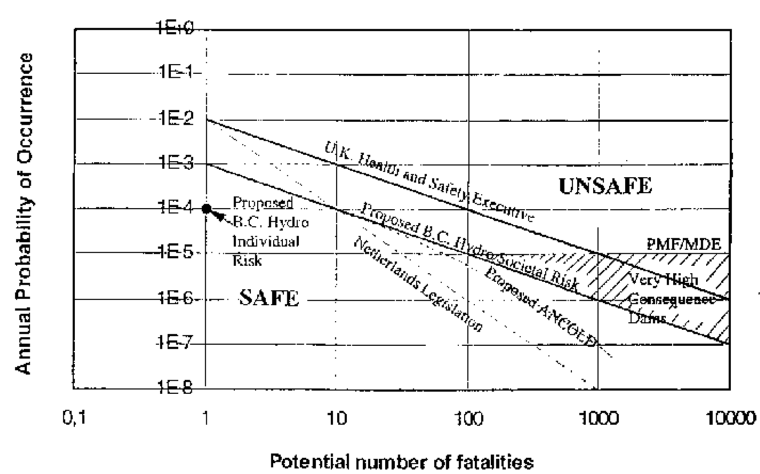


Fig. 18 Risk criteria chart proposed by B.C. Hydro (Vick and Stewart, 1996)

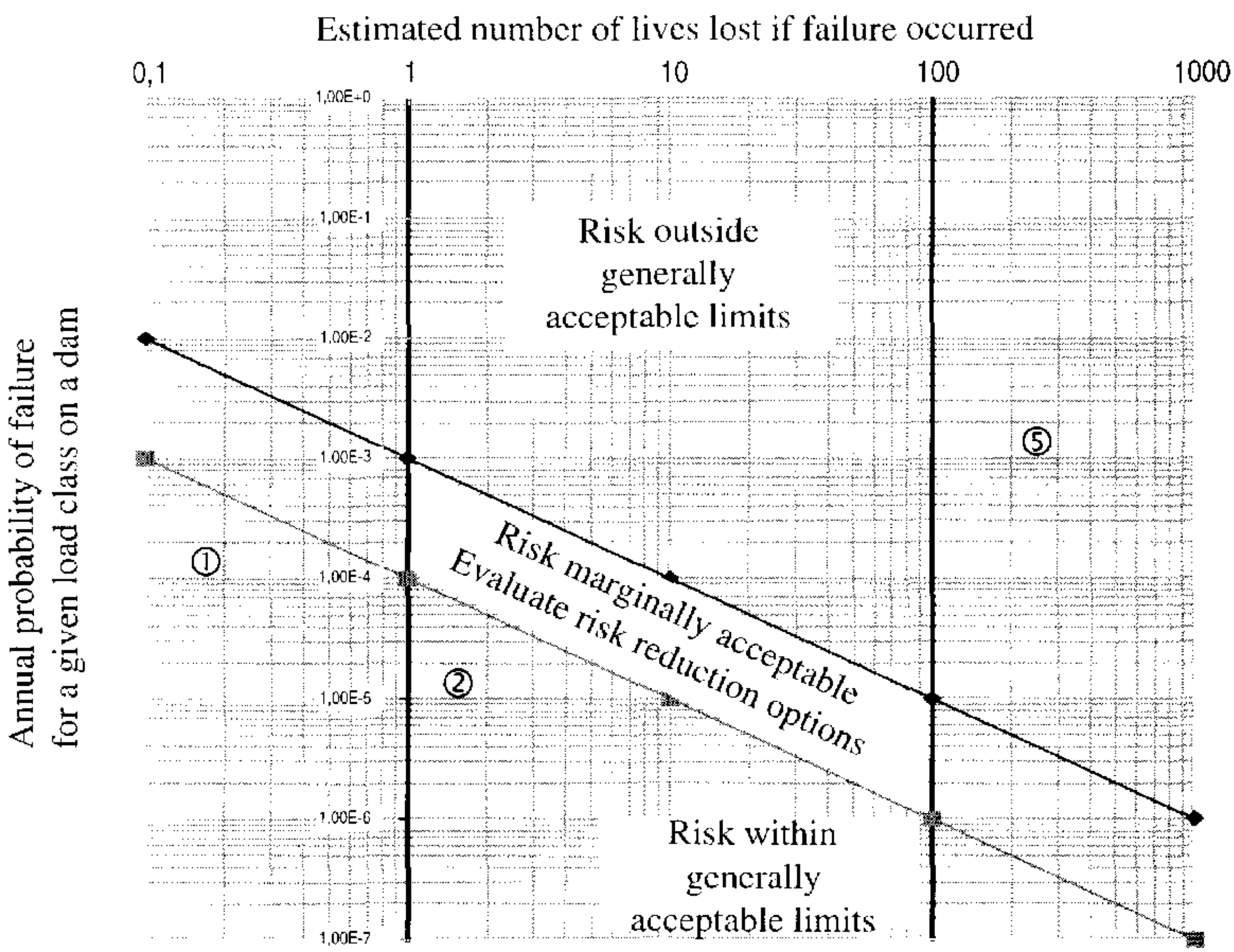
Risk analysis can be considered as an approach to establish a diagnostic. The procedure, or some of its steps, provides a framework for the systematic use of engineering judgement in decision-making, when uncertainties are present. In geotechnical engineering, uncertainties will always be present because of the nature of the material we are dealing with and the fact that there will never be enough data that will remove all uncertainty.

This systematic approach is also a means of documenting that the different critical aspects of a problem have been considered where and how in the analysis engineering judgement has been used. Such documentation is essential today when quality assurance and quality control should be at the basis of our work (whether it is required by the client or not). It is the duty of the engineering profession to present and explain the uncertainties involved, and the conventional safety factor does not do that.

In a reliability approach, assumptions can be clearly separated and criteria for conservatism can be placed where they belong. The approach will indicate high probabilities of failure, which will have a sobering effect, because not all our design have been optimum. It is no use to hide behind a safety factor that is probably wrong, because it does not account properly for the uncertainties, nature has a way to catch with this «ostrich's head-in the-sand» attitude.

One of the important benefits of an reliability-based analysis lies in carrying out the analysis. This aspect could even be said to be more important than the actual result of the analysis, as it brings to light the most important issues in a design.

Reliability methods also brings together the professionals from different engineering speciality areas and creates a dialogue which has long been needed. Examples of this have been presented by Lacasse and Nadim (1994) and Høeg (1996.



① Low Consequence Level
- economic consideration generally govern, consider alternative means for life-loss reduction
② Risk generally accepted
③ Marginally acceptable risk
④ Risk outside generally acceptable limits
⑤ High Consequence Level
- use best available methods, multiple defence design, and maximum loading conditions

Fig. 19 Risk evaluation guideline proposed by USBR (quoted by Whitman, 1997)

A geotechnical structure can usually be made safer by spending more money. The real challenge, however, is to improve the reliability of the structure without spending more money. To do his, it is important to adapt the level of complexity of the analysis to the problem that needs to be solved and the additional expense that can be saved.

RECOMMENDATIONS

A single risk analysis format is not universally applicable to all issues in geotechnical engineering. There lies one of the strong points of the approach. Methods and procedures can be varied according to the type of the problem, failure modes and the nature and uncertainty of the conventional (deterministic) analysis, the purpose of the analysis and the needs the analysis is meant to fill. Differences in methods can be associated with differences in response, consequences or safety issues.

The analyses should be robust: they should withstand criticism and scrutiny. They should be credible, defensible, transparent and error proof. This requires good documentation.

Risk and reliability based analyses are established but they are really only prototypes. Much work still needs to be done. The approaches should not be oversold, but there is no doubt in the author's mind that the approach can provide additional information to the designer, which otherwise stays hidden in the deterministic analysis. The more critical this information is to the design, the more important it is to include them in the analysis with the appropriate degree of attention such that the consequence connected to each critical aspect is included in the analysis.

Risk and reliability based methods, while not a substitute for the conventional deterministic design analyses, offer a systematic and quantitative way of accounting for uncertainties. The approach is most effective when used to organise and quantify the uncertainties in engineering design and to help making decisions. They can be helpful for a wide range of problems, especially when there is not enough experience available. This recommendation was also reached by the committee on «reliability methods for risk mitigation in geotechnical engineering» (NRC, 1995)

There are some types of problems where doing reliability-based evaluations will not give adequate assistance: when the uncertainties are very large, when the mechanisms of the problem are not well understood, or when the parameters of analysis model are not well defined.

With respect to future implementations, they should concentrate on the practical application of reliability models to take advantage of the added knowledge the methods can give the designer in particular and profession in general. Fault trees could be used more often to look into the possible outcome of a design, It has the advantage of being easy to understand and having a flexible logic and it is very useful to map the understanding of a problem. The degree of confidence in the numbers used must be identified, as well as how they were derived (analysis, judgement opinion - e.g. informed or uninformed guess).

NEEDS AND TRENDS

A few words on future needs and trends, although it is clear that there may be a high probability of being proven wrong with this prediction. The expected trends are also certainty influenced by the fact that the authors strongly believe that reliability-based approaches have much to offer to help us in our designs.

Good progress has been done, convincing examples do exist. However, acceptance of the methods by taking them into practice is mainly concentrated in the offshore industry.

We need to continue working along the following axes:

- A fundamental precondition for improvement of risk assessments is the availability of databases with reliable data. It is therefore important to continue collecting data on performance, especially on deficient aspects of our designs. The purpose being to enable the engineering profession to gain more confidence in the subjective judgements that need to be done to evaluate coefficients of variation and probabilities. Quantifying more rigorously than today the model uncertainty for an analysis is also a crucial need, if we are to improve our analyses and our confidence in the results.

- We need to continue to discuss the risk acceptance level between practitioners, between countries, to gain an improved understanding of the differences that need to be made.

- We should encourage as much as possible the application of simple probabilistic and risk-based procedures. This will contribute to gradually have this way of thinking an integral part of our engineering skills. The approach is a good pedagogic instrument, and it will help a young engineer to develop his sensitivity for which event can occur and how likely they are.

- We should always be critical of the method; erroneous input will lead to erroneous results. Engineering imagination and judgement still are the most important contributors to the analysis for design, construction and operation of a structure. One should always evaluate the goodness and robustness of the risk analysis done and the results obtained.

- We should attempt to quantify further model uncertainty, hopefully with the help of well-documented databases of case studies.

- There is also the need to develop methods to check the results of the reliability-based analyses.

It takes years of experience to make a good geotechnical engineer. The same is true for all branches of engineering and mathematics. To achieve good results, one needs to assemble specialists, and exchange knowledge, vocabulary and experience, so that they can communicate and understand each other, the significance of a parameter and the ways their respective inputs are to be used in the analysis.

The existing terminology could be improved to facilitate communication between the geotechnical practising engineer and the one used to statistics and probability concepts. This could easily be done, it just needs that one person decides to do it.

Reliability approaches, if they are to become more widely accepted, will need to be based on well-recognised conventional (deterministic) approaches, and will have to ensure that uncertainty can be built into them.

It is hoped that the examples given and the discussion made will help convince the reader that risk and reliability approaches have the potential for wider applications in geotechnical engineering, and that the implementation of the methods should benefit the profession, both the consultant and the client.

In concluding, one should not always have recourse to reliability analyses, but the authors find that the approach fits in well with what R.B. Peck taught us back in 1962:

> «Indeed the conventional procedures now used to calculate bearing capacity, settlement, or factor of safety of a slope are valid and justified only to the extent that they have been verified by experience. The science of soil mechanics merely provides devices for interpolating among the specific experiences of many precedents in order to solve current problems which are recognized to fall within the limits of previous experiences. In addition, however, soil mechanics provides the means by which we can go beyond the limits of our own experience to that of others. It points the way to new solutions of old problems, or to the solution of previously unsolved problems. In this respect, soil mechanics is a means of extrapolating our experience. Of course, such extrapolation involves a measure of uncertainty until the pertinent experience becomes available.»

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