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Risk Assessment and Management for Geotechnical Design of Offshore Installations

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Abstract

More than before, society and standards require "risk-informed" decision-making. The paper demonstrates the benefits of implementing reliability concepts in offshore geotechnical design. Reliability-based approaches will assist preparing engineering recommendations and decision-making. The paper gives an overview of basic concepts of reliability-based design, discusses the advances of hazard, risk and reliability in geotechnical engineering and illustrates their use with case studies from offshore practice. Risk and reliability evaluations can vary from simple statistical evaluations to full probabilistic modelling of the hazards and consequences for a single or a system of offshore installations. The examples include pipeline siting, jack-up mobile units, piled foundations and the reliability of code requirements. The paper discusses the strengths of the reliability-based approach and key issues such as tolerable and acceptable risk and the selection of characteristic value. The paper concludes that reliability-based approaches offer a useful complement to deterministic analyses, and enable analyzing complex situations with uncertainties in a systematic and more complete manner than deterministic analyses alone, both for design and for reevaluation. Managing geotechnical risks should today become a natural part of the engineer's work.

Introduction

When doing a geotechnical design, the engineer always looks at the level of safety of his design. The engineer wants to know whether or not the foundation can fail to perform adequately under the applied loads. Many uncertainties affect geotechnical calculations, and the engineer needs to consider their effect on the performance. Terzaghi (1961) wrote:

> *"Soil engineering projects [...] require a vast amount of effort and labor securing only roughly approximate values for the physical constants that appear in the equations. The results of the computations are not more than working hypotheses, subject to confirmation or modification during construction. In the past, only two methods have been used for coping with the inevitable uncertainties:*

either adopt an excessively conservative factor of safety, or make assumptions in accordance with general, average experience. The first method is wasteful; the second is dangerous. ...

This paper summarizes part of the earlier work using reliability and risk concepts in an effort to more fully understand the benefits of using reliability-based concepts in the design of offshore installations, including the significance of level of safety, the influence of uncertainties on the computed factor of safety and the selection of characteristic value. The need for reliability-based approaches has risen because society and standards require "risk-informed" decision-making (ISO2394:2015) more than before. The paper also demonstrates that geotechnical engineers have the necessary skills to do risk-based analyses and reliability-based design.

The engineering literature (*e.g.*, Morgenstern 1995; Vick 1994) identifies three ways of estimating annual event probabilities:

- Based on the frequency calculated from observations (historical data);
- Derived from probability theory (reliability-based design with some mathematical modelling); and
- Using and quantifying, where possible, expert judgment (subjective probabilities).

Benjamin and Cornell (1970) stated that *"the sources of the probability [estimates] may include observed frequencies, deductions from mathematical models, and in addition, measures of an engineer's subjective degree of belief regarding the possible states of nature"*.

Uncertainties, their sources and ways to treat them statistically could be a paper in itself. The quantification of uncertainties is not covered herein. The reader can refer to several books and papers on this subject, including Ang and Tang (2007), Baecher and Christian (2003), Keaveny *et al* (1990). Lacasse and Nadim (1996), Lacasse *et al* (2017), Nadim (2015), Tang (1973; 1984; 1987) and Uzielli *et al* (2006). This paper concentrates on risk-based approaches and what they can bring to improve the design of offshore installations and illustrates their use with practical applications.

Quantification of Safety Margin

Silva, Lambe and Marr (2008) presented Figure 1 combining historical and subjective probabilities to establish an approximate correlation between safety factor and failure probability. The diagram was devised for use in engineering practice. The figure is an updated version of the ones presented by Lambe (1985) and Baecher and Christian (2003), and compiles data from over 75 projects spanning over 4 decades. The projects include zoned and homogeneous earth dams, tailings dams, natural and cut slopes and earth retaining structures. The annual probabilities of failure for the different case studies were quantified iteratively, through experience, engineering judgment and published (historical) statistics.

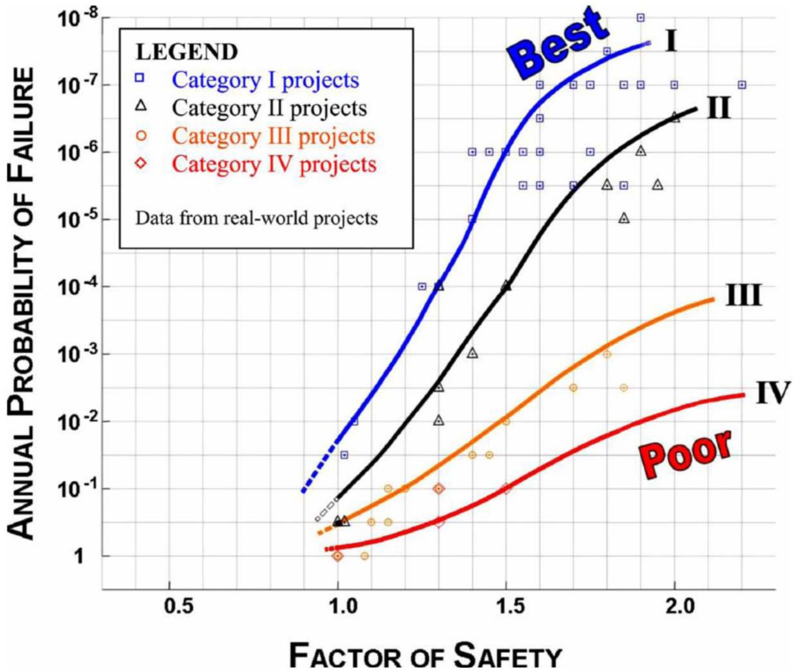


Figure 1—A practitioner's view of how factor of safety varies with uncertainty (Silva *et al* 2008)

The figure cannot be used to establish annual probabilities of failure in a design or verification situation, but it illustrates clearly the effect the uncertainties can have on the perceived factor of safety. Figure 1 classifies the structures into four categories, based on a judgment of the level of engineering. The level of engineering was established subjectively on the basis of the practices followed for design, investigation, testing, analysis, documentation, construction, operation and monitoring:

– Category I: facilities designed, built, and operated with state-of-the-practice engineering;
– Category II: facilities designed, built, and operated using standard engineering practice;
– Category III: facilities without site-specific design and sub-standard construction or operation;
– Category IV: facilities with little or no engineering.

The family of curves were anchored on two sets of coordinates: a factor of safety of 1.5 for an annual probability of failure of 10^{-4} based on the historical performance of earth dams designed and constructed with conservative engineering practice (Baecher *et al* 1980; Whitman 1984; Christian *et al* 1992); and a 50% annual probability of failure for a safety factor of unity, based on a normally distributed uncertainty in the factor of safety (Vick 1994).

The curves in Figure 1 reflect the concept in Figure 2: a design with a high factor of safety can have a higher probability of failure than another with a lower factor of safety. A larger factor of safety does not necessarily imply a smaller risk, because its effect can be negated by the presence of large uncertainties.

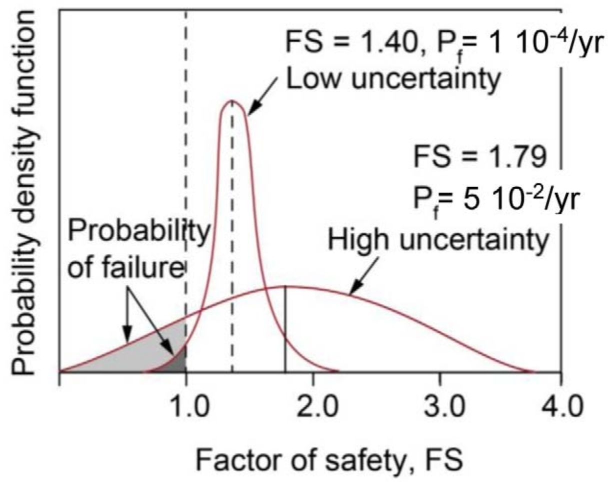


Figure 2—Factor of safety (*FS*) and probability of failure (P_f) for piled jacket at two periods in lifetime.

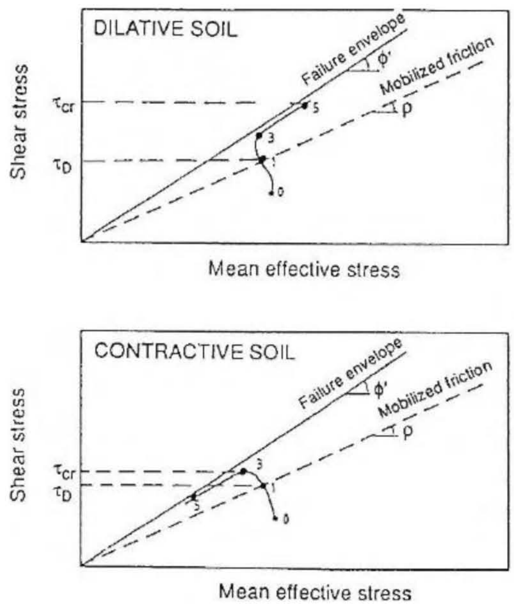
Figure 1 suggests that, in the view of three practitioners, a factor of safety of 1.5 can have an annual failure probability between 10^{-7} and 10^{-2} depending on the uncertainties, and a factor of safety of 1.3 an annual failure probability between 10^{-4} and 50%. The range of perceived failure probability for a "given" factor of safety is extremely wide. Notwithstanding Figure 1, there is no general relationship between factor of safety and probability of failure. Therefore, establishing such relationship should not be attempted.

Factor of safety is not a sufficient indicator of safety because it does not account for the uncertainties in the analyses. Duncan (2000) pointed out:

> "*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.*"

In offshore foundation design, the engineer tries to partly compensate for the uncertainties with characteristic values. However, the selection of characteristic value is still very subjective, engineer-dependent and unclear for many.

Soil properties and quality of engineering are not the only sources of uncertainty. The methods used to calculate the capacity of piles (Lacasse *et al* 2013a; 2013b; Liu *et al* 2019) or the bearing capacity of shallow foundations can themselves lead to significant uncertainties. Figure 3 gives an example where stability analyses were done with the effective stress (ESA) and the total stress (TSA) approaches on a contractive and a dilative soil. The first approach uses friction angle (ϕ'), cohesion and pore pressures (or the effective stress path), while the second uses undrained shear strength and in situ effective stresses (total and effective stress paths). The effective stress paths and the resulting safety factors and nominal probability of failure are illustrated on Figure 3. The computed failure probability differed significantly for the two approaches, although the factors of safety were identical for the dilative material. The differences in the factors of safety, *FS*, and in the nominal probability of failure, P_f, were due to the uncertainty both in the soil parameters and in the calculation methods.



Analysis	Soil behavior	*FS*	Nominal P_f
ESA	Contractive soil	1.9	$2.0 \cdot 10^{-5}$
TSA	Contractive soil	1.4	$2.5 \cdot 10^{-3}$
ESA	Dilative soil	1.4	$6.7 \cdot 10^{-3}$
TSA	Dilative soil	1.5	$2.0 \cdot 10^{-6}$

Notation
ESA Effective stress analysis
TSA Total stress analysis
FS Factor of safety
P_f Probability of failure

Figure 3—Difference between factor of safety and failure probability, two methods of analysis (Nadim *et al* 1993).

Concepts of Risk and Reliability

The terminology used in this paper is generally consistent with the recommendations of ISO Guide 73:2009 and ISSMGE (2004), where the following definitions can be found:

- *Danger (Threat):* phenomenon that could lead to damage.
- *Hazard:* probability that a danger (threat) occurs within a given period of time.
- *Exposure:* the circumstances of being exposed to a threat.
- *Vulnerability:* the degree of loss to a given element or set of elements affected by a hazard.
- *Risk:* measure of probability and severity of an adverse effect to life, health, property or environment.

In its simplest definition, risk is the product of the probability of an undesirable event occurring, or the product of the hazard and the consequences (a function of exposure and vulnerability) of that event. Risk is a measure of both the potential occurrence and impact of an undesirable event. Risk is most often expressed in terms of a combination of the consequences of an event (including changes in conditions or circumstances) and the associated likelihood of occurrence. ISO Guide 73:2009 defines risk as "*effect of uncertainty on objectives*". Although the ISO Guide 73:2009 definition may seem different, the two definitions both refer to the effect of the uncertainties on the likelihood of a failure or non-performance.

Deterministic and Probabilistic Analysis

The goal of a safety assessment is to demonstrate that the risk associated with the construction, operation or decommissioning of any facility is at an acceptable level. In this paper, the terms "deterministic" analysis and "probabilistic" analysis are used.

- A deterministic analysis looks at a deterministic problem, without taking the probabilities of different event sequences into account. A deterministic analysis aims at demonstrating that a facility is tolerant to identified faults/hazards within a "design basis". A deterministic analysis evaluates a "nominal" performance. The approach does not consider the full range of possible outcomes and does not quantify the likelihood of each of these outcomes. Consequently, deterministic scenario(s) may actually underestimate the potential risk.
- A probabilistic analysis aims at providing an estimate of the risk associated with a facility, and an estimate of the uncertainties involved. While a deterministic analysis considers the impact of a single

risk scenario, probabilistic analysis considers all possible scenarios, their likelihood and impacts. Probabilistic risk assessments also help understand and account for the uncertainties. In addition, discussing uncertainty (ies) will usually promote a debate that should lead to more robust decisions.

Reliability-Based Design

There are three approaches to design:

– The "Working stress" design (WSD) is the traditional approach based on an overall factor of safety, and has been used for a long time.
– Modern design codes are based on partial safety factors (or coefficients): the LRFD approach (Load and Resistance Factor Design) in North America and the characteristic values and "partial safety factors" approach in Europe. The partial safety factor is used to reflect the level of uncertainty and/or the relative importance of a particular parameter in design.
– Reliability-based design (RBD) using a target annual failure probability or target reliability index to verify margin of safety.

Risk Assessment and Risk Management

Risk Management. Risk management is the process of identifying, analyzing and assessing risks to enable informed decisions on accepting or treating and controlling risks to minimize them. Risk management integrates the recognition and assessment of risk with the development of appropriate mitigation (risk treatment) strategies.

Risk management comprises six main tasks: (a) Danger or hazard identification; (b) Causal analysis of the dangers or hazards; (c) Consequence analysis, including vulnerability analysis; (d) Risk assessment combining hazard, consequence and uncertainty assessments; (e) Risk evaluation of whether the risk is acceptable or not; and (f) Risk treatment (or risk mitigation).

Risk management has been formalised into a framework by ISO 73:2009 (Fig. 4). It is an integrated process involving communication and consultation on the one hand, and monitoring and review on the other hand, each imbedded in the risk framework. The process systemizes the knowledge and uncertainties, to evaluate the significance of risk and for comparing options.

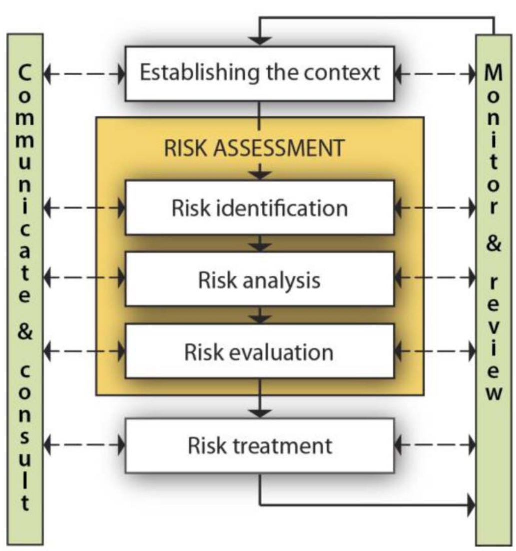


Figure 4—Risk assessment and risk management process in ISO Guide 73:2009.

Methods for Risk Assessment. There are several methods to do risk assessment, from simple qualitative risk matrices to more advanced numerical tools. Lacasse and Nadim (2007) summarized many of the methods in detail. The methods are only briefly mentioned in this paper.

Qualitative methods

The most common tool is the "traffic-light" matrix (Fig. 5). The qualitative matrices can be very useful especially when assessed through the consensus of several individuals with different expertise. Over the years, the 5x5 matrix has gained more popularity than the original 3x3 matrix.

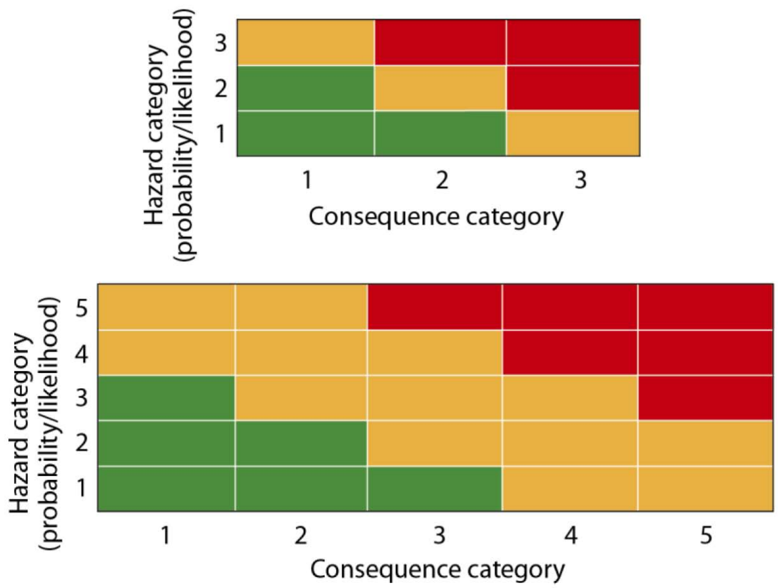


Figure 5—Qualitative risk assessment with 3x3 and 5x5 matrix: Hazard categories 1 to 3 or 1 to 5 for Very Low to Very High hazards; Consequence categories 1 to 3 or 1 to 5 for Very Small to Very Large consequences.

In such matrix, green designates "Low risk", red designates "High risk" and orange designates a situation in between, "Medium risk". Such qualitative estimates are useful as a first pass tool to establish whether or not more detailed analyses are needed if some of the scenarios considered in the analysis fall in the red or orange zones. When establishing such matrix, implemented *e.g.* in a macro-operated Excel sheet (Langford *et al* 2019), to define unambiguously and consistently the significance of 'very low', 'low', 'medium', 'high' and 'very high' hazards and the impact of 'very small', 'small', 'medium', 'large' and 'very large' consequence.

Other qualitative methods, devised mainly for large dams and other critical facilities, include the Life Cycle Inventory and Life Cycle Assessment, also known as "Cradle to Grave" analysis (US EPA 2010) and the "Failure Modes and Effects Analysis" (FMEA) and the Failure Modes, Effects and Criticality Analysis (FMECA), described in USACE (2011), USACE (2014) and FEMA (2015).

Quantitative methods

In the quantitative probabilistic analysis, the same formulation (equation) as the deterministic calculation is used. The only difference with the probabilistic analysis is that the soil and load properties are described by a probability distribution function with a mean and standard deviation and that an additional variable is introduced, the method uncertainty. Quantitative methods include Event tree analysis, Fault tree analysis, Bayesian updating and the First Order Second Moment (FOSM) method. Somewhat more complex tools (most are software packages) include: Monte-Carlo simulations, Event tree analysis combined with Monte Carlo simulations; Bayesian networks, the First Order Reliability Method (FORM), the Second Order Reliability Method (SORM) and the methods for system reliability analysis, such as SYSREL (RCP GmbH 1999). It is not unusual to combine two or several reliability approaches to obtain reliability solutions. For example, a FORM analysis of a site-specific stability analysis combined with an event tree analysis covering all possible events in the lifetime of an installation.

Acceptable and Tolerable Risk

A challenging task in reliability-based design lies in the selection of risk acceptance criteria. Acceptable risk refers to the level of risk requiring no further reduction. It is the level of risk society desires to achieve. Tolerable risk refers to the risk level reached by compromise in order to gain certain benefits. A construction with a tolerable risk level requires no action/expenditure for risk reduction, but it is desirable to control and reduce the risk if the economic and/or technological means for doing so are available.

A Frequency-Consequence chart (or an F-N chart) is a practical way to present risk levels and evaluate them relative to guidelines or compare the risk levels of different facilities. The *F-N* curves relate the annual probability (*F*) of causing *N* or more fatalities to number of fatalities. The term "*N*" can be replaced by other measures of consequences, such as costs. Figure 6 presents a slightly modified version of the first Whitman (1984) *F-N* chart (Baecher and Christian 2003).

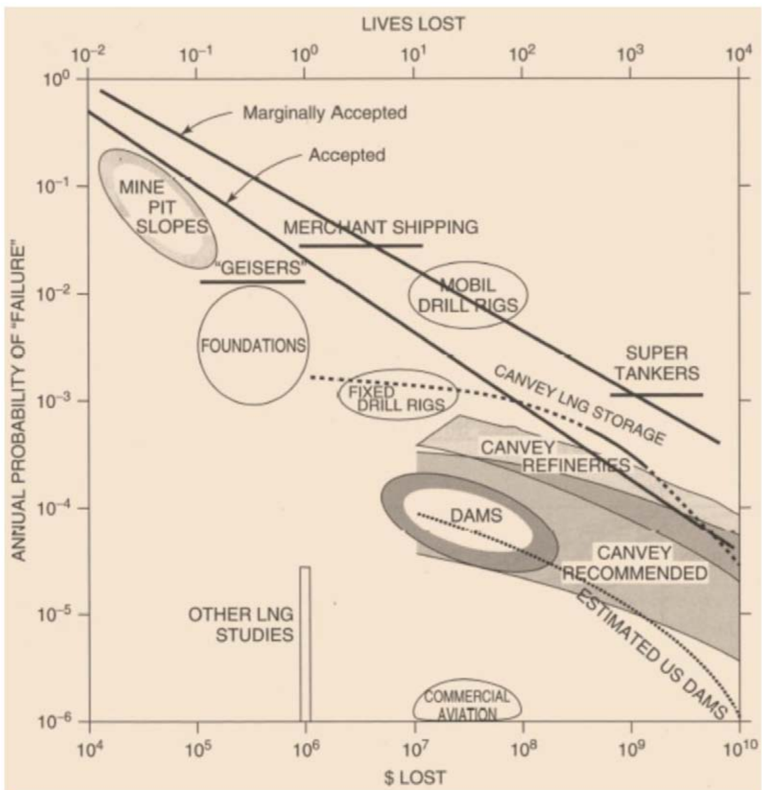


Figure 6—Adapted Whitman (1984) curves of acceptable and marginally acceptable levels of risk on F-N chart (Baecher and Christian 2003) (note: 1984 US dollars)

A number of guidelines have been suggested by several countries, some expressly for slopes or for dams, some more general (Fig. 7). Although there are differences, the acceptable/tolerable risk level centers around 10^{-4}/year for ten fatalities. Figure 8 presents the range of these guidelines and the Hong Kong criterion, which is one of the most frequently used. This would be good guideline for offshore structures and is more stringent than those in Figure 7. The boxed area to the right (fatalities>1000) requires detailed assessment and reflects risk aversion in the case of a high number of fatalities due to a single event.

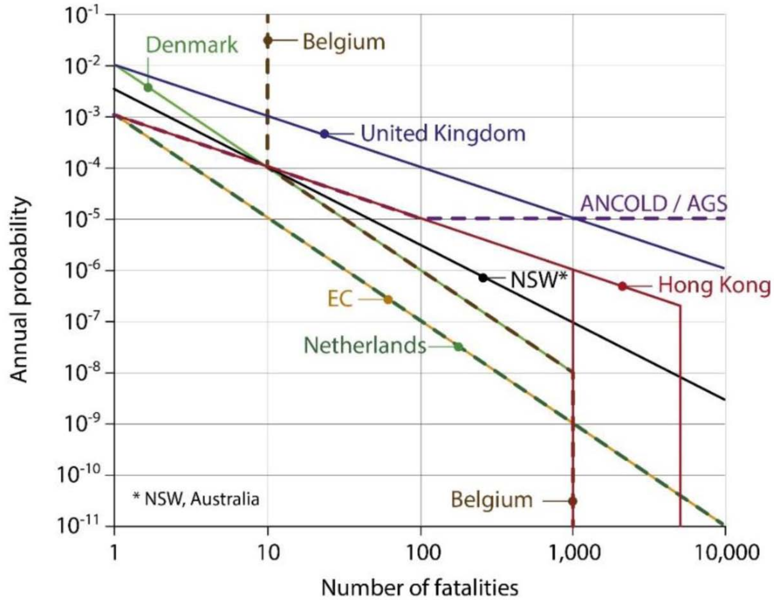


Figure 7—Overview of F-N chart risk guidelines in different countries (K.Ho Pers. Communication. Hong Kong Nov. 2008).

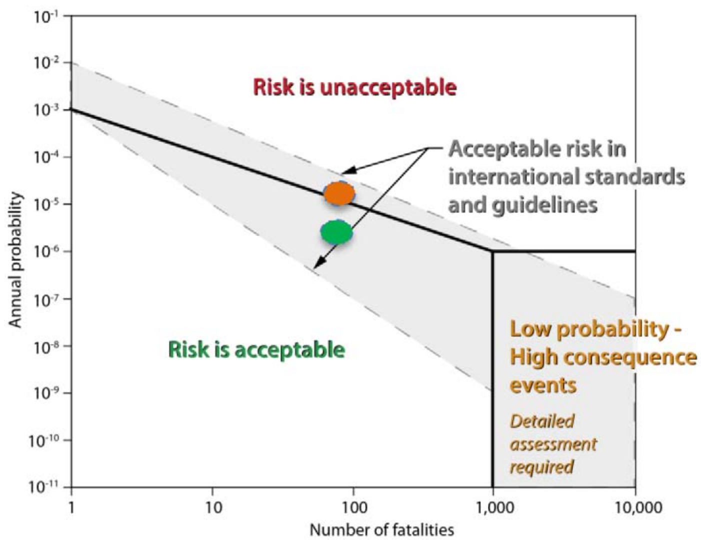


Figure 8—Range of risk guidelines and recommended Hong Kong guidelines.

The demarcation between acceptable and unacceptable risk should be seen as a gradual transition rather than punctual values, as illustrated with the shaded zones in Figure 9 from the US Bureau of Reclamation (2011). One can also define an *ALARP* zone on the F-N chart, or limits where the risk level should be kept "As Low As Reasonably Practicable". The ALARP zone describes an acceptable level of risk that cannot be reduced further without efforts and cost being disproportionate to the benefit gained or where the solution is impractical to implement.

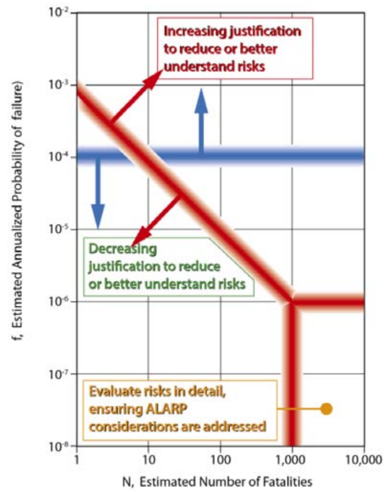


Figure 9—US Bureau of Reclamation (2011) risk guidelines.

An annual failure probability of 10^{-4} has an actual significance. Figure 10 shows the mortality rate in Canada, due to all causes, as a function of age. At age 5-10, the probability of dying in the next year is 1/10,000 or 10^{-4}. At age 40, the probability increases to 1‰, at age 65 the probability is 1% and at age 90, the probability of dying in the next year is close to 10% (as expected).

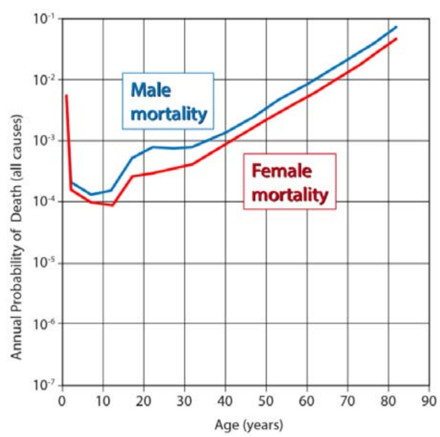


Figure 10—Annual death probability as a function of age.

Deterministic or Probabilistic Safety Target?

For offshore installations and large dams, there are often discussions of the safety target to achieve, and whether the safety target should remain the same during the entire life. Yes, the safety target should remain the same. The individuals on an offshore platform should not be exposed to a higher risk, and any potential environmental damage should not increase with time. Everyone would agree with this.

However, what should the safety margin to achieve be during the entire lifetime of an offshore installation? Is a fixed deterministic safety factor sufficient to represent the safety margin throughout the lifetime of an installation? Good practice can be reflected in the same safety margin, but not necessarily in the same calculated safety factor, because the uncertainties do not remain the same during the lifetime of an installation. A target annual reliability index or an annual failure probability, on the other hand, allows a fairer comparison of the safety margin at different times of the life of an offshore installation. The reliability-based approach, with probabilistic analysis, can account for all observations and experiences during the course of operation.

An offshore installation in operation for *e.g.* 30 years, represents 30 years of evaluated experience, not unlike a prototype test on site for 30 years. In most cases, the uncertainties at the time of design and construction will have reduced with time as more information and data become available. A hypothetical reduction of the failure probability with time for the same consequence is illustrated with the two circles in Figure 8. It is not possible to show such evolution with a fixed factor of safety target, and the safety margin would be underestimated.

Case Study: Siting of Drill Centres and Pipeline

Case Study. The Shell Stones development in the Gulf of Mexico in 2900 m water depth (Choi *et al* 2017) is located below the Sigsbee Escarpment where extensive slumping and debris flow activity has occurred in the past. The development comprises two drill centres (DCs) to produce hydrocarbons.

Questions to Answer. The design required an assessment of the risk posed by debris flows from the escarpment to the two planned drill centres. Is it safe to place the pipeline at the bottom of the slope?

To assess the annual probability (or hazard) of a debris flow reaching one of the locations of interest, the safety assessment was split in two (Fig. 11): (1) what is the likelihood of a slide on the escarpment; and (2) what is the likelihood of the debris reaching one of the drill centres on the seabed?

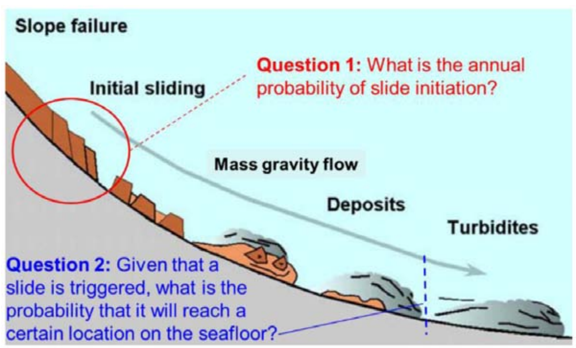


Figure 11—Schematic representation of submarine landslide evolution (Nadim *et al* 2014)

Applied Methodology. A pipeline, or any other linear infrastructure such as roads and railways, often extend over tens, hundreds or even thousands of kilometers. For these extents it is not feasible or economically viable to do detailed investigations of the risk posed by all geohazards. A three-step approach (Nadim and Lacasse 2004), with increasingly quantifiable results, can be used to assess the risk to identify the slopes that could impact the pipeline.

- Step 1 - Global identification of landslide hazard: one can identify the areas prone to sliding activity over long distances from publicly available data (www.proventionconsortium.org). The relative landslide hazard level can be determined based on the nature of a slope, lithology, soil characteristics and, for example, a seismic trigger indicator. The resulting relative hazard level categorizes locations with significant landslide hazard potential.
- Step 2 - Regional zonation of landslide hazard: the most critical slide prone areas can be more detailed qualitatively and calibrated with local experience or historical records. A qualitative risk matrix (Fig. 5) can be established (*e.g.* Nadim and Lacasse, 2004).
- Step 3 - Site specific evaluation of landslide hazard: the slopes classified in the "high" hazard class are investigated in greater detail, usually with a calculation model to verify stability.

The annual probability of damage to the drill centres is the product of the probability of a slide occurring and the probability of the debris reaching the drill centres. The annual probability was estimated from the number of debris flow deposits on the seafloor using a simplified Bayesian model (Nadim *et al* 2014).

To answer Question 2 in Figure 11, Monte Carlo simulations of the debris runout evaluated the probability of runout reaching the drill centres. Figure 12 illustrates one of the slope models, using a pseudo-2D debris flow model (i.e. models the runout along a slice) that had been calibrated against pseudo-3D simulations and evidence from relict debris flows.

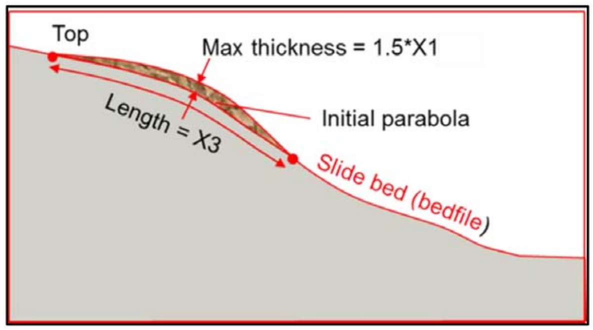


Figure 12—Initial slide on North Escarpment in runout simulations, Shell Stones project (Choi *et al* 2017)

Results of Reliability Analyses. Figure 13 (top) shows the maximum calculated runout distances from the Monte Carlo simulations for cross-sections P19 and P20. The *x*-axis is the location of the final runout distance with respect to the toe of a slope section. To simulate the effect of the method uncertainty, a random error with a standard deviation of ±37m added to the calculated runouts.

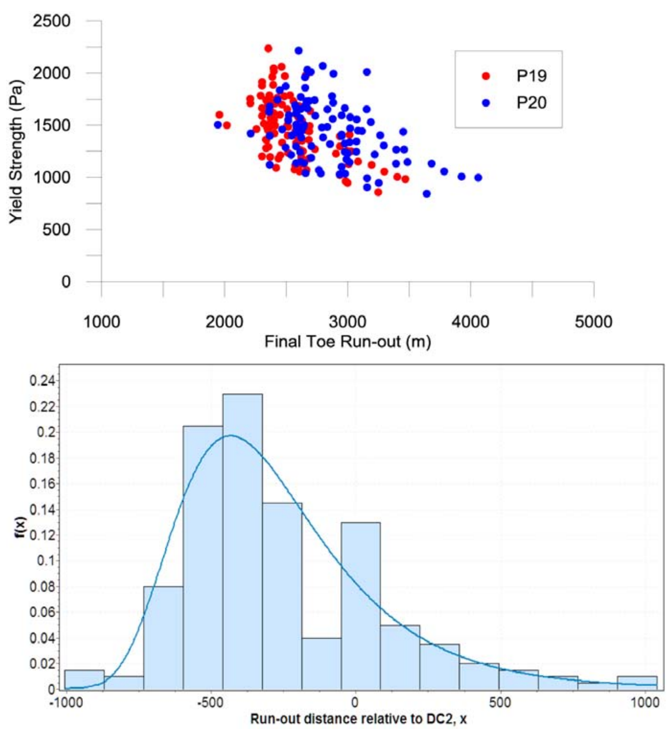


Figure 13—Probabilistic distributions of runout distance (Choi et al 2017).Top: Calculated final runout distance, cross-sections P19 and P20.Bottom: Histogram of calculated final runout distance and best-fit probability distribution.

Figure 13 (bottom) shows the histogram of the maximum runout distance difference in m with respect to drill centre 2 (DC2 in Fig. 14), as well as the distribution function that provided the best fit to the results. Figure 14 shows the contours of the probability of a debris flow reaching drill centre DC2.

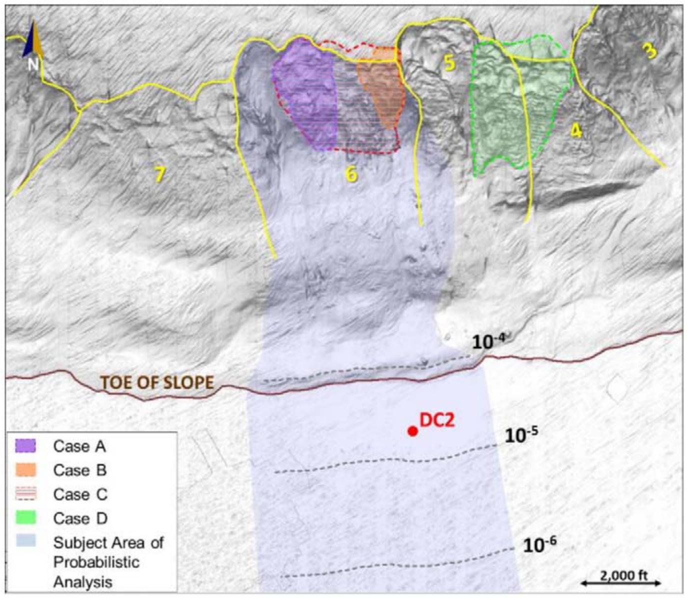


Figure 14—Contour of annual probability of slope instability leading to runout reaching DC2 (Choi *et al* 2017).

The results of this assessment estimated the annual probability of debris flow impact on seabed installations at DC2 to be about 2η10^{-5}. This probability does not represent the probability of damage to subsea installations, which was characterized by further study and is significantly lower than 10^{-5}/year.

Probabilistic Complement to Deterministic Analysis. The probabilistic analyses allowed to estimate the likelihood of a debris flow at the location of the drill centres based on historical landslides at the location.

The combined deterministic and probabilistic analyses established the most probable and the minimum and maximum runout distance, should a debris flow occur. The uncertainty in the probabilistic runout distance was also quantified (by its standard deviation) to help quantify the danger of damage to the drill centres on the seafloor. The results of the analysis provided sufficient information to help making a decision on the safety of the installations on the seafloor.

Case Study: Reliability of Codes of Practice for Offshore Foundations

Case Study. Two shallow foundations in the Gulf of Mexico were studied: the first a vertically loaded well manifold on normally consolidated highly plastic clay on a soft clay (Fig. 15), the second a subsea isolation valve with inclined load on medium dense sand (Fig. 16) (Liu *et al* 2015a). The loads and soil characteristics are also shown. For the well manifold, the vertical load is due to the weight of the manifold and jumpers. Maximum load occurs during the first year. For the subsea isolation valve, the vertical load is due to the weight of the valve. The horizontal load is a short-term, extreme load due to winds, waves and currents. Maximum environmental load can occur at any time during the 30-yr design life.

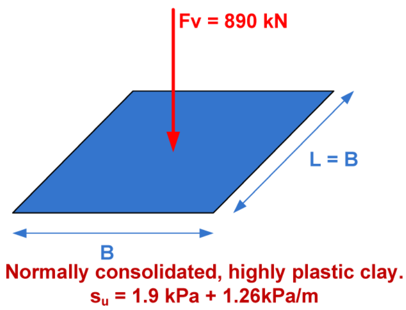


Figure 15—Vertically loaded well manifold on normally consolidated highly plastic clay (Liu et al 2015a).

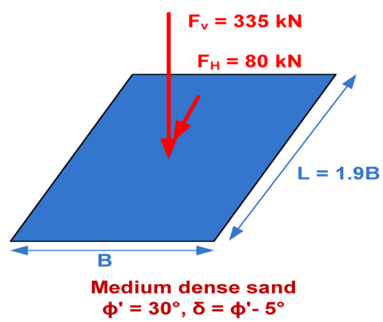


Figure 16—Subsea isolation valve with inclined load on medium dense sand (Liu et al 2015a).

Questions to Answer. What is the probability of failure implied by the bearing capacity and sliding equations in the API RP 2GEO and the API RP 2A LRFD 1st edition guidelines and the ISO 19901-4 standard? Are the requirements equivalent with respect to margin of safety? The required safety factor, resistance factor and material coefficient are given in Table 1.

Table 1—Required safety factors for API RP 2GEO, API RP 2A LRFD and ISO 19901-4.

Guideline/standard	Type of safety factor	*Requirement*
API RP 2GEO	Global, *FS*	*FS* = 2.0 (bearing)
		FS = 1.5 (sliding)
API RP 2A LRFD	Resistance factor, ϕ	ϕ = 0.67 (bearing)
		ϕ = 0.8 (sliding)
ISO 19901-4	Resistance coefficient on soil property, γ_m	γ_m= 1.5 (bearing, undrained)
		γ_m = 1.25 (bearing, drained)
		□$_m$ = 1.25 (sliding)

Applied Methodology. To assess whether or not the required safety factors in the three guidelines result in a consistent margin of safety (or level of reliability), three probabilistic models were used: (1) the first-order second moment (FOSM) approximation, (2) the first order reliability method (FORM) and (3) Monte Carlo simulations (MC). Lacasse *et al* (2017) described in detail the reliability methods and the formulations in each guideline and standard, and listed the input parameters for the soil and loads and the uncertainties thereof. To ensure that the Monte Carlo simulations gave reliable results, five (5) millions simulations were conducted. For method uncertainty, several studies have been conducted to quantify the method uncertainty in the undrained bearing capacity of a shallow foundation (e.g. Nadim and Lacasse 1992, Forrest and Orr 2011). A COV of 0.15 was used for the present analyses.

Results of Reliability Analyses. The reliability analyses compared the probability of failure obtained with the FOSM, FORM and MC approaches for the prescribed design factors in Table 1.

Undrained Bearing Capacity on Clay:

The three guidelines gave similar nominal probabilities of failure. The probability of failure for the API RP 2GEO guideline (factor of safety of 2), the API RP 2A LRFD (resistance factor of 0.67) and the ISO 19901-4 standard (material coefficient of 1.5) all showed a nominal failure probability of about 10^{-5}. The results of the FOSM, FORM and MC approaches were essentially identical. This is due to the equations for calculating bearing capacity (limit state function) are close to linear.

Drained Bearing Capacity on Sand:

Figures 17 to 19 present the results, in terms of the calculated failure probability over 30-yr design life on the horizon show the results as function of the safety parameter. The prescribed safety parameter for each guideline is indicated by horizontal lines in the graphs:

- The FORM and MC results gave similar results. The failure probability for the API RP 2GEO guideline (FS = 2) and the API RP 2A LRFD guidelines (ϕ = 0.67) was close to $5 \cdot 10^{-4}$. The ISO 19901-4 standard with a material coefficient of 1.25 resulted in a failure probability of $1.4 \cdot 10^{-3}$, or a higher probability of failure by a factor of three to four.
- The FOSM, however, gave much higher failure probabilities than the FORM and MC results. The linearized limit state function around its mean point in the FOSM formulation is the explanation for the difference. The results seem to converge, the closer one gets to failure.

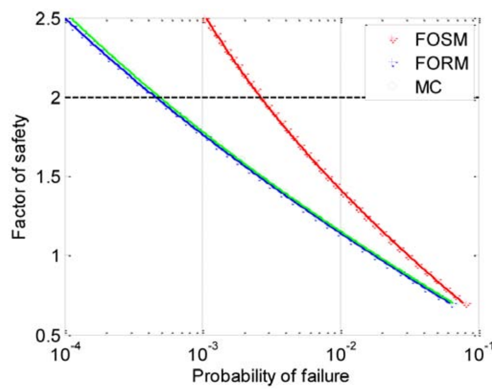


Figure 17—Bearing capacity failure probability for API RP GEO, drained conditions on sand.

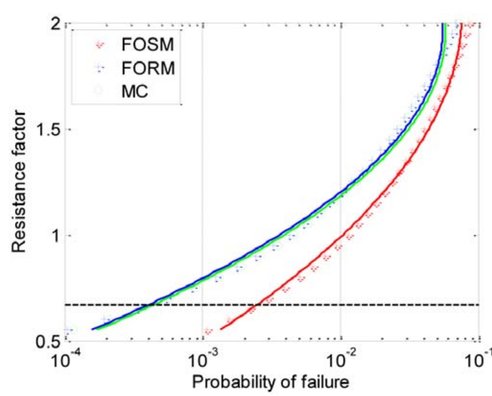


Figure 18—Bearing capacity failure probability for API RP GEO-LRFD, drained conditions on sand.

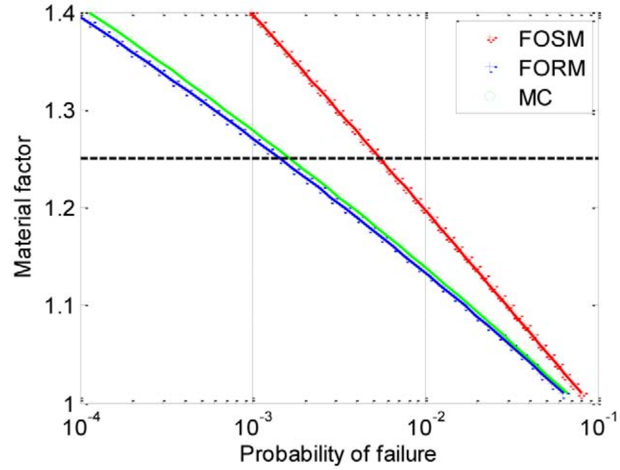


Figure 19—Bearing capacity failure probability for ISO 19901-4, drained conditions on sand.

Effect of Uncertainty in Soil Parameters. The effect of the uncertainties in the soil parameters was studied for the drained bearing capacity and sliding failure modes on medium dense sand.

Bearing Capacity on sand

Values of the coefficient of variation (COV) were first set to 15% and then to 25%. Figure 20 compares the probability of failure obtained by the FORM approach for each of the prescribed safety factor in the three guidelines. The higher uncertainty by 10 percentage point increases significantly the calculated failure probability.

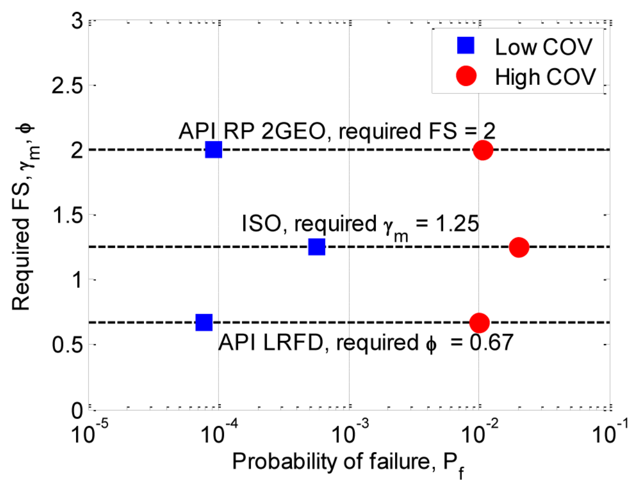


Figure 20—Effect of COV on failure probability for subsea isolation valve with inclined load on medium dense sand - Drained bearing capacity, three offshore design guidelines.

Sliding on Sand:

Similar analyses were run for the subsea isolation valve on medium dense sand under sliding. Figure 21 illustrates the results obtained with the SORM approach, including the effect of the uncertainties in the soil parameters. The calculated failure probabilities vary significantly for each prescribed safer factors in the three guidelines. They also increase consistently and significantly with a 10 percentage point increase in the uncertainty in the soil parameters (COV from 15 to 25%).

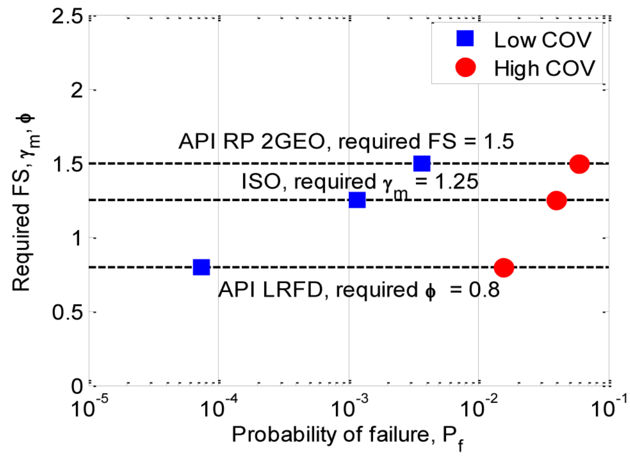


Figure 21—Effect of COV on failure probability for subsea isolation valve with inclined load on medium dense sand - Drained sliding, three offshore design guidelines.

Effect of Including Spatial averaging in the Analysis. Lacasse *et al* (2017) did a study of the effect of including the spatial averaging of the undrained shear strength in the vertical and horizontal directions, again comparing the three guidelines for practice. The bearing capacity of the shallow foundation in Figure 15, but this time on a highly plastic clay, was modelled with both the first order and second order reliability methods (FROM and SORM), the second-order method having one degree higher sophistication than the first-order method. Accounting for spatial averaging, resulted in a reduction of the failure probability by a factor of 4 for the API RP 2GEO guideline, a factor of 8 for the API RP 2A LRFD guideline and a factor of 2 for the ISO 19901-4 standard (see also Liu *et al* 2015b).

Probabilistic Complement to Deterministic Analysis. The probabilistic comparison of the three design requirements illustrated that, for both clay and sand, the margin of safety achieved with current practice varies. The results indicated that calculations with the three guidelines would result in different foundation sizes for the same reliability level.

The FORM and MC approaches gave similar reliability level for both clay and sand, and are more reliable tools for probabilistic design than the FOSM approach. The FOSM approach will however give a reliability level similar to that from FORM and MC analyses for the case where the limit state function (formulation of failure state) is close to linear.

Not accounting for spatial averaging of a soil property leads to slightly conservative design. It is however difficult, in most offshore cases, to have enough geotechnical data to be able to quantify the scale of fluctuation, at least horizontally, and therefore do a site-specific spatial averaging calculation.

Case Study: Piled Foundations – Single Pile

Case Studies. Three piled jackets in the North Sea were analyzed. At the site of Jacket A, the soil conditions were characterized by mainly clay layers with intermittent sand and silt layers and a very hard sand layer at 88 m. For Jacket B, the soil consisted of mainly dense to very dense sand layers. At the location of Jacket C, the soil profile consisted of alternating very dense sand and very stiff clay units. Lacasse *et al* (2013a;b) described in detail the sites and soil properties at the three sites. Table 2 summarizes the pile geometry, the water depths and the loading conditions for the three jackets. The 100-yr environmental load for most heavily-loaded pile group for each of the jackets.

Table 2—Study of axial pile capacity of single piles - description of three jackets analysed.

Jacket	A (clay)	B (sand)	C (layered sand and clay)
Water depth	119 m	92 m	≈100 m
# pile groups	4	4	4
# piles/group	4	6	6
Pile diameter	96"(2.438m)	96"(2.438m)	96"(2.438m)
Pile wall thickness	90 mm	100 mm	100 mm
Static load (including pile weight)	116 MN	221 MN	216 MN
100-yr environmental loads	102 MN	97 MN	114 MN

Questions to Answer. The API RP 2GEO (2011) and ISO 19902 (2007) guidelines include four CPT-based methods, namely the Fugro, ICP, NGI, and UWA methods, for calculating the axial capacity of piles in sands. The design guidelines require that if newer methods are to be implemented in design, the same level of safety shall be documented for new methods as for existing methods. During design, the deterministic calculations of the axial capacity for Jacket A resulted in very long piles, which were very difficult to hammer in place. The deterministic API and CPT-based design methods gave significantly different pile lengths. What is the margin of safety? What resistance factor should one use with each of the CPT-based methods?

Applied Methodology. In a deterministic design, the load and resistance factors are applied as follows:

$$[\gamma_{l\ stat} \bullet P_{stat} + \gamma_{l\ env} \bullet P_{env}^{100\text{-}yr}] = Q_{ult}/\gamma_m \qquad \text{(Eq. 1)}$$

where $\gamma_{l\ stat}$ = Load factor on static load
P_{stat} = Characteristic static load
$\gamma_{l\ env}$ = Load factor on environmental load
$P_{env}^{100\text{-}yr}$ = Characteristic environmental load (typically the load with 100-yr return period)
Q_{ult} = Deterministic ultimate axial pile capacity
γ_m = Resistance factor

The probabilistic analysis was done with the same equation, only this time expressed probabilistically and including the uncertainties in the parameters in Eq. 1. The analysis included:

- Statistical evaluation of the soil and load parameters;
- Statistical analysis of the analysis method uncertainty for each pile capacity method;
- Deterministic analyses of the axial pile capacity with the different pile capacity methods;
- Probabilistic analyses of the annual failure probability under axial loading;
- Calculate the required resistance factor for a target annual failure probability P_f of 10^{-4}.

The capacity of the most heavily loaded pile was analyzed. The procedures for estimating the mean and standard deviation for independent and dependent soil variables used the procedures in DNV (2012). The probabilistic calculation procedure was described in Lacasse *et al* (2013a;b). The pile capacity in compression and tension was obtained with an in-house design tool (NGI, 2011). The annual probability of pile foundation failure P_f and reliability index, β, were estimated with a second in-house software (NGI 1998). The results were combined with the statistics of the maximum annual axial pile load to calculate the annual probability of failure with the software COMREL (RCP GmbH 1999).

Figure 22 is a simplification in 2D of the overlap of the probabilistic ultimate pile capacity (Q_{ult}) and probabilistic environmental load (P_{env}). An extended study of the method uncertainty for each axial pile capacity calculation method was carried out at the time, but that work has now been superseded with the method uncertainty data derived from the new "Unified Pile Load Test Database". Updated method uncertainties were presented in Lehane *et al* (2017) and Liu *et al* (2019).

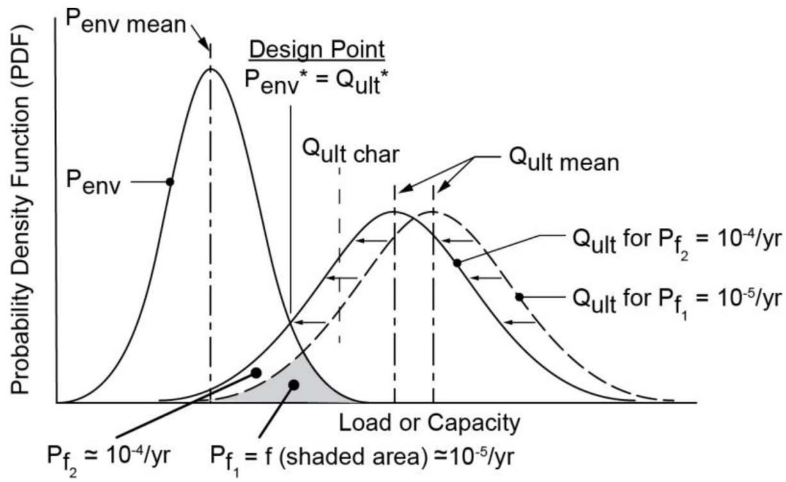


Figure 22—2D simplification of the calculation of the load and resistance got annual P_f of 10^{-4} and 10^{-5} (Lacasse *et al* 2013a).

Results of Reliability Analyses. For axial pile capacity, Tables 3 and 4 present some of the results of the probabilistic analyses for Jacket A with the API method and three CPT-based methods. Similar results were obtained for Jacket B and C. Table 4 gives the annual failure probability as a function of pile penetration depth. The mean and characteristic values were used in the analyses, and the pile capacity was modelled with a normal and a lognormal probability distribution. For the example results in Table 4, the NGI-05 CPT-based pile design method was used.

Table 3—Probabilistic analyses of axial capacity in compression, Jacket A with 90-m pile.

Method	Mean ± SD of Q_{ult} (COV)	Deterministic capacity*(MN)	Annual P_f and β
API	152 ± 31 (21%)	139	P_f= 2.0η10^{-7}, β = 5.0
NGI-05	121 ± 20 (17%)	119	P_f= 1.3η10^{-6}, β = 4.7
ICP-05	137 ± 25 (19%)	137	P_f = 3.9η10^{-7}, β = 4.9
Fugro	115 ± 31 (27%)	136	P_f = 2.4η10^{-4}, β = 3.5

Calculations done with mean soil parameters

Q_{ult} = Ultimate axial pile capacity

SD = Standard deviation

P_f = Probability of failure

β = Reliability index

Table 4—Annual failure probability as a function of pile penetration depth, NGI-05 method, Jacket A.

Penetr. depth (m)	$Q_{ult\ mean}$ (MN)	$Q_{ult\ char}$ (MN)	$P_{f\ annual}$ FORM, Q_{LN}	$P_{f\ annual}$ FORM, Q_N
75	96.6	77.8	$2.1 \cdot 10^{-5}$	$5.2 \cdot 10^{-5}$
80	103.9	84.1	$2.3 \cdot 10^{-5}$	$2.3 \cdot 10^{-5}$
90	118.6	96.8	$1.2 \cdot 10^{-6}$	$1.3 \cdot 10^{-5}$

Calculated with mean soil parameters

$Q_{ult\ char}$ = Ultimate axial pile capacity calculated with characteristic value

$Q_{ult\ mean}$ = Ultimate axial pile capacity calculated with mean value

N = Normal distribution

LN = Lognormal distribution

The results of the probabilistic analyses indicated that (1) the deterministic capacity calculated with the mean values of the soil parameters were in general quite close to the probabilistic mean for each of the different methods; (2) the method uncertainty for each of the calculation methods, both on the skin friction and the end bearing, was by far the most important contributor to the uncertainty in the capacity; and (3) the probabilistic distribution of the axial pile capacity was best modelled with a lognormal distribution.

With respect to the required resistance factor for an annual target P_f of 10^{-4}, Table 5 presents the calculated required resistance factor in compression for Case Studies A, B and C for a target annual failure probability of 10^{-4}. The resistance factor was obtained based on the axial pile capacity calculated with the characteristic undrained shear strength ($Q_{ult\ char}$). The load factor obtained at the design point in the probabilistic analysis was less than 1.3. In design however, the resistance factor should be associated with a load factor of 1.3 on the 100-yr environmental load. The calculations are from 2013, and used the now superseded method uncertainties (Lacasse *et al* 2013a; Liu *et al* 2019).

Table 5—Required resistance factor, #m, related to characteristic Qult ($Q_{ult\ char}$) for target Pf<10-4/yr.

Method	Site A (clay), 90-m pile ($\gamma_{l\ env}$=1.3)	Site B (sand), 26-m pile ($\gamma_{l\ env}$=1.35)	Site C (clay and sand), 40-m pile (γ_{lenv}=1.35)
NGI	1.23	1.35	1.20
ICP	1.52	1.45	1.32
Fugro	1.31	1.72	1.55
API	1.35	2.36	1.93

The calculated resistance factors vary with the pile design method. The resistance factors reflect the varying influence of the uncertainty in the soil parameters and of method uncertainty. The resistance factors in Table 5 apply to those sites only, and cannot be transferred to other sites without site-specific reliability studies.

Table 6 compares the required pile penetration depths for loading in compression for a target annual failure probability of 10^{-4} with the pile penetration depth calculated by the original deterministic analysis. The NGI-05 pile design method was used. The first number in the table is the penetration depth from the deterministic analysis with a resistance factor of 1.5 on the CPT-based methods. The second number is the penetration depth ensuring that the annual probability of failure was less than 10^{-4}. The significant reduction in the required pile penetration depth was possible because one could demonstrate that the annual probability failure was less than the target P_f of 10^{-4}/year. It was then possible to use a resistance factor of 1.3, as for the current API method, instead of the *a priori* resistance factor of 1.5 set for the newer CPT-based design methods.

Table 6—Pile penetration depth for design in compression for target Pf<10-4/yr (NGI-05 method).

Jacket	Site A (clay)	Site B (sand)	Site C (clay and sand)
Pile penetration depth			
-Deterministic design	90 m	51 m	45 m
-Probabilistic design	75 m	27 m	38 m

The resistance factors and the annual reliability indices and annual probabilities of failure computed are specific for each of the sites studied, and cannot be directly transferred to other sites. The reliability methodology is however general.

Probabilistic Complement to Deterministic Analysis. Ensuring adequate reliability under severe loading is a necessary consideration in design, and the calculated safety margin depends on the uncertainty in the parameters used in the analyses and the method uncertainty. The reliability study gave insight in the required resistance factor for different design methods of axial pile capacity to achieve a target margin of safety. The results depend on the method uncertainty. The selection of the characteristic value was also a key parameter influencing the required resistance factor.

Reliability methods provide a basis for the rational and systematic treatment of the uncertainties in offshore environmental loadings, geotechnical parameters and calculation methods, and can document the safety margin implied by deterministic safety factors. The approach helps ensure a more uniform safety margin for offshore installations.

Reliability analysis is a powerful tool to develop cost-effective design. The approach is general and presents the alternative of designing according to an annual probability of failure and the required resistance factor to achieve this level of reliability. The target annual probability of failure of 10^{-4} is only an example, other targets could have been used. In the above case studies, the pile penetration could be greatly reduced with the reliability design, even with a target annual failure probability of 10^{-5}. The finding has important implications for the design of piles and can often result in significant savings.

The analyses suggested that the CPT-methods are as reliable or more reliable as the current API method.

Case Study: Piled Foundations – System analysis

Case Study. Chen and Gilbert (2017) analyzed piled jackets that had been subjected to hurricanes in the Gulf of Mexico. With the objective of better understanding the uncertainties in the foundation design for offshore pile systems, eight jackets having experienced loads close to or beyond the ultimate capacity of the foundation system were studied. Tables 7 and 8 summarize the platform and pile foundation data. The pile system layouts, with 3, 4, 6 and 8 leg, were typical for the Gulf of Mexico. One pile foundation system failed in overturning (A4), while all the others in Table 7 survived the hurricanes.

Table 7—Summary of study platforms, pile foundation system (after Chen and Gilbert 2017).

Platform	Hurricane (year)	Foundation performance	Installation year	Platform age (yrs)	Water depth (m)
A1	Katrina (2005)	Intact	1965	40	42.7
A2	Katrina (2005)	Intact	1966	39	42.7
A3	Katrina (2005)	Intact	1984	21	67.1
A4	Ike (2008)	Failed, overturning	2001	7	109.7
A5	Rita (2005)	Intact	1972	33	57.9
A6	Rita (2005)	Intact	2000	5	91.4
A7	Katrina (2005)	Intact	1967	38	45.7
A8	Katrina (2005)	Intact	1973	32	45.7

Table 8—Summary of pile foundation systems for study platforms (Chen and Gilbert 2017).

Platform	No. of piles	Pile dia. (m)	Pile length (m)	Dominant soil type		Information from borings
				Base shear	Overturning	
A1	8	0.84	41.1	Clay	Sand	26.5 m clay, underlain by dense sand
A2	6	0.91	42.7	Clay	Sand	Same as A1
A3	4	1.22	83.5	Clay	Sand	25.3 m clay shaft friction, tip in sand
A4	3	1.22	67.0/80.8	Clay	Clay	Clay
A5	4	1.22	77.7	Clay	Clay	Tipped in sands for ≈9 m, expect pull-out of pile loaded in tension
A6	4	1.22	80.5/85.6	Clay	Clay	4.3 m sand in to shaft friction
A7	8	1.07	42.7	Clay	Sand	Top 24 m clay over dense sand
A8	6	1.22	64.0	Clay	Sand	Same as A7

Questions to Answer. Given the observations during three hurricanes, what is the reliability of the current design methods? How can the design of foundations for offshore pile systems be improved?

Applied Methodology. The capacity of each jacket and the pile systems was calculated with 3D finite element modelling of pushover analyses (Energo Engineering 2005; PMB Engineering 1993, 1996). The geotechnical properties were used for the soil and the mean yield strength for the steel piles. The results were supplemented with upper and lower bound plasticity methods calibrated with the pushover analyses (Chen 2016; Chen *et al* 2016). The ultimate capacity of the pile systems was expressed in terms of the maximum base shear at the mudline. The wave forces and their uncertainties were estimated from the hind cast data. Method uncertainty was calibrated by comparing the analytically predicted values with the actually observed platform pile foundation system performance using Bayes' theorem.

Results of Reliability Analyses. The calibrated method uncertainty was used to assess the pile system reliability of Tripod A4 and the 8-leg structure A1.

Base shear

For a pile diameter of 1.22 m, the wall thickness was varied to develop designs that satisfy the criteria in API RP 2A-WSD (2000) and API RP 2MET (2014) and the "high-consequence" L-1 and "medium-consequence" L-2 categories. The annual failure probability for the base shear failure mode was assessed by suppressing an overturning failure. The effect of the contribution of the well conductors to the base shear capacity was also obtained. Chen and Gilbert (2017) drew the following conclusions (Fig. 23):

- The annual base shear failure probability for L-1 consequence category jacket designed according to API RP 2MET (2014) is about 75% of that designed according to API RP 2A-WSD (2000). The annual base shear failure probability for an L-2 consequence category jacket designed according to API RP 2MET (2014) is about half of that designed according to API RP 2A-WSD (2000).
- The annual base shear failure probability for an L-1 platform is about half of that for an L-2 platform.
- For all design cases, the lateral capacity of the 20 well conductors significantly increased the reliability of the pile system against a base shear failure.

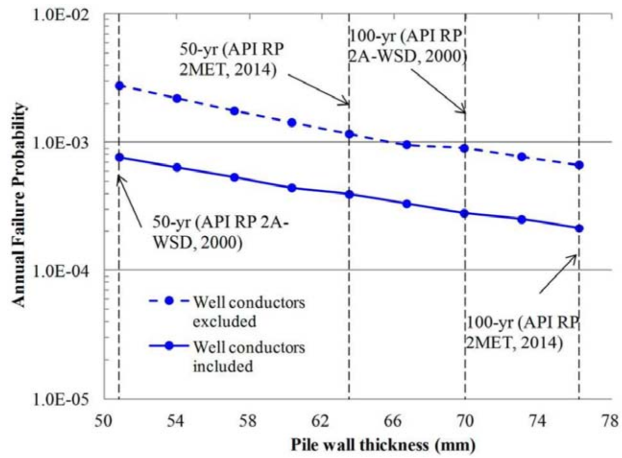


Figure 23—Annual failure probability in base shear for different requirements (Chen and Gilbert 2017)

Overturning

The pile length was varied to develop designs that satisfied the older and current metocean criteria, and the L-1 and L-2 consequence categories. The annual failure probability in overturning was assessed by suppressing a base shear failure. The annual failure probability in overturning decreased with increasing pile embedment (Fig. 16). Chen and Gilbert (2017) drew the following conclusions (Fig. 24):

– The annual overturning failure probability for an L-1 platform designed according to API RP 2MET (2014) was about 70% of that designed according to API RP 2A-WSD (2000). The annual overturning failure probability for an L-2 platform designed according to API RP 2MET (2014) was about half of that designed according to API RP 2A-WSD (2000).
– The annual overturning failure probability for an L-1 platform was about half of that for an L-2 platform.
– If site-specific geotechnical information is not available, the annual overturning failure probability increases by about as much as one order of magnitude.

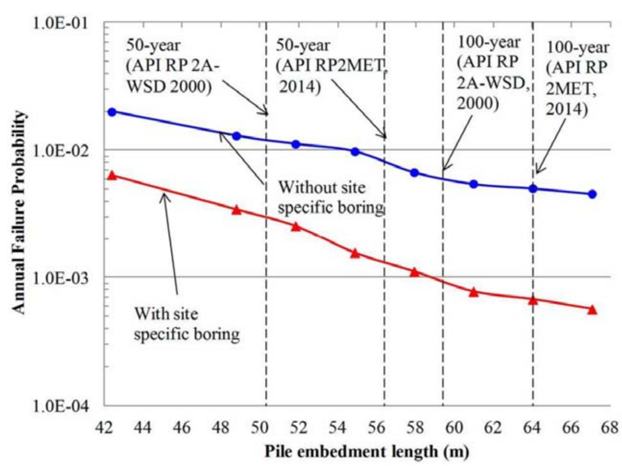


Figure 24—Annual failure probability in overturning for different requirements (Chen and Gilbert 2017)

Probabilistic Complement to Deterministic Analysis. Under similar hurricanes, the annual probability of overturning of the 8-leg pile system was about one-fifth of that for the 3-leg pile system. The analyses showed that (1) the most probable failure mode is overturning for both a 3-leg and a 8-leg pile system. A base shear failure appears much less probable for the 3-leg platform and the 8-leg platform.

The current design practice could potentially be improved by taking into account the mode of failure and redundancy in the pile system to arrive at a more uniform level of reliability. For comparison, Chen (2016) concluded that the superstructure (jacket) has an annual failure probability of about 10^{-3}. This indicates that the most probable failure mode is pile system overturning for the 3-leg platform and a super-structure failure for the 8-leg platform. To support this, the only documented failure of a pile system in a hurricane so far is for a tripod failing in overturning (Chen *et al* 2013). Conversely, there are hundreds of documented superstructure failures for 4-, 6-, and 8-leg platforms under hurricanes (Energo Engineering 2005: 2007; 2010). The reliability approach provides therefor a means to optimize design.

The analyses also indicated that a pile system designed with the earlier metocean criteria and subjected to current metocean conditions has a failure probability about twice that of a new platform.

Chen and Gilbert (2017) also concluded that the Bayesian calibration indicates that the conventional design method for pile capacity is slightly conservative by about 10% for base shear failure of pile systems in clay, unbiased for overturning failure of pile systems in clay, and conservative by more than 50% for overturning failure of piles systems in sand.

Case Study: Jack-up Installation

Case Study. For jack-up platforms with buoyant hull resting on three independent legs, each leg is an inverted cone or spudcan pushed into the seabed. The spudcans are preloaded to check the bearing capacity by exposing them to a higher load than expected during service. Installation in a non-homogeneous seabed is problematic offshore. This case study is for a jack-up installation in sand overlying a soft clay layer, where there is possibility of the spudcan punching through the sand layer into the weaker underlying clay.

The probabilistic method in this case study, developed by Li *et al* (2017) and Cassidy *et al* (2015), was used for a prediction of a Marathon LeTourneau Class 116-C jack-up rig installed in the Gulf of Mexico.

The spudcan had effective area of 144 m^2 and volume of 383 m^3. The soil conditions were a 7-m thick layer of medium-dense to dense fine sand overlying soft to stiff clay. The undrained shear strength of the clay increased linearly with depth, and was thus weakest at the top of the layer.

Questions to Answer. A reliable assessment of the potential for a punch-through failure is crucial for reducing the risk during installation and operation of the jack-up platforms. Traditionally, the peak penetration resistance and depth at which it occurs are determined deterministically without consideration of the uncertainties in the soil or the calculation method. Deterministic load penetration curves are prepared before an installation, with a 'best estimate', and at times a 'lower' and 'upper' prediction, yet no guidance if the observed response during installation deviates from the prediction (as it usually does). Even with the same soil data, engineers calculate *a priori* a wide range of peak resistance, because of the number of methods available and different interpretations of the soil parameters (van Dijk and Yetginer 2015).

Can a more reliable approach, based on observations during installation, be developed? estimate of peak resistance be improved to reduce the risk of punch-through?

Applied Methodology. Li *et al* (2017) developed a quantitative Bayesian framework (see also next section). The approach allows (1) a probabilistic prediction of punch-through load accounting for the uncertainties in the analysis and (2) the updating of the load-penetration prediction with monitored data. The results of 66 'sand over clay' centrifuge tests were used to calculate the uncertainty in the calculation method. The probabilistic framework was used to predict one of the jack-up cases in van Dijk and Yetginer (2015).

Results of Reliability Analyses. Using the Hu *et al* (2014) deterministic model for jack-up spudcan penetration, the punch-through load was predicted to occur for a load of 57.9 MN at a penetration depth of 0.84 m. This prediction was larger than the *in situ* measured punch-through load of 56 MN. The range of predicted loads was 25 to 50 MN (van Dijk and Yetginer 2015). Using the Bayesian framework and the Hu *et al* (2014) formulation, Li *et al* (2017) presented the probability contours of punch-through load as a function of depth (Fig. 25 left). The results indicate a fairly high punch through probability (> 0.2%, *i.e.* $2 \cdot 10^{-3}$) for the deterministically predicted values. The contours suggest that punch-through may occur within the load range of 50 to 65 MN for depth between 0.3 and 1.3 m.

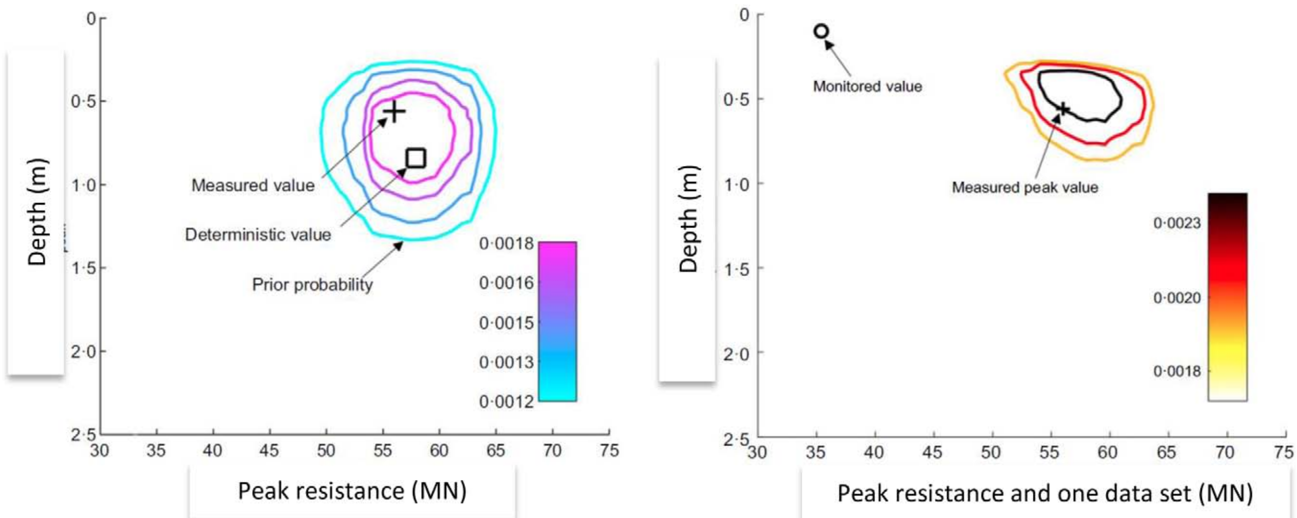


Figure 25—Left: Actual and a-priori predicted punch-through load (after Li *et al* 2017).Right: Probabilistic peak resistance updated with one measurement dataset(color scale indicates probability contours for peak resistance: 0.0020 indicates 0.2% probability).

The probability of the peak resistance were updated with Bayes theorem using the observations during spudcan installation. The monitored data were sets of resistance load and penetration depth. The updated peak resistance as one, two and three datasets became available are shown in Figure 25 (right) and Figure 26. The updated probability contours become more concentrated and show much less uncertainty as the number of datasets increase. With three monitored datasets, the most probable punch-through load ranges from 55.7 to 58 MN, and is close to the actual punch-through load of 56 MN.

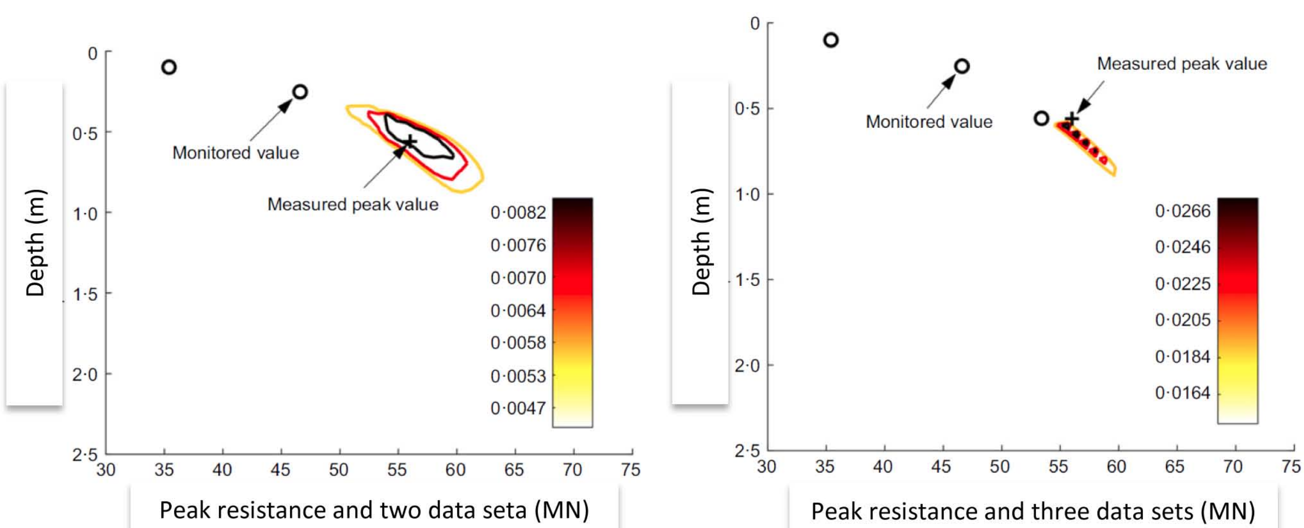


Figure 26—Probabilistic peak resistances updated with two and three measurement datasets (after Li *et al* 2017).(color scale indicates probability contours for peak resistance: 0.0020 indicates 0.2% probability).

Probabilistic Complement to Deterministic Analysis. The new Bayesian method provided an improved estimate the load vs depth curve during jack-up installation, and more importantly good estimates of the uncertainty in the prediction. The probability contours provide more information than the single deterministic value. The range of loads, accounting for the uncertainties in the soil, spudcan and calculation method, is very useful for the operators while installing a jack-up. The method uncertainty based on results of 66 high quality centrifuge tests represents a very useful databank for further understanding of the installation of jack-up structures.

The Bayesian method allows for a real-time update of the probabilities associated with values of peak resistance and depth during installation. The method is able to provide useful guidelines for assisting decision-making during the installation of jack-up units offshore.

The Observational Method and Bayesian Updating

Why isn't the Observational Method not used more for the follow-up of offshore installations? Terzaghi (1961), already quoted in the introduction, added:

> "… In the past, only two methods have been used for coping with the inevitable uncertainties: either adopt an excessively conservative factor of safety, or make assumptions in accordance with general, average experience. The first method is wasteful; the second is dangerous. A third method is provided that uses the experimental method. The elements of this method are ‘learn-as-you-go:’ Base the design on whatever information can be secured. Make a detailed inventory of all the possible differences between reality and the assumptions. Then compute, on the basis of the original assumptions, various quantities that can be measured in the field. On the basis of the results of such measurements, gradually close the gaps in knowledge, and if necessary modify the design during construction."

The Observational Method

The seminal Observational Method (Peck 1969) consists of:

1. Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavorable conceivable deviations from these conditions. In this assessment geology often plays a major role.
3. Establishment of the design based on a working hypothesis of behavior anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
5. Calculation of values of the same quantities under the most unfavorable conditions compatible with the available data concerning the subsurface conditions.
6. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
7. Measurement of quantities to be observed and evaluation of actual conditions.
8. Modification of design to suit actual conditions.

Geotechnical engineers work in both a theoretical dimension and a practical dimension. Both have aleatoric and epistemic uncertainties, which can be reduced but never completely eliminated. Because of the uncertainties, there is always a finite, even if very small, probability that a failure may occur.

The Observational Method requires a consideration of the uncertainties in the parameters and a clear plan for dealing with unexpected events: it includes uncertainty and risk (*"assessment of the most probable conditions and the most unfavorable conceivable deviation"s*), hazards *("calculation of values of the same* *quantities under the most unfavorable conditions*"), and mitigation measures *("selection in advance of a course of action or modification of design for every foreseeable significant deviatio"n* and *"modification of design to suit actual condition"s*). Lacasse and DiBiagio (2019) presented cases where the Observational Method was used for an offshore platform and for a tailings dam. In the case of the CDP1 platform in the North Sea, the Observational Method provided a much better understanding of the soil conditions and soil behavior under the environmental loads and enabled the platform to remain in operation over its planned life, with mitigation measures and preparedness plans in the case of very severe storms.

Connecting Bayesian Updating to the Observational Method

Bayes' Rule says that uncertainties expressed as probabilities can be modified (*i.e.*, updated) by observational information according to the conditional probability (Likelihood) of those observations. Bayes' theorem is a tool made for geotechnics, as most of what geotechnical engineers do is Bayesian (Christian and Baecher 2011)! Most often, the estimates of soil profiles, soil properties, method uncertainties and predictions are based on both measurements and earlier experience and engineering judgment. Two sets of data, expressed as mean value and standard deviation and assuming both datasets are normally distributed, can be combined by Bayes theorem as follows:

$$\mu_{updated} = (\mu_1/\sigma_1^2 + \mu_2/\sigma_2^2)/(1/\sigma_1^2 + 1/\sigma_2^2) \quad \text{(Eq. 2)}$$

$$\sigma_{updated} = (\sigma_1^2 \cdot \sigma_2^2)/(\sigma_1^2 + \sigma_2^2) \quad \text{(Eq. 3)}$$

where μ_1 and σ_1 are the mean and standard deviation of the first dataset (the prior), μ_2 and σ_2 are the mean and standard deviation of the second set of data (the measurements or the likelihood) and $\mu_{updated}$ and $\sigma_{updated}$ are the updated (posterior) estimates of the mean and the standard deviation. The result is an updated average weighted by the inverse of the standard deviations.

A dynamic updating of the risk picture (means and standard deviations) with real-time measurements and already prepared response scenarios would be an easy way to make designs safer and provide support for "risk-informed" decision-making, also in real-time. The use of the Observational Method truly reflects a geotechnical engineer's ambition: a coherent combination of observations, analysis, judgment and risk-informed decision-making for optimising civil engineering structures.

Selection of Characteristic Value

Some of the analyses herein pointed out the importance of the selection of the characteristic value. Yet, there is no consensus on how the geotechnical engineer should select this characteristic value. This is a deficiency in our guidelines.

Many geotechnical design codes recommend that characteristic value be based on "a cautious estimate of the mean soil properties affecting the occurrence of the limit state", or similar wording. The number of discussions published on the topic of the selection of characteristic shows that the present wording in the standards, and the understanding of the wording, are not clear, and therefore not amenable to be a prescriptive text. Because of the ambiguity, the selection of the characteristic parameters to use in deterministic analysis can be very subjective and varies widely in a project.

In geotechnical practice today, the selection of a characteristic value depends very much on the experience, expertise and risk tolerance of the engineer performing the design. One engineer may pick a value based on the mean. A risk-averse engineer may choose a characteristic value based on the minimum suggested by the soil data. The simplest would be to use the mean value of the parameters, based on statistics where possible. This would make the definition of characteristic value completely unambiguous; however, this would require a recalibration of all the guidelines and codes.

Frequency of failure

Christian and Baecher (2011) asked the question "Why are failures less frequent than our reliability studies predict?" Typical coefficients of variation (COV) reported for soil engineering property are 20-30% or less (*e.g*. Baecher and Christian 2003). For a mean factor of safety of 1.5, the corresponding reliability index (β) is about 1.1 to 1.7, thus implying a failure probability close to 10% for "normally distributed" uncertainties. This is at least one order of magnitude larger than the observed frequency of adverse performance, and two orders of magnitude larger than the frequency of earth dam failure (Baecher *et al* 1980).

One explanation is that the uncertainty in soil properties may be overestimated. If the COV *in situ* is actually smaller than 20-30% used to estimate the failure probability P_f, P_f will be overestimated.

A second explanation is that one does use a conservative fraction of the mean in the analyses. US Army Corps of Engineers (USACE 2003) uses a 1/3-rule in choosing engineering properties: the design value is taken at a value which is larger than 1/3 of the observations and smaller than 2/3. For normally distributed data, that is approximately the mean less 0.4 standard deviation. This implies a reliability index with respect to the mean of 1.5 to 2.1, or failure probabilities close to 5%.

There is also a tendency for engineers to be conservative in estimating soil properties, underestimating strength and overestimating compressibility, even when trying to identify the best estimates. It is also possible that the actual rate of unsatisfactory performance may be underreported.

5% Fractile Value

ISO 19900:2013 requires in Section 9.3.1 that "… the characteristic value shall generally be defined as the value below which 5% of the values are expected to fall". NORSOK N-001 (2004) has equivalent requirements.

Orr (2015; 2017), a veteran on standards and characteristic value, stated clearly that a characteristic value of "mean -1.65σ", or in other words the 5% fractile of a population of measurements where σ is the standard deviation, is not applicable in geotechnical design because the volume of soil involved in a failure is usually much larger than the volume of soil involved in a field test or in a test on a laboratory specimen. Orr added that "*since soil, even homogeneous soil, is inherently variable, the values affecting the occurrence of a limit state is the mean, i.e. average, value over the relevant slip surface or volume of ground, not a locally measured low value. Hence, in geotechnical design, the characteristic strengths is the 5% fractile of the mean strengths measured along the slip surface or in the volume of soil affecting the occurrence of the limit state*". Orr also specified that a characteristic value of "mean -1.65σ" applies only for a normally distributed population and when the volume of the material in the actual structural element being designed is similar to the volume of the test element. This is the situation for structural element such as the results of tests on concrete tubes. However this is not the case for geotechnical parameters.

Schneider and Schneider (2013) asked the question: "What is the 5% fractile values, and which one?". They answered with the left-hand distribution in Figure 27. The figure illustrates the interpretation of the 5% fractile (see also Schneider 1997). The 5% fractile at a local point (small zone of influence) is very different from the 5% fractile for a large zone of influence. The values at the bottom of the graph are similar to those obtained when evaluating values for 95% confidence of the mean (DNV 2012; Ronold 2016).

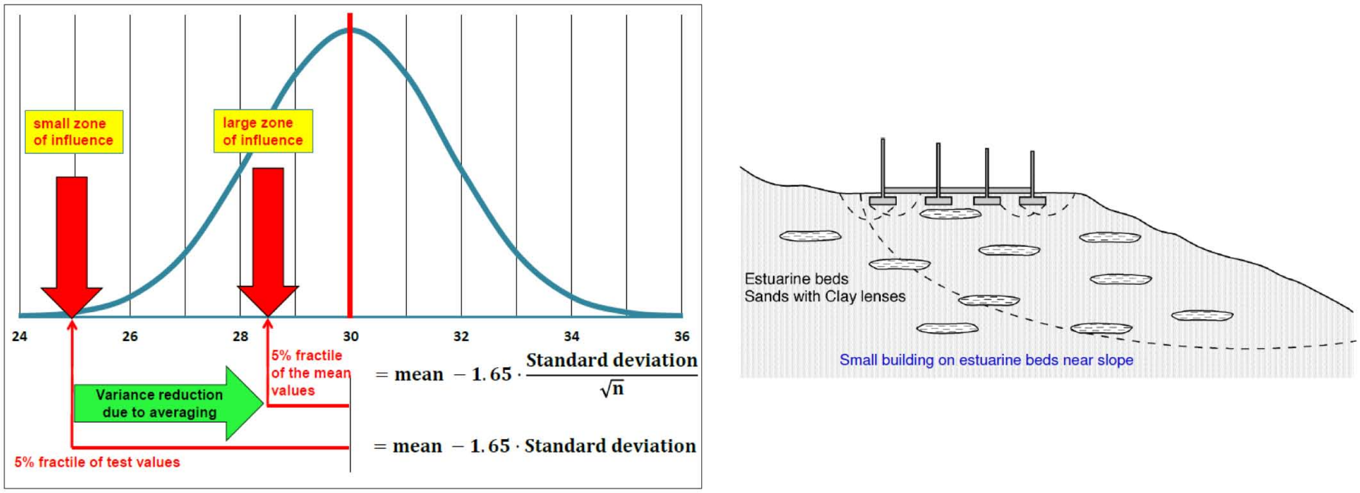


Figure 27—Left: Illustration for 5%fractile for small and large "zones of influence" (Schneider, H.R. Personal comm. Sept. 2013).Right: Illustration of zones of ground governing the behavior of a geotechnical structure at a limit state (Simpson 2014).

The *5% fractile of a population of data points* should be taken as the mean±1.67σ (σ is the standard deviation) while the *5% fractile of the mean value at a limit state* should be taken as the mean±0.66σ.

Simpson (2014), another veteran on standards and characteristic value, concluded that "cautious" means having a worse effect on the response than the most probable value and is an estimate of the value affecting the occurrence of the limit state. He stated further that the characteristic value can be taken as a value of 0.5σ below the mean when the limit state depends on the value of a parameter averaged over a large volume of ground (i.e. a mean value), the ground property varies in a homogeneous and random manner and at least 10 test values are available to estimate the mean. The zone of ground governing the behavior of a geotechnical structure at a limit state is usually much larger than a test sample or the zone of ground affected in an *in situ* test, as also stated by Orr (2015).

Simpson (2017) concluded that: *"It should be clear that the characteristic value required by EC7 is not a 5% fractile of test results, but rather that there is a 5% probability that a worse value could be representative of the whole body or surface of soil that governs the occurrence of the limit state*." A similar proposal was made by Dahlberg and Ronold (1993) for design of offshore foundations and by Becker (1996a; b) for more general use. Simpson (2014) also noted that the characteristic value equal to the "mean±0.5σ" should be a useful consideration and not an unbendable rule.

Characteristic Value Used in Practice

In connection with Eurocode developments, Simpson (2012) reported that characteristic values tend to be selected in Europe at about 0.5σ below the mean, where σ is the standard deviation. Simpson reported that in the US, the characteristic value is selected at values between 0.50 and 0.75 σ below the mean.

From a review of the practice at several sites in the North Sea and the Gulf of Mexico, Lacasse *et al* (2007a; b) observed similar trends where the characteristic value was taken at 0.5σ below the mean.

Cheon *et al* (2015) documented the practice to select undrained shear strength for the design of shallow and deep foundations in deepwater in Gulf of Mexico. They investigated more than 100 design profiles of undrained shear strength for normally to slightly overconsolidated, highly plastic, marine clays. Undrained shear strengths were measured/derived from miniature vane, direct simple shear, unconsolidated-undrained triaxial compression, field vane and cone penetration test. They concluded that the mean of the measured shear strengths was within ±12 percent of the characteristic strengths selected for design, for all types of strength measurements. They also observed that that cone penetration test (CPT) measurements gave smaller variability about the mean than all the laboratory measurements.

Using a 95% confidence level concept (DNV 2012), it is possible to calculate a representative value as a function of the number of data points available for the estimation. For cases such as CPT data or the Cheon *et al* (2015) numerous data in the Gulf of Mexico, the cautious estimate can be taken closer to the mean than 0.5σ below the mean. For 50 data points, the estimate with 95% confidence is 0.23σ below the mean; for 100 data points, the estimate with 95% confidence is 0.16σ below the mean; and so on.

Summary on Characteristic Value

In summary, the characteristic value is not one particular fractile of the results of particular laboratory tests on soil specimens, but it should be a cautious estimate of the value(s) affecting the occurrence of the limit state. The characteristic value shall also include the designer's expertise and experience. The characteristic value should aim to be a moderately conservative and a representative value or what good designers have always done in the past (Simpson 2012). The considerations of the above 5% fractile interpretation leads to the following conclusions:

When the characteristic value is based on laboratory or few discrete *in situ* data, the characteristic value shodul be selected as the mean ± 0.5σ, where σ is the standard deviation about the mean, for conditions where the limit state depends on the value of a parameter averaged over a large volume of ground. For a smaller volume of ground involved in the limit state, it is recommended, on the basis of Schneider (2013) and studies of requirements for 95% confidence, to take the characteristic value at the mean ± 1σ. The profession should use more extensively statistical analysis when selecting parameters because it provides additional unbiased information.

When the characteristic value is based on nearly continuous cone penetration tests (CPT/CPTU), complemented with some laboratory data, the characteristic value of the cone resistance should be taken close to the mean value:

– Using estimates with 95% degree of confidence, the characteristic value of the cone resistance in sand should be selected very close to the mean cone resistance q_c or q_t, *e.g.* 0.10σ below the mean when the cone resistance is used directly in the design equations. The standard deviation of the cone resistance will be smaller than calculated due to spatial averaging, which is usually not included in the analysis of the data.
– For CPT data in clay, one should be more cautious than for sands because the cone resistance is a proxy for the undrained shear strength, which usually shows higher variability than the friction angle (Lacasse and Goulois 1989). However, the standard deviation of the cone resistance will also be reduced due to spatial averaging. If the cone resistance measurement is used to derive undrained shear strength, the transformation uncertainty from cone resistance to undrained shear strength should be included in the evaluation of the standard deviation of the resulting undrained shear strength. This suggests a more cautious characteristic value for clays than for sands when the estimation is done from cone resistance values.

Summary and Conclusions

This paper aimed at illustrating that a reliability-based approach is more rigorous and more "complete" than the deterministic approach alone because it accounts for the uncertainty in the analysis parameters, their correlation, and leads to a more robust design. The reliability-based approach is not meant to replace the traditional deterministic approach. Instead, it should be used as a complement to deterministic analyses. Five examples were used to illustrate the complementary information provided by probabilistic analyses. Recommendations are made in the paper for acceptable or tolerable risk level, on exploiting the seminal Observational Method to a greater degree in offshore geotechnical engineering and for the selection of the characteristic value for deterministic analysis.

An analysis that allows for both deterministic and probabilistic modelling provides an improved understanding of the potential range of behavior under various modelled uncertainties. For example, a probabilistic analysis can be used to confirm the validity of the deterministic safety assessment. Probabilistic modelling (i.e. running multiple scenarios with different probabilities of occurrence) can be used to generate a deterministic scenario, such as the worst-case, e.g. the maximum losses, the best-case, e.g. the losses that can be absorbed or the most "likely" case, e.g. the losses that are most likely to occur. The design criterion in reliability-based design is defined in terms of a target annual failure probability, and the profession needs to develop easy-to-understand probabilistic results. Method uncertainty remains the predominant source of uncertainty in most cases.

There is a gradually increasing demand to adopt a reliability-based approach for the design of geotechnical structures (ISO 2394:2015). Probability and risk concepts have now reached maturity in the geotechnical profession. The techniques have considerably evolved in past years, and the probabilistic analysis software packages are accessible to most geotechnical engineers, and easy to use.

It is important to select the risk assessment and management approaches in response to a client's needs for quantified or qualified risk assessment, commensurable to the costs of the project. Most owners and operators understand the concept of risk. They expect engineers to provide probability and risk information and/or quantification to help them make risk-informed decisions.

A reliability-based estimate is not necessary for all geotechnical problems, but such estimates should be used to a greater extent for more difficult designs or when there are significant uncertainties that could influence the result or lead to decisions other than those based on deterministic results alone. All levels of reliability analyses are an underutilized tool for making better engineering and management decisions.

The perceived difficulty and expense associated with determining failure probability by rigorous mathematical means has held back the widespread adoption of risk-based analyses in geotechnical engineering. This perception is not true today.

Considering computed annual failure probabilities, one may ask: "What is the true annual failure probability of a foundation?" Interpretation of the calculated annual failure probability is important when the results of the reliability analyses are used to optimize a foundation. This requires an understanding of the significance of different categories of uncertainty on the calculated results. The DNV Guideline for Offshore Structural Reliability (1996), prepared as part of a DNV-NGI cooperation, pointed out that the calculated failure probability in a reliability analysis is a "notional failure probability":

> "*The calculated reliability is not a property of the structure. It is a property of our knowledge of the structural reliability of a structure, e.g., new information about the uncertainty of a parameter will change the calculated reliability. This implies that there is no contradiction when two analysts calculate two different reliabilities for the same structure, since they may have different information. However, two analysts with the same information should arrive at the same reliabilities. Since the uncertainties related to knowledge [aleator"y and "epistemi"c uncertainties] are modelled and since they contribute to the calculated probabilities of failure, reliability analysis will tend to overpredict the failure probabilities from failure/accidental data when gross errors are removed from the database*."

This statement suggests that probabilistic analyses will yield a somewhat conservative estimate of the margin of safety. The calculated "notional" failure probability includes the effects of both epistemic and aleatoric uncertainties. The true failure probability, on the other hand, is only a function of the aleatoric uncertainties, which is always less than the sum of epistemic and aleatoric uncertainties.

There is today a cultural shift in risk and reliability of civil engineering structures and the social perception of the engineer's work. Table 9 compares the earlier focus (first column) with the new priorities replacing the earlier focal points (second column). Science and engineering help us to predict hazards and their probability of occurrence. Knowing the hazards and the risk helps making risk-informed decisions. The time to implement the tools that enable us to qualify or quantify the risk is now. In the future, the profession will need to show that the key decisions were "risk-informed" (ISO2394:2015).

Table 9—Cultural shift required for landslide risk management

From earlier focus	To new priorities
From Hazard	To Consequence
From Response	To Preparedness and Risk Reduction
From Reactive	To Proactive
From Science-driven	To Multi-disciplinary
From Response management	To Risk Management
From Single agencies	To "Everyone's business
From Planning for communities	To Planning with communities
From Decision-making	To Risk-informed decision-making

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