Risk management in rock engineering

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1 Introduction

General risk management includes also financial, political, personal, environmental or social risks etc. however, this chapter focuses on the ‚geo‘-related risk components. An extensive overview is provided by Proske (2004).

Design and dimensioning in rock engineering differs significantly from design in engineering in general. Compared to manufactured materials like steel, geosynthetics, concrete, bricks, ceramics etc. rock masses are characterised by the following features:

- Much larger uncertainty in properties, e.g. strength, stiffness, permeability etc.
- Large uncertainty in primary (initial) loading conditions, e.g. initial stress state, initial pore water pressure etc.
- Large uncertainty in the near-field conditions, e.g. existence of nearby faults, cavities etc.
- Large representative elementary volume (REV) and significant scale effects

In general, uncertainty can be subdivided into two categories:

- **Episdemic uncertainty** = uncertainty caused by restricted knowledge about the rock mass under consideration (in a broader sense this includes also omissions, measurement errors or model uncertainties)
- **Aleatory uncertainty** = uncertainty caused by spatial-temporal natural fluctuations (randomness) in properties and initial conditions

Aleatory uncertainty is given by nature and cannot be influenced by humans. Episdemic uncertainty can be reduced by increasing of investigations. However, amount and quality of the knowledge about the situation is always limited due to technical, financial and ecological reasons. Therefore, in rock engineering always a relatively high uncertainty remains. High uncertainty increases the risk and vice versa, whereby the risk is defined as:

$$\text{Risk} = \text{Probability of event} \bullet \text{Severity of consequences}$$

Fig. 1.1 illustrates the risk for several events. Risk is a dynamic term, that means it changes with ongoing construction (becomes smaller due to better knowledge with ongoing construction stages) and it could be defined in a different manner for different construction stages (see also Fig. 5). Risk reduction can be reached either by reduction of uncertainties or by reduction of consequences in case of failure.

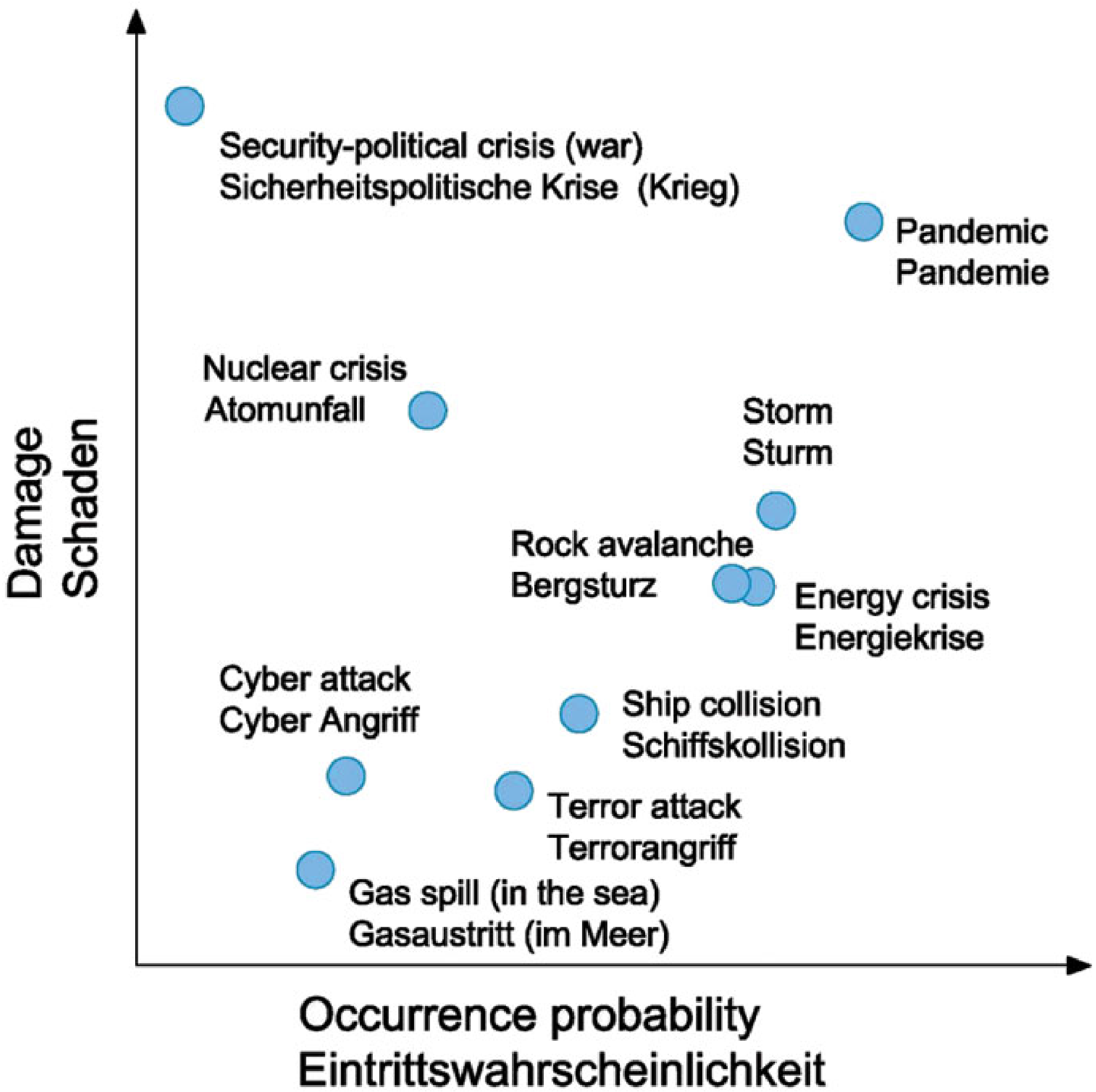


Fig. 1.1: Norwegian risk matrix (Poisel et al., 2012).

2 Acceptable risks

Whether a risk is acceptable or not depends on several questions or factors (Herrmann & Konietzky, 2016):

- Is exposure to risk by choice or unintentionally (e.g. natural disasters)?
- Is risk manageable or not?
- Is the person familiar with a risky operation?
- Are there alternatives to reduce or avoid risks?
- Ratio between potential advantages and potential negative consequences.
- Reporting in communication media.
- Time delay between event and point in time of evaluation
- Personal consternation / experience.

Fig. 2.1 illustrates in-situ observed and accepted risks in geotechnics in terms of probability of failure and consequences in terms of number of fatalities and money.

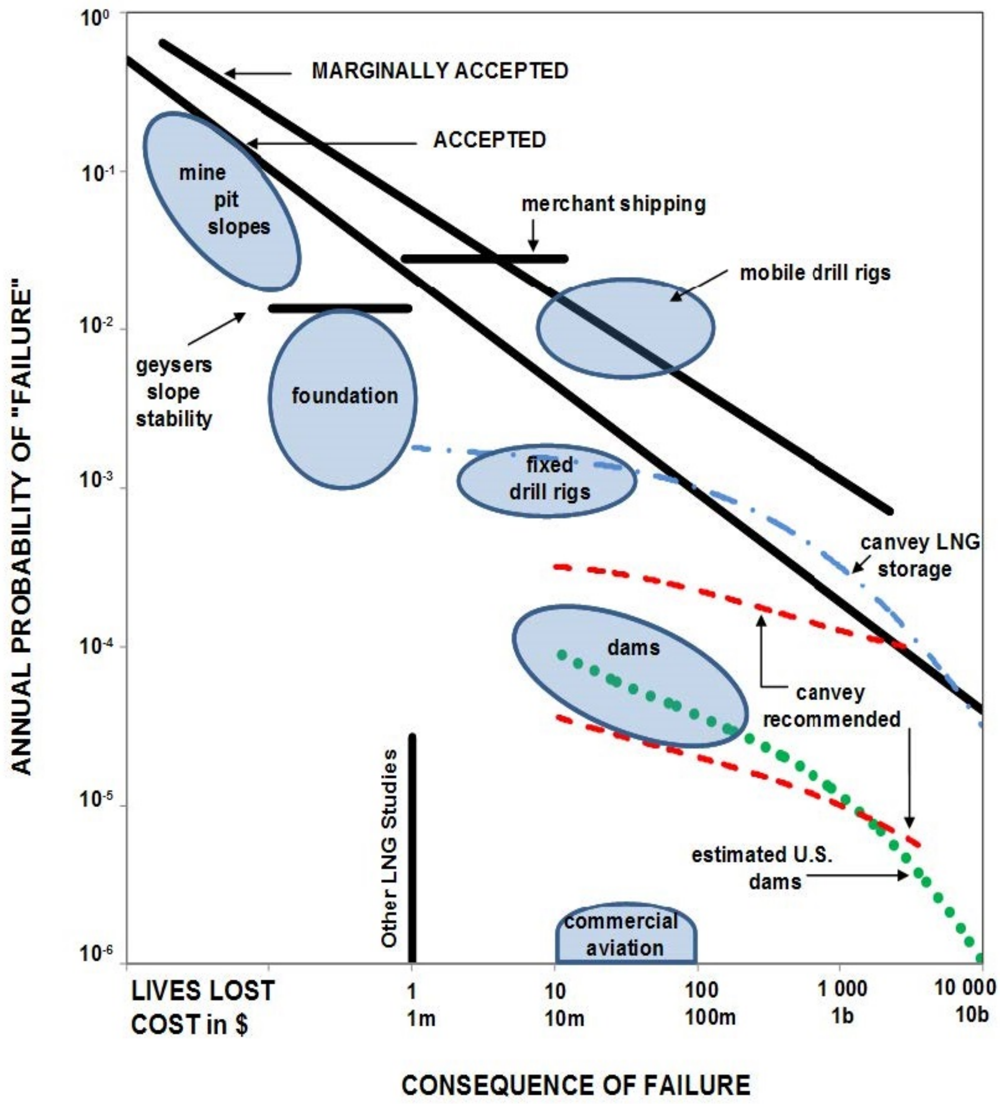


Fig. 2.1: Geotechnical risks and their acceptance (Cathie, 2014).

According to Whipple (1986) the following acceptable risk levels are proposed:

- Short-term risk (e.g. recreational activities): $< 10^{-6}$/h
- Occupational risk: $< 10^{-3}$/year
- Public risk (e.g. dam failure) $< 10^{-4}$/year

Examples for acceptable risks are:

- Existing engineered slopes:
  - 10^{-4}/year for person most at risk (Lacasse, 2016),
  - 10^{-5}/year for average person at risk (Lacasse, 2016),
- New engineered slopes:
  - 10^{-5}/year for person most at risk (Lacasse, 2016),
  - 10^{-6}/year for average person at risk (Lacasse, 2016),
- Natural hazards in Austria: 10^{-5}/year (ÖGG, 2014).

Risks are frequently ignored or accepted when individual risk falls below 10^{-6}/year. According to Curbach & Proske (2003) an acceptable risk can be calculated as follows:

$$E(N) + k \cdot \sigma(N) < \beta \cdot 100,$$
$$E(N) = N_A \cdot P_F \cdot k_v \cdot N,$$
$$\sigma(N) = k_v \cdot N \cdot \sqrt{N_A \cdot P_F \cdot (1 - P_F)},$$

where: N number of effected people per event, $E(N)$ and $\sigma(N)$ = corresponding expectation and standard deviation, respectively, P_f = failure probability, k_V = ratio of number of effected people und number of fatalities, k = confidence interval (in most cases k = 3) and β = "factor of political influence". β can range between 0.01 (e.g. natural hazards) and 100 for risk by choice for activities with direct benefit for the corresponding person.

A common form to represent risks is the so-called F-N-diagram (number of fatalities versus probability of occurrence per year). Such diagrams are plotted in a double-logarithmic scale. F-N design curves follow the following relation:

$$F \cdot N^a = k ,$$

where k is a constant and a is the factor describing the subjective risk aversion. Fig. 2.2 illustrates the influence of the two factors k and a. The area below the F-N curve is defined as acceptable risk. Typically, F-N curves are constructed by the definition of an anchor point k (for instance 10 and 10^{-4} for a bigger accident in a factory) and a corresponding aversion risk factor a. In the 1980s and 1990s several F-N-curves are developed, see Fig. 2.3.

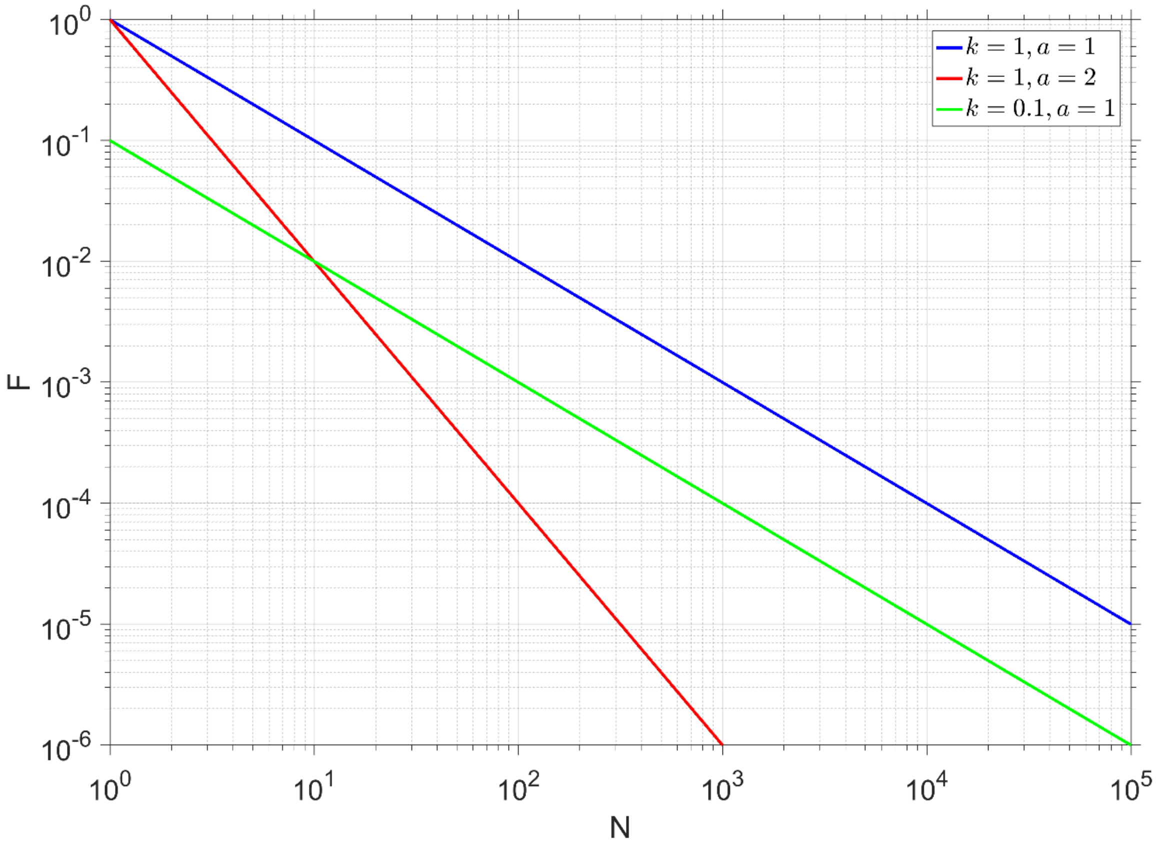
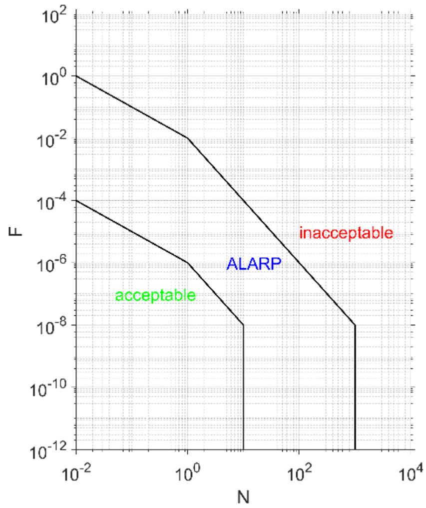


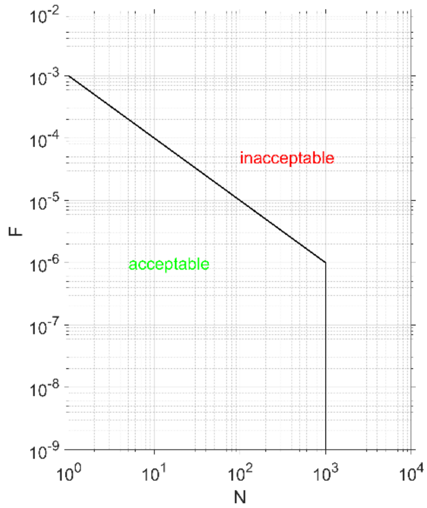
Fig. 2.2: F-N design curves illustrating the effect of parameters *a* and *k.*

There are some alternatives to the F-N-diagram:

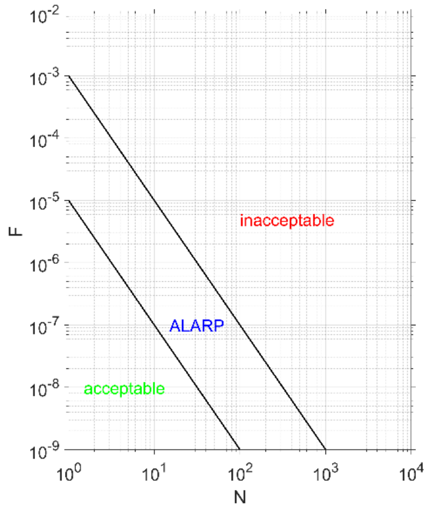
- F-D-diagram: The damage (D) is given in financial means. The value for loss of live is fixed.
- F-PAR-diagram: The number of people at risk (PAR) is given on the x axis.
- F-T-diagram: The measure of damage is the time (T), which is required for repairing the damage.
- F-E-diagram: Energy is lost due to a damage event, e. g. energy lost through injury or death and the energy needed to repair the damage. These lost energy (E) is plotted on the x axis.



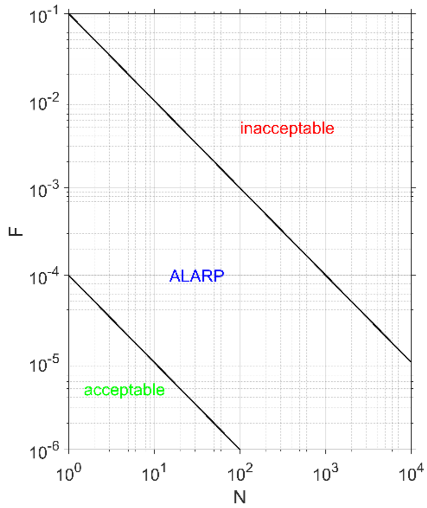
a.) Groningen 1978.



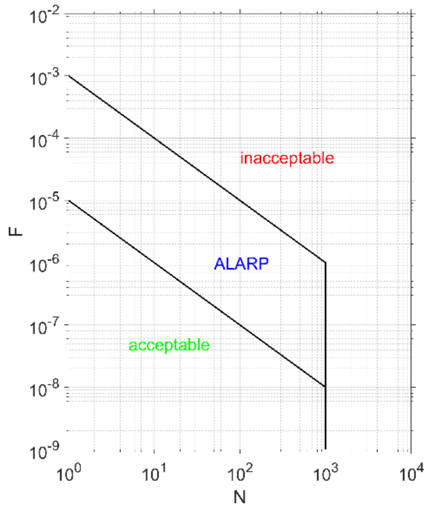
b.) Hongkong 1988.



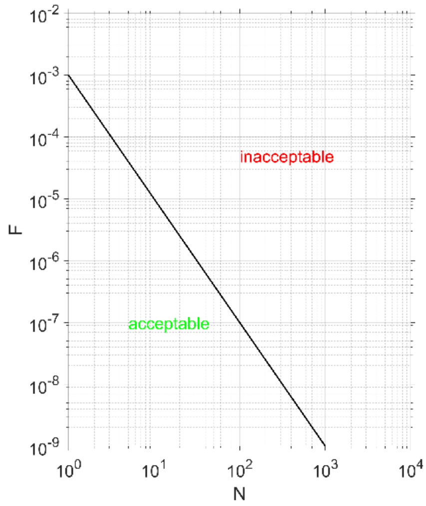
c.) Netherlands 1980th .



d.) ACDS Great Britain 1991.



e.) Hongkong 1993.



f.) Netherlands 1996.

Fig. 2.3: Several F-N-curves (Proske, 2004).

3 Risk based design procedure

According to ISO-31000 (Fig. 3.1) a risk-based design procedure consists of 4 steps:

1. **Risk identification** = identification of hazards
2. **Risk analysis** = determination of risk based on the probability of the event and its consequences
3. **Risk evaluation** = decision if risk is acceptable or not
4. **Risk treatment** = measures to reduce the risk

Risk identification, risk analysis and risk evaluation steps together are also called risk assessment (Brown, 2012). Risk management has to be performed and up-dated in each stage of the rock engineering project like illustrated in Fig. 3.2.

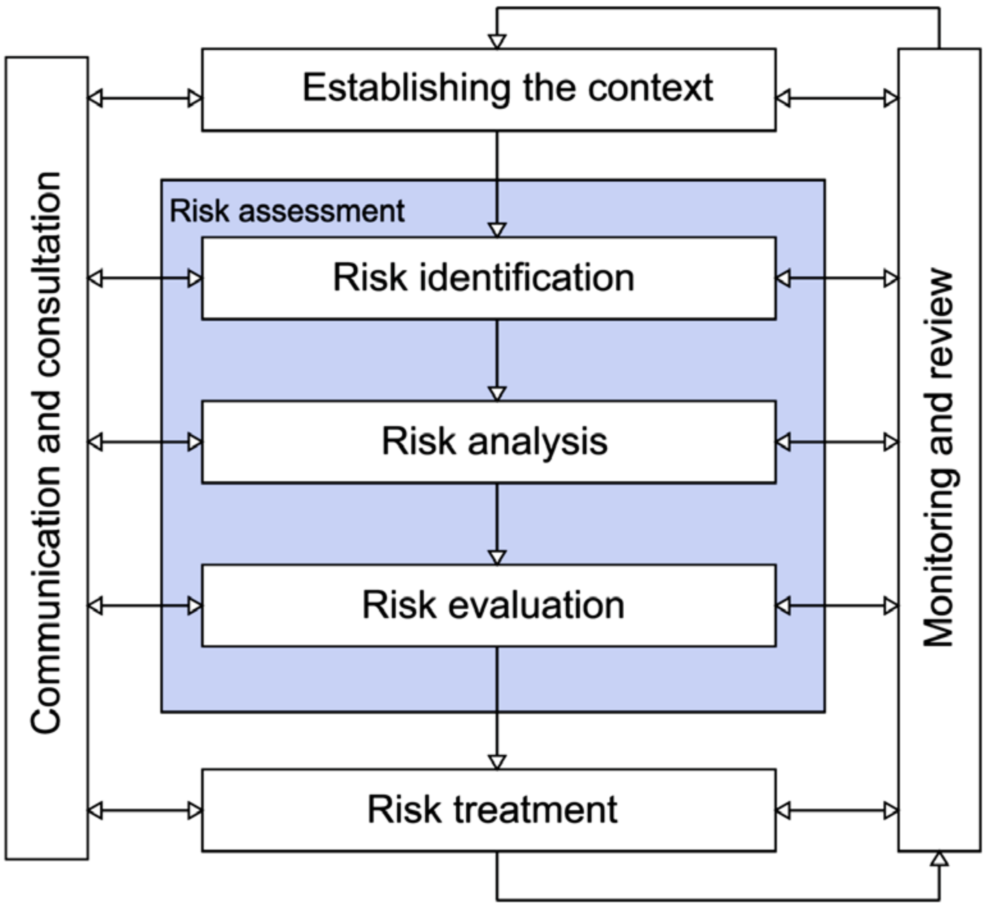


Fig. 3.1: Risk management cycle according to ISO 31000 (ISO, 2018).

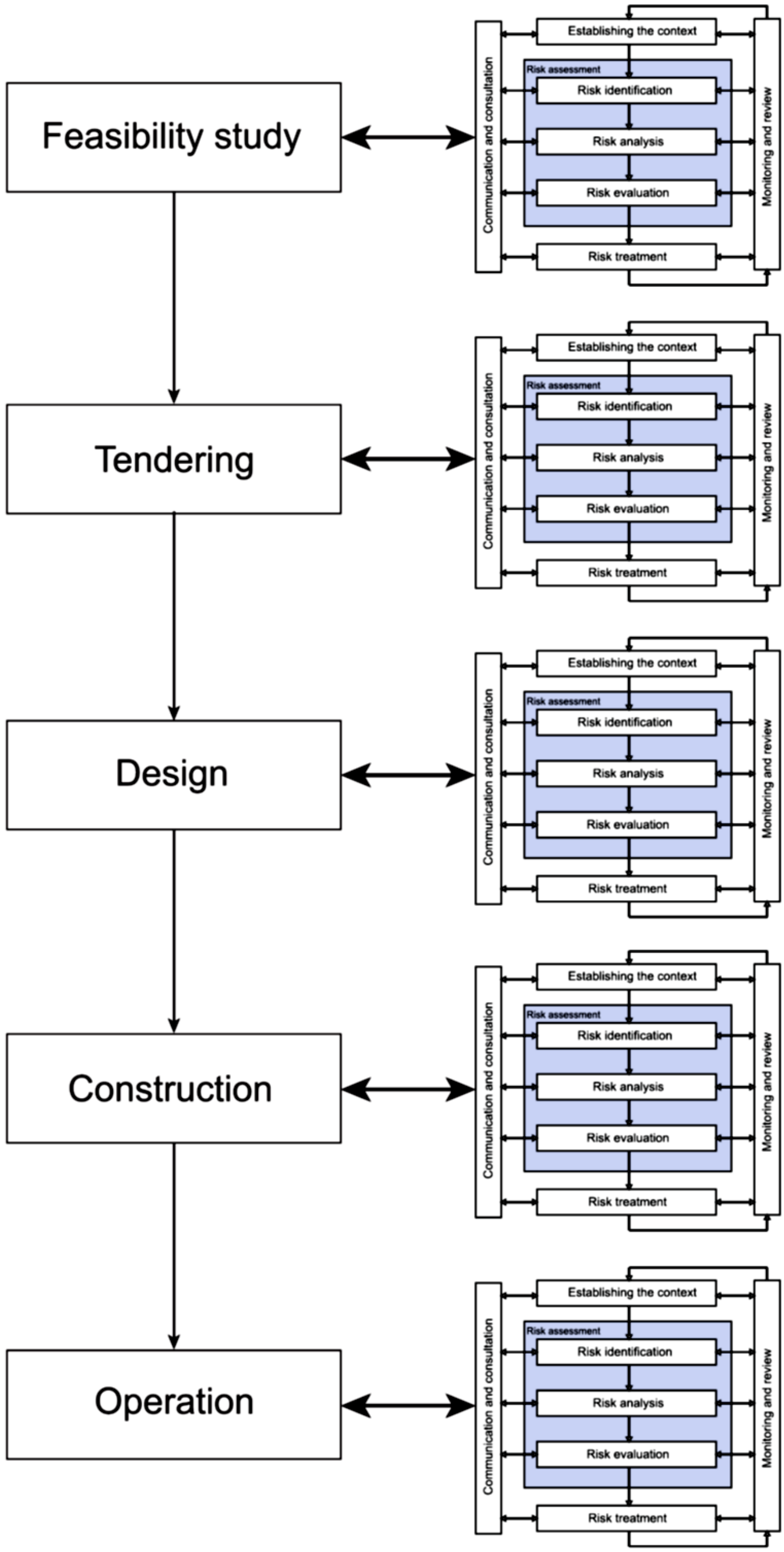


Fig. 3.2: Application of risk management in different stages of rock engineering projects (Spross, Olsson & Stille, 2018).

Risk analysis and evaluation comprises the following typical procedures (Brown, 2012):

- Fault tree analysis = identification, quantification and illustration of faults and failures in a diagram form
- Event tree analysis = systematic mapping of event scenarios with potential of major incidents as well as relationships of events with time
- Consequence or cause-consequence analysis = combination of fault tree and event tree analysis, which results in a diagram showing the relationships between causes and consequences or outcomes of an incident
- Bowtie diagrams = illustrate how control mechanisms may eliminate or minimize the likelihood of risk generating events or how consequences can be reduced
- Probabilistic risk analysis = probabilistic simulations for quantitative risk analysis (e.g. Monte Carlo simulation)
- Decision analysis = analysis of the outcomes of decisions or choices based on available information
- Multi-risk analysis = approximate computational method involving multiple statistically independent risks or hazards each treated as stochastic variable
- Analytical hierarchy process = mathematical approach for multi-criterion decision-making to rank decision alternatives based on pair-wise comparison
- Bayesian networks = probabilistic-based graphical tool to show relationships between system components
- Artificial intelligence methods = Fuzzy logic, neural networks etc.

Risk is difficult - sometimes even impossible - to quantify. Therefore, in rock engineering risks are often classified into more general categories. According to the Eurocode 7 (DIN, 2015) three safety classes (SC1, SC2 and SC3) are defined according to three geotechnical categories (GC1, GC2 and GC3). Higher numbers correspond to higher safety classes and higher geotechnical complexity, respectively (Tab. 3.1).

Tab. 3.1: Risk management classes according to Eurocode 7, orange indicates highest risk (DIN, 2015).

Safety class	Geotechnical category		
	GC1	GC2	GC3
SC1	1	2	3
SC2	1	2	3
SC3	-	3	3

Tab. 3.2: Recommendations for minimum values of the reliability index β according to Eurocode 7 (DIN, 2015).

Reliability class	Minimum value for β	
	Reference period 1 year	Reference period 50 years
RC 3	5.2	4.3
RC 2	4.7	3.8
RC 1	4.2	3.3

The Eurocode 7 also defines three categories of damage consequences (classes CC3, CC2 and CC1):

- CC 3: Huge negative consequences for human or huge economic, social or environmental consequences (for instance: concert hall, stadium, hospital etc.)
- CC 2: Midsize negative consequences for human or considerable economic, social or environmental consequences (for instance: residential buildings, office buildings etc.)
- CC1: Minor negative consequences for human or negligible economic, social or environmental consequences (for instance: agricultural storage facilities etc.)

These three damage consequence classes can be directly related to three reliability classes RC3, RC2 and RC 1 (Tab. 3.2). The reliability index β is a measure for the reliability and is defined by the following equation:

$$P_f = \phi(-\beta),$$

where ϕ is the cumulative distribution function for the standardised normal distribution. Tab. 3.3 illustrates the relation between β and P_f.

The failure probability can be expressed in terms of the failure envelope f_e:

$$P_f = probability\left(f_e \leq 0\right),$$

with (R = resistance, L = load):

$$f_e = R - L$$

In case f_e is normally distributed, the reliability index is the quotient of mean value μ and standard deviation σ of f_e:

$$\beta = \frac{\mu}{\sigma}$$

A similar classification scheme is proposed by DGGT and DMV (DGGT/DMV, 2017) to evaluate the risk of abandoned mines (Tab. 3.4). According to this scheme the risk increases from class IV to I. The borderline between the green and orange marked classes defines the tolerable risk.

Tab. 3.3: Relation between β and P_f

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

Tab. 3.4: Risk classes according to DGGT/DMV, orange indicates highest risk (2017)

		Extent of losses			
		unimportant	low	high	very high
probability of occurrence	very likely	IV	III	II	I
	likely	IV	IV	III	II
	unlikely	IV	IV	IV	III
	practical im-possible	IV	IV	IV	IV

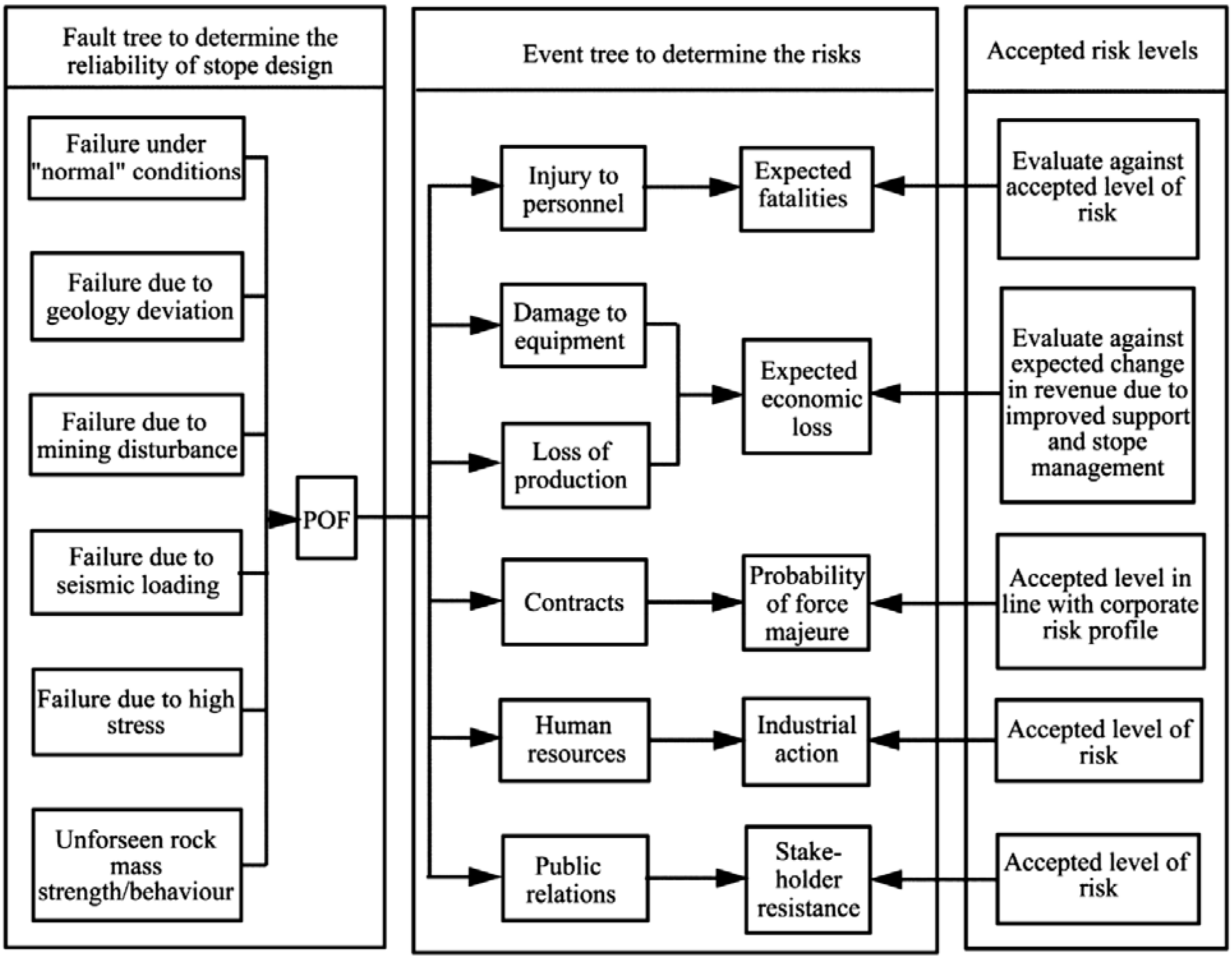


Fig. 3.3: Combination of fault and event tree analysis (Stacey et al., 2006).

Fig. 3.3 illustrates a procedure which combines fault and event tree analysis. Quantitative approaches to evaluate the amount of safety are mainly based on the comparison between distribution functions of load and resistance as shown in Fig. 3.4. This figure also illustrates the effect of standard deviation (scatter of parameter): if both mean values are identical, different standard deviations can lead to quite different safety values. Fig. 9 illustrates how the design procedure can is able to reduce uncertainty and consequently to increase safety and reduce risk.

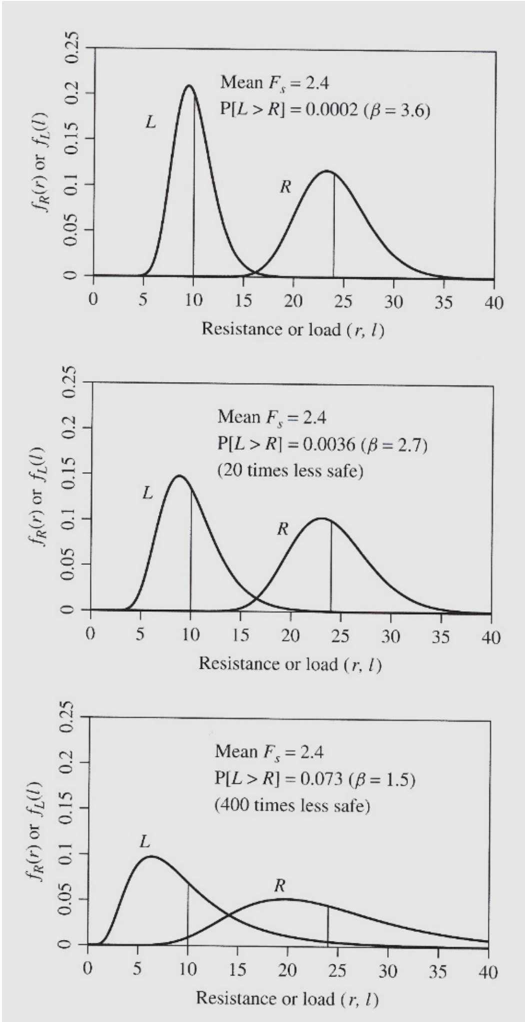


Fig. 3.4: Distribution functions for load and resistance defining the safety margin. Left: general representation, Right: influence of scattering on safety in case of identical mean values (Fenton & Griffiths, 2008).

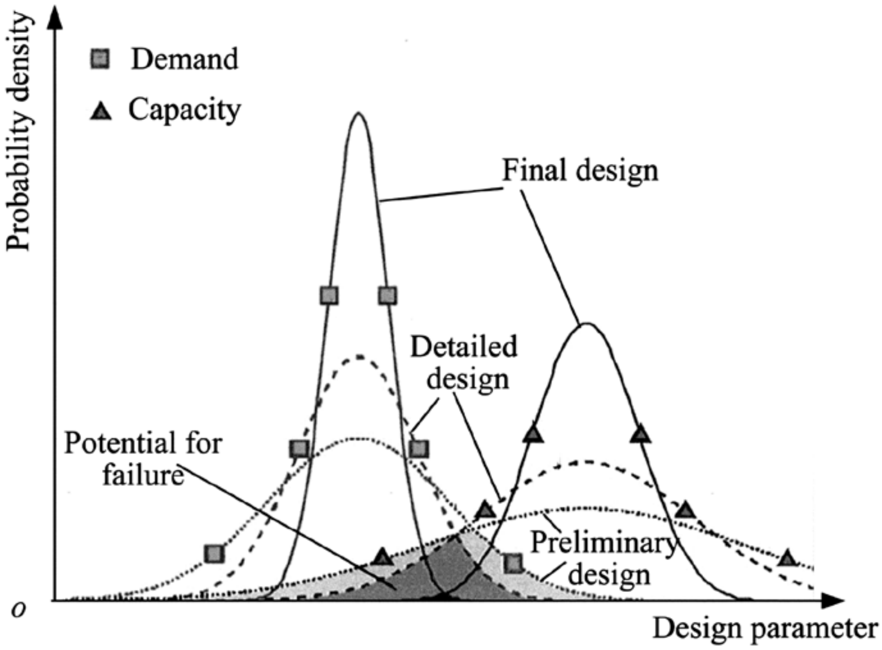


Fig. 3.5:Illustration of uncertainty reduction during design procedure (Valley, Kaiser & Duff, 2010).

4 FEP analysis

To identify potential hazards incl. there consequences and occurrence probability, a so-called FEP (= Features – Events – Processes) analysis is a helpful procedure. FEP analysis is a common procedure in many fields of engineering. In geotechnical engineering this technique is frequently applied for risky operations like waste storage, hydraulic fracturing, leaching processes or CO_2-Sequestration (NEA 2019; Yamaguchi et al. 2013; GRS 2012; Jobmann et al. 2017).

The three keywords in FEP have the following meaning:

Features: characteristics of the site (rock mass properties, discontinuities etc.)

Events: Sudden and rapid processes, short in compasrion to safety assessment timeframe (earthquakes, explosions, vulcano eruptions etc.)

Processes: Slow and long lasting processes (erosion, subrosion, weathering, glaciation etc.)

A FEP analysis answers the question if and to what extend different events and processes influence (change, modify) certain features. A FEP analysis allows to define relevant scenarios for a safety assessment.

A slightly different approach is used by Jobmann et al. (2017) who distinguish between components, properties and processes (see Fig. 4.1).

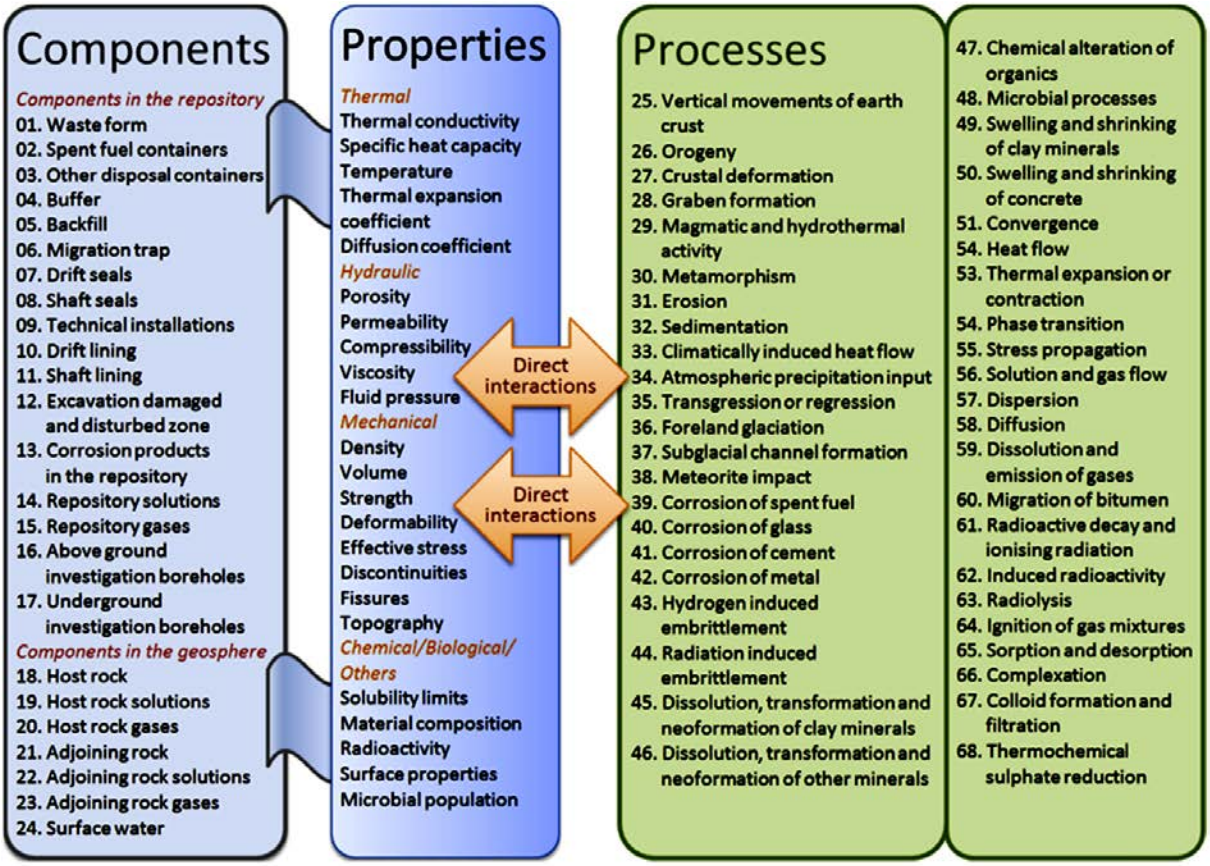


Fig. 4.1: List of components, properties and processes for a high-level nuclear waste repository (Jobmann et al., 2017).

5 Simulation methods

Additionally, there are several simulation methods:

5.1 First-Order Second-Moment method

The First-Order Second-Moment method (FOSM) is based on the first-order Taylor series of the function to be evaluated. The first and the second moment of the random variable has to be known. Using the Taylor series and the both moments, the first and the second moment of the dependent variable can be determined. An advantage is, that the exact density function of the random variable has not to be known. But the FOSM cannot be used, if some correlated random variables are involved. If higher-order derivations are needed, the quality of this method is decreasing. This method calculates the distance from the mean point to the failure area in direction of the gradient.

5.2 First-Order Reliability Method

The First-Order Reliability Method (FORM) evaluates the minimal distance between mean area and failure area. As a non-linear function can have some local minima, derivations are required. This method is simple to implement, but the explicit limit state function must be known. The FORM is available for all probability distributions, but it is not simple to combinate it with numerical methods.

5.3 Point Estimate Method

The aim of the Point Estimate Method (PEM) is the evaluation of the first, second and third moment of the dependent variable. The probability density function is replaced by a discrete function with the same first three moments. So, exact knowledge about the density function is not necessary. The PEM is a weighted average method. There are several variations of that method, e. g. Harr's method, Hong's method and Zhou's and Nowak's method. The original method has been developed by Rosenblueth in the 1970s.

5.4 Monte Carlo Simulation

The Monte Carlo Simulation (MCS) is a numerical method to calculate the failure probability. The basic idea is to use random samples to approximate the exact result. An advantage is the simplicity of the algorithm. Without MCS the numerical handling of complex problems is very impractical, because the number of arithmetical operations is high. A disadvantage is the very low accuracy with reasonable computing time. For every calculated decimal place, the number of required calculations has to be increased by the factor 100. The number of required calculation steps is inversely proportional to the probability of failure.

5.5 Example: Point Estimate Method

A slope in silty sand is given: the height is 10 m and slope angle is 1V : 2H. Several direct shear tests on soil samples gave the following soil parameters:

	Density [kN/m^3]	Friction angle [º]	Cohesion [kPa]
Mean value μ	19	25	5
Standard Deviation σ	-	3	2

Laboratory testing showed normal distributed values for friction angle and cohesion. Density is assumed to be constant, see Fig. 5.5.1.

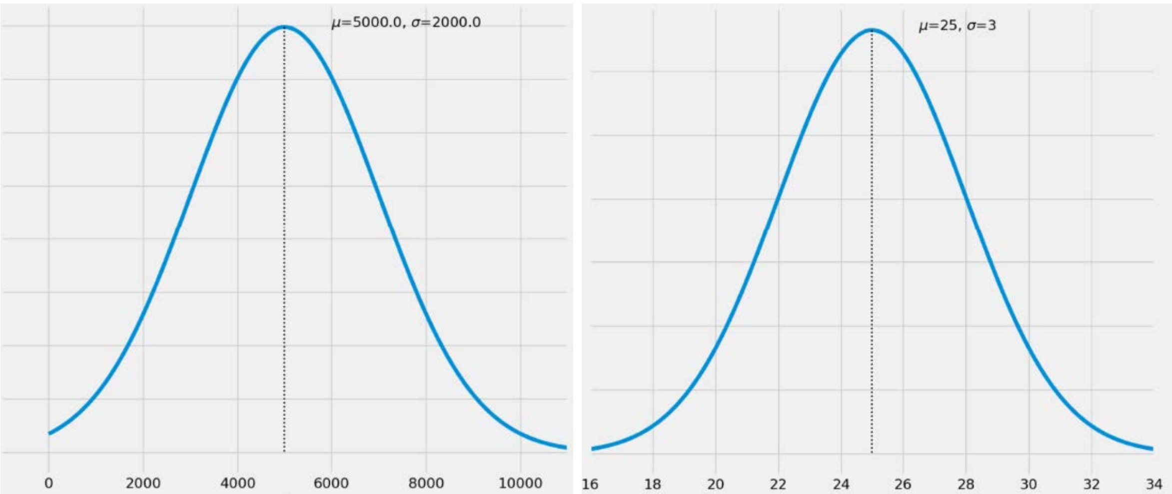


Fig. 5.5.1: Normal distribution of cohesion (left) friction angle (right) of the silty sand soil material

The geometrical setup is shown in Fig. 5.5.2. The material behaviour is ruled by the Mohr-Coulomb constitutive model and the elastic parameters are: Young's modulus 50 MPa and Poisson's ratio 0.25.

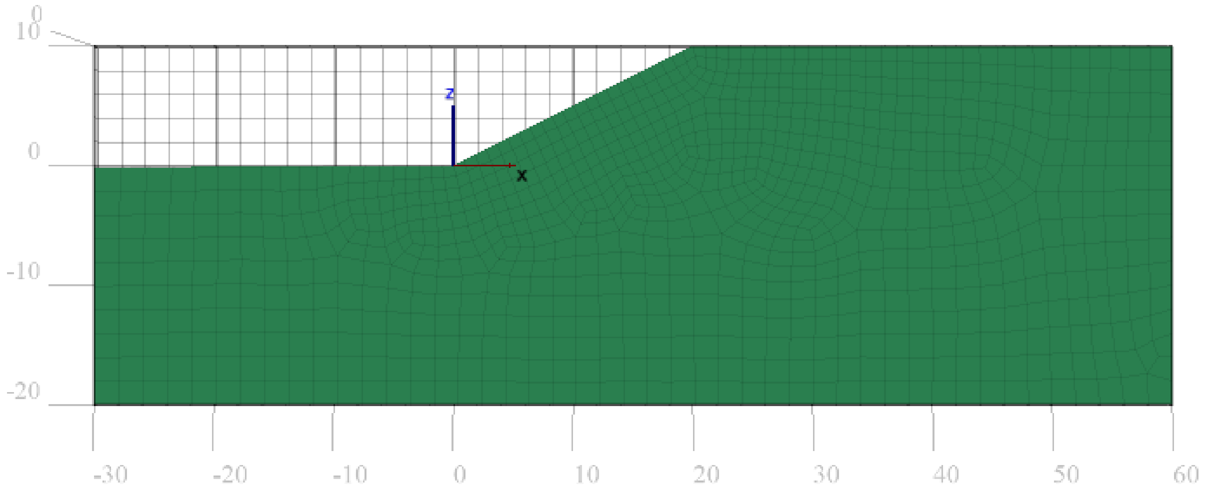


Fig. 5.5.2: Model of the slope (1V : 2H)

The PEM (Rosenblueth) is now used to obtain the probability of failure. Two soil parameters (n=2) should have random distribution. According to Rosenblueth's formula ($m = 2^n$), the numbers of required evaluations is m=4. Both, friction angle and cohesion are random parameters. They are normal distributed, so the realization points are located at $\mu \pm \sigma$.
The following table indicates the combinations of parameters resulting in 4 evaluations. The resulting Factor-of-Safety (FOS) for each evaluation is indicated as well.

Evaluation	Cohesion [kPa]	Friction angle [º]	Factor of safety
1	3	22	1.14
2	3	28	1.43
3	7	22	1.39
4	7	28	1.71

Figure 5.5.3 shows the result of one of the four evaluations.

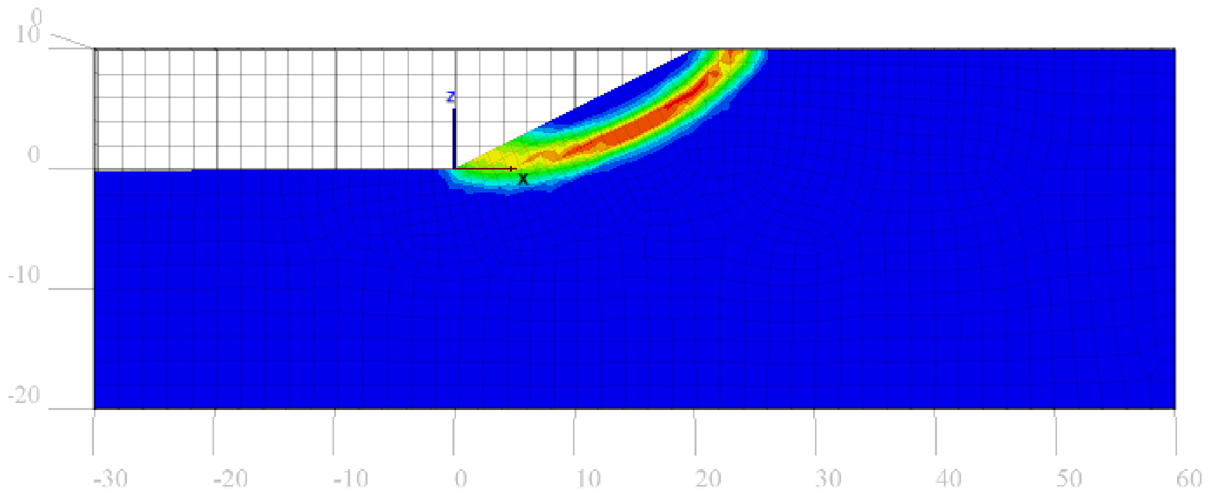


Fig. 5.5.3: Failure mode of the slope, indicating shear bands

Fig. 5.5.4 shows analysis results. The mean value of FOS is $\mu = 1.42$ with a standard deviation $\sigma = 0.2303$, resulting in a probability of failure of 3.4%.

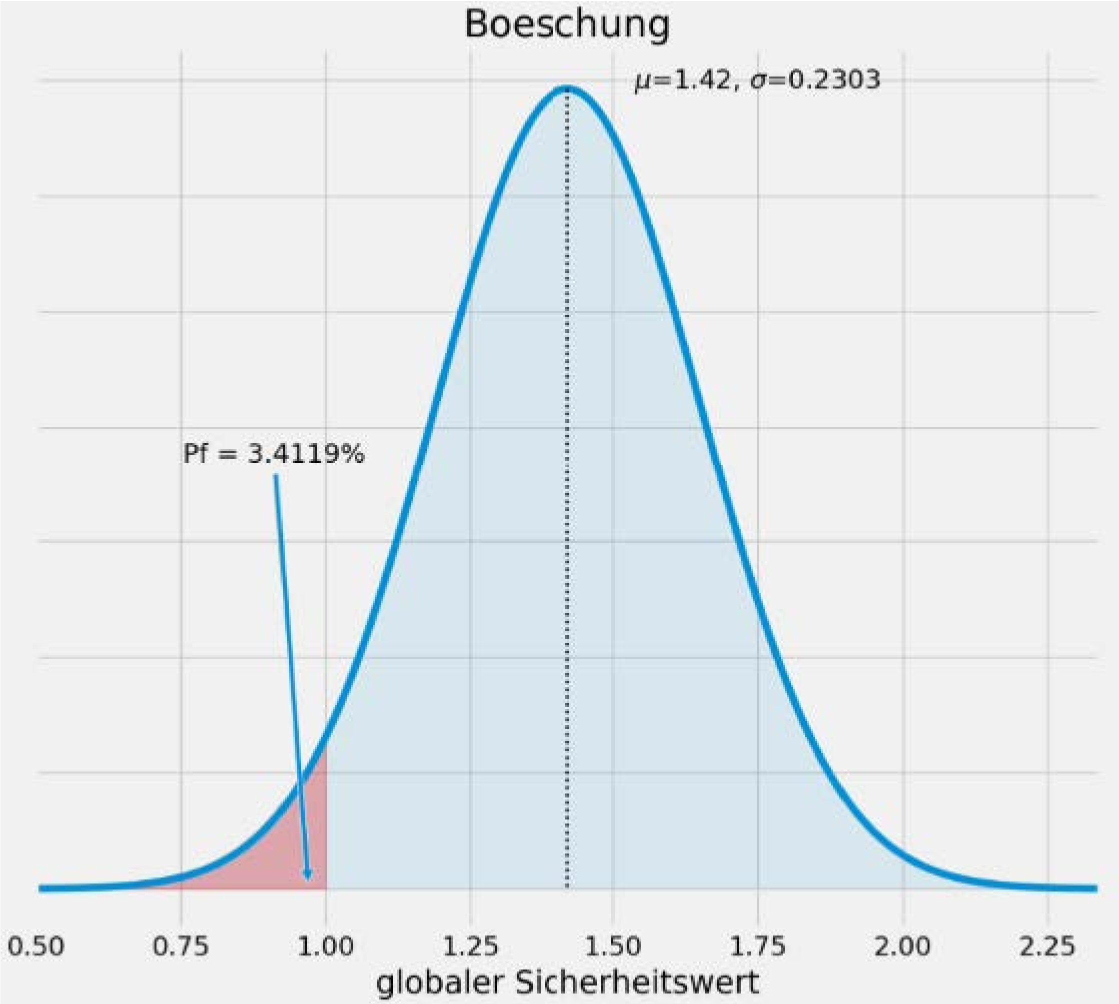
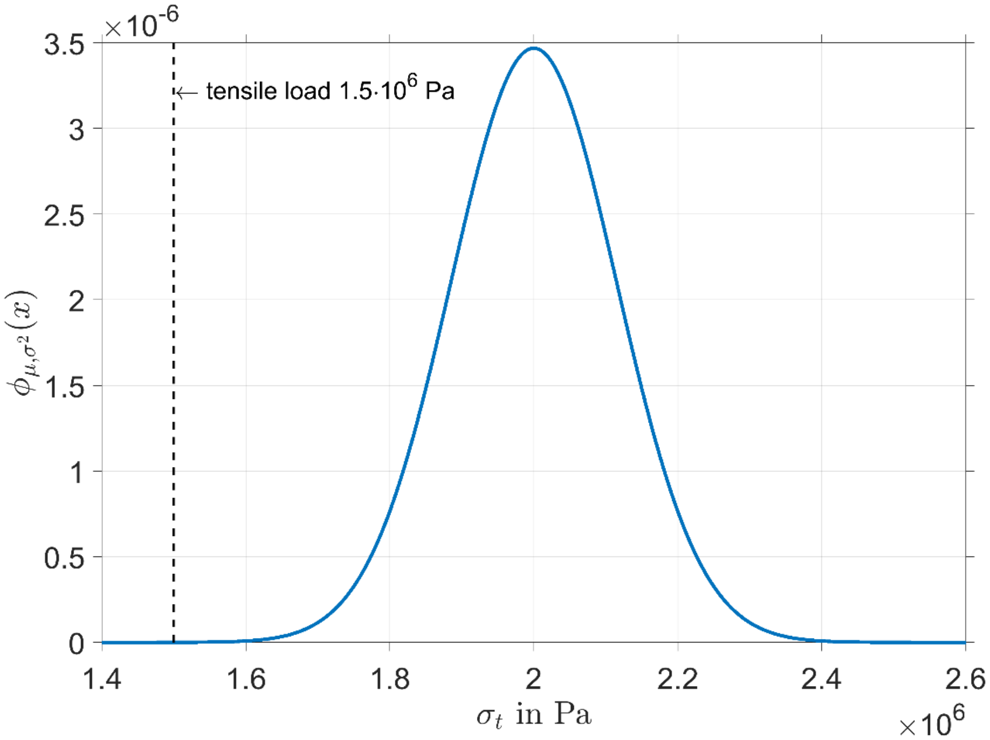


Fig. 5.5.4: Normal distributed FOS result with probability of failure (Pf)

5.6 Example: Monte Carlo Simulation

The following simple example illustrates the general procedure. With the Monte Carlo Simulation, the failure probability of a rock sample under uniaxial tensile load should be calculated. Within the sample the parameters, e. g. tensile strength, Young's Modulus, Poisson's ratio, friction angle and cohesion, are not constant. A random value is determined for each parameter and each element (of the simulated model) by a distribution function. The Monte Carlo simulation calculates several hundred or thousand different constellations of the parameters. Then the probability of failure is determined. Exemplary, in Fig. 5.6.1, a normal distribution density function for tensile strength can be seen. In the presented case, the uniaxial tensile load is 1.5 MPa. So the probability, that the tensile strength of an element in a sample is lower than the tensile load, is very low (see Fig. 5.6.1). Fig. 5.6.2 shows the tensile strength distribution within one sample determined by this normal distribution and the resulting crack obtained by a uniaxial tensile test. The macroscopic tensile fracture will most likely occur in a region of low microscopic tensile strength values. Fig. 5.6.3 shows the failure probability calculated by the Monte Carlo simulation. The probability of failure is the quotient of the number of already failed samples and the number of already calculated samples. Therefore, Fig. 5.6.3 shows an oscillating curve which slightly fluctuates around one value as the number of samples increases. The more samples are used, the more accurate is the determined probability of failure. In the presented case, an exact prognosis about the probability of failure is possible after using about 3000 samples.



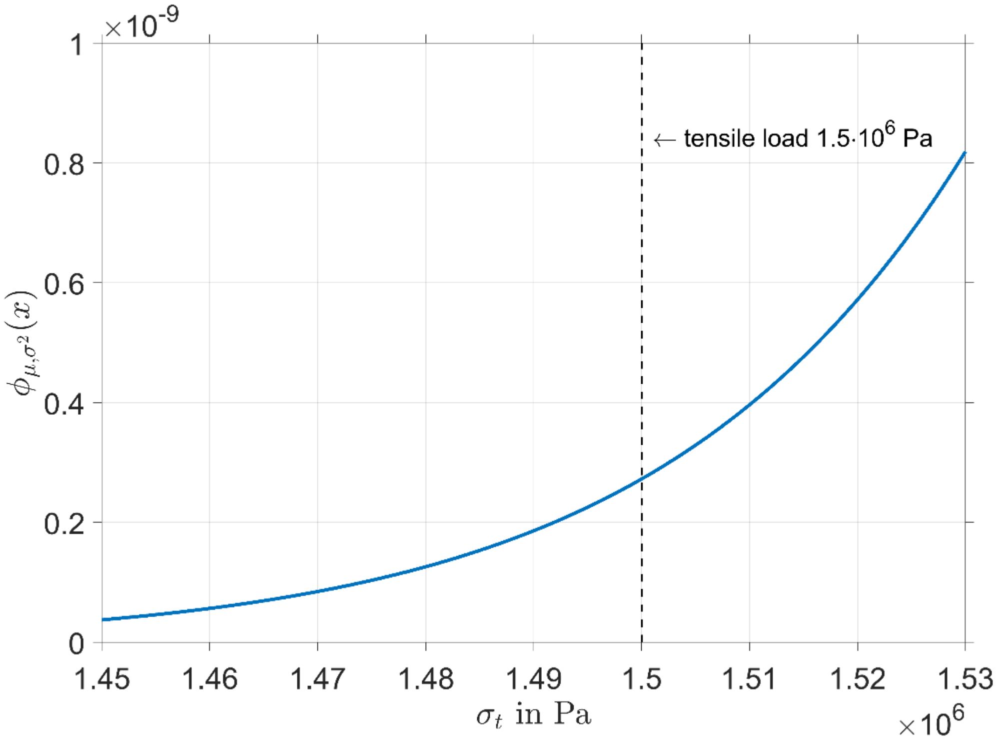


Fig. 5.6.1: Normal distribution density function for tensile strength [Pa] with mean $\mu = 2 \cdot 10^6$ Pa and standard deviation $\sigma = 1.15 \cdot 10^5$ Pa

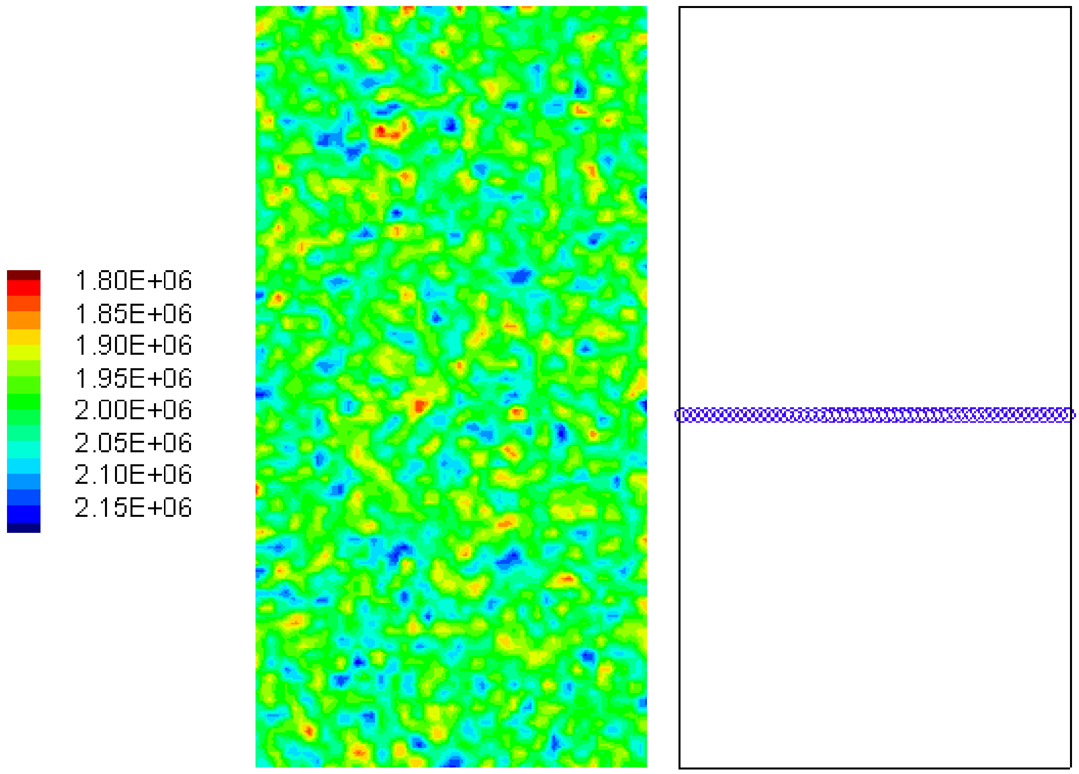


Fig. 5.6.2: Tensile strength (in Pa) inside one of the samples (left) and the crack obtained by the corresponding simulation (right)

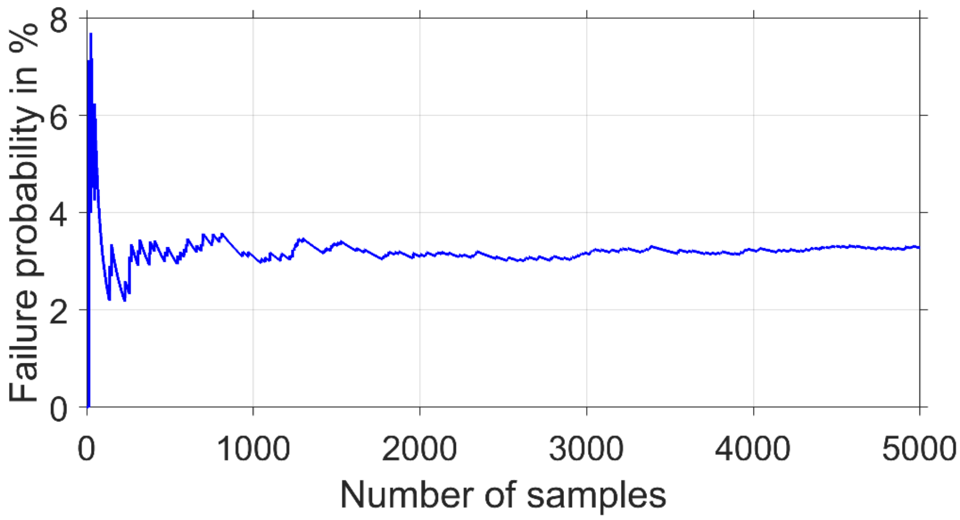


Fig. 5.6.3: Failure probability calculated by the Monte Carlo simulation

6 Risk management for major geotechnical projects

For major geotechnical projects systematic risk management is strictly recommended and increasingly required by clients. Referring to chapter 3 the basics of risk based project management with focus on costs are given here. Please note also our E-book "Geotechnical Building Information Modelling", which describes how risk management is integrated into BIM.

The total costs (BRV) can be subdivided into 4 main components like illustrated in Fig. 6.1:

- Base costs (B)
- Additional costs (Z)
- Cost to cover risks (R)
- Costs to cover escalation (V)

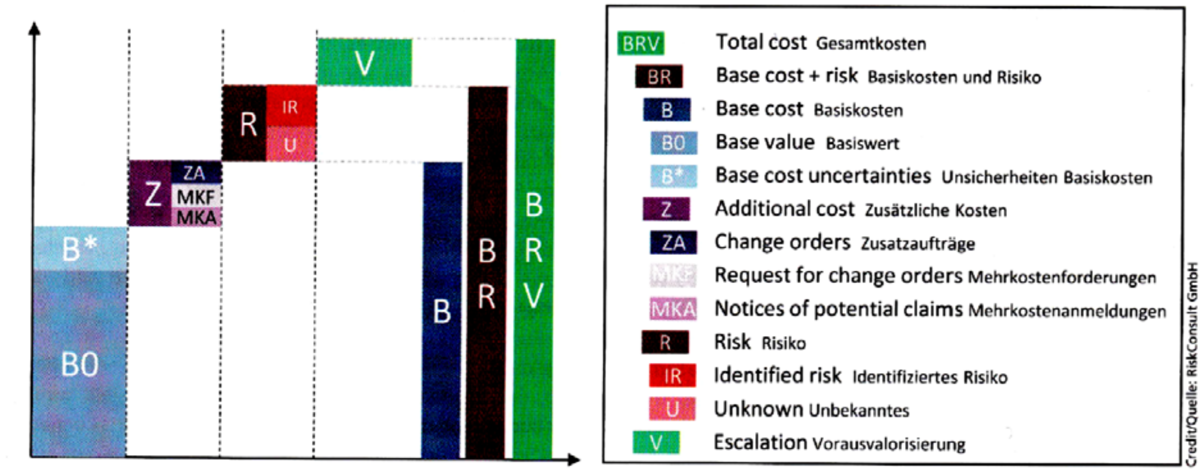


Fig. 6.1: Typical cost component structure (Sander et al. 2021)

Base costs consider the costs at this stage under the assumption that the project itself runs smoothly. Additional costs cover expectable additional costs. Risk costs cover potential addition costs due to threats / accidents etc. which have to be expected with a certain lower probability. Escalation costs cover cost for increasing prices in the future.

Fig. 6.2 illustrates how risk management, project costs and budget planning interact. Budget should be slightly greater than contract costs to be able to cover risk costs. The diagram on the right hand side in Fig. 6.2 illustrates a situation where the budget will be underrun with a probability of 80%, but overrun with a probability of 20%.

Note, that risk analysis can be done deterministic (classical approach) or probabilistic (up-to-date approach). The probabilistic approach quantifies uncertainties, which allows a more profound decision making and management (see Fig. 6.3).

A practical implementation of risk based management for road construction is presented by Riemann & Sander (2020).

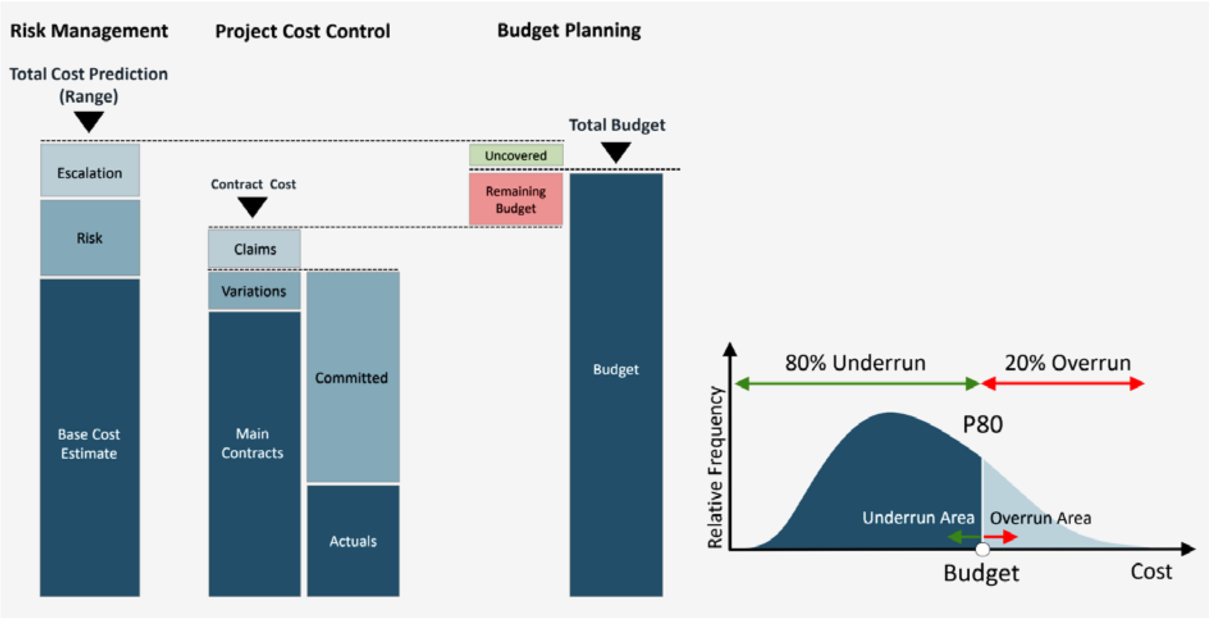


Fig. 6.2: Interaction of risk management, project costs and budget planning (Sander et al. 2021)

	Deterministic method	Probabilistic method
Input	The deterministic risk assessment uses a single number for consequence as a descriptive statement including conservative assumptions, and risk is expressed by the probability of occurance multiplied by the impact of the particular hazard.	The probabilistic assessment of risk requires at least one number or – for an entirely probabilistic modelling – a PDF for the probability of occurrence and several values for the impact (e.g. minimum, most likely and maximum) expressed as distribution functions, therefore including uncertainty.
Result	A simple mathematical addition yields the aggregated consequence for all risks (point value calculation). This results in an expected consequence for the aggregated risks but does not adequately represent the bandwidth (range) of the aggregated consequences. The deterministic calculation can be supplemented with upper and lower bounds (different model setups) to show the sensitivity of the input on the results using a sensitivity analysis, which are per se separate deterministic calculations.	Simulation methods e.g. Monte Carlo simulation produce a bandwidth (range) of aggregated natural hazard risks as a probability distribution based on thousands of coincidental but realistic scenarios (depiction of realistic risk combinations). The method allows for an explicit consideration and treatment of all types of reducible uncertainty.
Qualification	Results (monetary value or fatality per time unit) are displayed as a single sharp number, which, in itself, does not have an associated probability.	Results are displayed using probability distributions, which allow for a value-at-risk (VaR) interpretation for each value within the bandwidth (range).

Fig. 6.3: Comparison between deterministic and probabilistic risk analysis (Oberndorfer et al. 2020)

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