



<http://www.diva-portal.org>

This is the published version of a paper published in *Structural Safety*.

Citation for the original published paper (version of record):

Spross, J., Johansson, F. (2017)

When is the observational method in geotechnical engineering favourable?.

Structural Safety, 66: 17-26

<https://doi.org/10.1016/j.strusafe.2017.01.006>

Access to the published version may require subscription.

N.B. When citing this work, cite the original published paper.

Permanent link to this version:

<http://urn.kb.se/resolve?urn=urn:nbn:se:kth:diva-192821>



When is the observational method in geotechnical engineering favourable?



Johan Spross*, Fredrik Johansson

Division of Soil and Rock Mechanics, KTH Royal Institute of Technology, SE-100 44 Stockholm, Sweden

ARTICLE INFO

Article history:

Received 10 April 2016

Received in revised form 21 January 2017

Accepted 21 January 2017

Keywords:

Observational method
Reliability-based design
Decision analysis
Eurocode 7

ABSTRACT

The observational method in geotechnical engineering is an acceptable verification method for limit states in Eurocode 7, but the method is rarely used despite its potential savings. Some reasons may be its unclear safety definition and the lack of guidelines on how to establish whether the observational method is more favourable than conventional design. In this paper, we challenge these issues by introducing a reliability constraint on the observational method and propose a probabilistic optimization methodology that aids the decision-making engineer in choosing between the observational method and conventional design. The methodology suggests an optimal design after comparing the expected utilities of the considered design options. The methodology is illustrated with a practical example, in which a geotechnical engineer evaluates whether the observational method may be favourable in the design of a rock pillar. We conclude that the methodology may prove to be a valuable tool for decision-making engineers' everyday work with managing risks in geotechnical projects.

© 2017 The Authors. Published by Elsevier Ltd. This is an open access article under the CC BY license (<http://creativecommons.org/licenses/by/4.0/>).

1. Introduction

In geotechnical engineering, much construction work is performed under significant uncertainty. Nevertheless, the acceptability of structural performance must be verified. The relevant limit states are typically verified before construction has started with either deterministic or probabilistic calculation methods. However, when the structural behaviour is particularly hard to predict, geotechnical engineers may apply an approach known as the observational method, which was first defined by Peck [1]. In the 1980s, a similar approach known as “active design” was successfully applied in Sweden [2]. Today, the observational method is—with some modifications from Peck's original version—an acceptable verification method for limit states in Eurocode 7 [3], which is the European standard for the design of geotechnical structures.

The benefit of applying the observational method instead of a conventional design approach is its potential for savings in time and money, while continuously maintaining safety [1]. The essence of the method includes preparing (1) a preliminary design based on what is known at the time, (2) a monitoring plan for verifying that the structure behaves acceptably during construction and (3) a

contingency action plan that is put into operation if defined limits of acceptable behaviour are exceeded. To be successful, the preliminary design must be chosen such that it avoids the use of costly and time-delaying contingency actions with sufficiently high probability. Over the years, successful applications of the observational method and discussions thereof have been reported [4–17].

However, the above examples seem to be exceptions: despite the potential savings, the observational method is not common practice, at least not in accordance to its formal definition. In fact, in a symposium arranged by the Institution of Civil Engineers for a special issue of Geotechnique on the observational method, it was found that further clarification of how to apply the method properly was needed, in particular, with respect to safety aspects [18]. Therefore, the concerns reported by Powderham [19] regarding uncomfortably low safety margins may not be surprising, especially as the advantage of the method is to allow less conservative designs than other design approaches. Recently, Harrison [20] and Bozorgzadeh and Harrison [21] identified a need for further elaboration of the observational method in Eurocode 7 for rock engineering applications. On this topic, Spross et al. [22] highlighted that Eurocode 7 does not explicitly require any safety margin for the completed structure, which may lead to an arbitrary safety at best and unknown safety at worst. In addition, there is currently no general guideline for establishing when the observational method is more favourable than other available conventional design methods.

* Corresponding author.

E-mail addresses: johan.spross@byv.kth.se (J. Spross), fredrik.johansson@byv.kth.se (F. Johansson).

In this paper, we challenge the unclear definition of the observational method by suggesting that design with this method should be made under a reliability constraint. Based on this constraint, we propose a probabilistic optimization methodology that aids the decision-making engineer in choosing between the observational method and conventional design. The methodology suggests an optimal design after comparing the expected utilities of the considered design options. The methodology also addresses an application problem in the observational method [11,22]: how to satisfy the Eurocode 7 requirement “[to show] that there is an acceptable probability that the actual behaviour will be within the acceptable limits” [3]. Here, the “acceptable probability” refers to the probability of not needing to put contingency actions into operation.

The paper is structured as follows. The methodology is first presented in general terms. Its applicability is then shown with an illustrative example, in which two available design options of a rock pillar are analysed to find the more favourable one. Finally, the importance of the suggested reliability constraint and its relation to the limits of acceptable behaviour are discussed. In addition, remarks are made on the practical difficulties of applying the methodology in more complex cases.

2. Applied definition of the observational method

When the observational method is referred to in geotechnical engineering, there is sometimes confusion about its meaning. Some use the term for any design that is mainly based on observations, while others use it only for designs following a strict definition [18]. We have the latter view, and this paper follows the definition in Eurocode 7 [3]. This definition is quoted below, in which “P” indicates principles that must not be violated:

- (1) “When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as ‘the observational method’, in which the design is reviewed during construction.
- (2) P The following requirements shall be met before construction is started:
 - acceptable limits of behaviour shall be established;
 - the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
 - a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
 - the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
 - a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.
- (3) P During construction, the monitoring shall be carried out as planned.
- (4) P The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded.
- (5) P Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.”

3. Bayesian decision framework for the observational method

The proposed methodology is based on classic Bayesian decision analysis, which assumes that the optimal decision maximizes the expected utility [23,24]. Bayesian decision analyses generally include four phases (Fig. 1): (1) a decision to perform an experiment or measurement, e , (2) an outcome, z , of the performed e , (3) a decision to take an action, a , based on z , and (4) the occurrence of an event, θ . Bayesian decision analyses have previously been shown to be useful in geotechnical engineering [25–29], and van Baars and Vrijling [30] have briefly discussed how such analyses can be applied together with the observational method. The decision analysis in this paper includes reliability assessments and Bayesian updates to prior assumptions of relevant parameters with measurements; each aspect is discussed in the following subsections.

3.1. Limiting the observational method with a target reliability

In general, the performance of a structure consisting of j components may be described by the combination of their limit state functions $G_j(\mathbf{X})$, where \mathbf{X} is a vector containing all relevant basic variables. On a component level, the event of unsatisfactory performance (hereafter denoted “failure”, F , for simplicity) is defined as $F_j = \{G_j(\mathbf{X}) \leq 0\}$, and its complementary event—i.e., the event of satisfactory performance—is \bar{F}_j . A measure of the probability of failure of the complete system, p_F , is given by the multidimensional integral

$$p_F = \int_{\Omega} f_{\mathbf{X}}(\mathbf{x}) d\mathbf{x}, \quad (1)$$

where \mathbf{x} is the realization of \mathbf{X} , $f_{\mathbf{X}}(\mathbf{x})$ is the joint probability density function of \mathbf{X} , and Ω is the region for the failure event, defined by

$$\Omega \equiv \bigcup_k \bigcap_{j \in c_k} \{G_j(\mathbf{X}) \leq 0\}. \quad (2)$$

This formulation of the failure region implies that the structure is seen as a system of j components, and failure of the structure occurs when some combination c_k of these components fails [31]. The p_F is frequently presented in terms of the reliability index, β , given by

$$\beta = -\Phi^{-1}(p_F), \quad (3)$$

where Φ^{-1} is the inverse of the standard normal distribution function.

Using a conventional design method, the suggested design of the structure should, prior to its realization, meet an acceptance criterion, which is usually defined by a design code. For probabilistic design, the criterion is defined by a target probability of failure, p_{FT} , such that $p_F^{(0)} \leq p_{FT}$, where the superscript $\{0\}$ indicates that the assessment is based on prior information, e.g., from pre-investigations and engineering judgement.

From a probabilistic view, the “realization” implies that the built structure is one realization of many possible outcomes [32]. The realization causes the aleatoric uncertainties of the structural

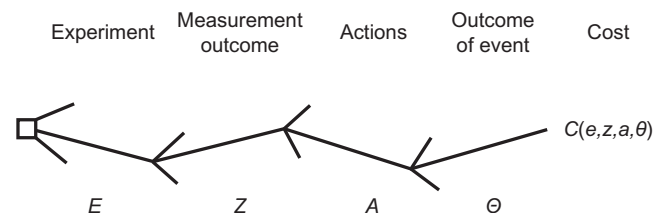


Fig. 1. General decision tree.

design to transform into epistemic uncertainties of an existing structure. The difference between the two categories of uncertainty is that, by definition, only epistemic uncertainties can be updated [33,34].

For the observational method, the design is only preliminary, and it will be altered if performed measurements or other observations indicate that the structure violates a predefined warning level, which in Eurocode 7 is known as the limit of acceptable behaviour (see Section 2). Before construction has started, the preliminary design has a probability of failure, $p_F^{(0)}$, comparable to that of the conventional design. Note, however, that $p_F^{(0)}$ for the preliminary design may exceed p_{FT} , as observations and contingency actions are used to improve the reliability during construction. This makes the distinction between aleatoric and epistemic uncertainties essential for the observational method, as only epistemic uncertainties can be reduced to improve the structural reliability. If the observations do not sufficiently reduce the epistemic uncertainties to satisfy p_{FT} , further action must be taken to ensure safety: e.g., strengthening the structure.

Note that the probabilities of failure in this paper should be calculated and interpreted in the Bayesian sense (i.e., not frequently or nominally). This implies that a calculated probability of failure is the best representation of the degree of belief in the occurrence of structural failure [35]. For further discussions on reliability-based design in geotechnical engineering, see e.g. [36–39].

3.2. Pre-posterior decision analysis

The objective of the proposed methodology is to find an optimal design after comparing the expected utility of applying the observational method with the expected utility of other available design options. As an example, the observational method includes the respective costs of a preliminary design, a measurement program that is more elaborate than usual, and the design and installation of possible contingency actions. However, regardless of the chosen design approach, the completed structure must satisfy the safety requirement established by society. Therefore, the objective is to identify the design that maximizes the expected utility while satisfying the reliability constraint, $p_F \leq p_{FT}$. We assume that all utility can be expressed in monetary terms such that utility is inversely proportional to the cost. However, the proposed framework can be straightforwardly adjusted for other preferences, if they can be expressed by utility functions (e.g., Ditlevsen and Madsen [40] and Straub [41]).

This decision problem is known as a pre-posterior decision analysis [23,24]. The additional information Z that is provided through monitoring or other observations will reveal whether the behaviour is acceptable or not. In the latter case, further action is needed. In general, any measurement action or contingency action may be followed by additional actions before the design is accepted. However, note that Eurocode 7 (see Section 2) requires that the actions are planned in advance and that the plans prespecify which observed z that activates a certain action. In the context of Bayesian decision analysis, this is known as a decision rule: a is a function of z ; i.e., we have that $a = f(z)$. Such actions may imply a modification of a limit state function from $G(\mathbf{X})$ into $G_{\text{mod}}(\mathbf{X})$.

The considered system states of the structure are collected in a set, $\Theta = \{\theta_1, \dots, \theta_m\}$. If only failure and non-failure behaviour is of interest, the events are $\theta_1 = F$ and $\theta_2 = \bar{F}$. The set $\mathbb{E} = \{e_1, \dots, e_l\}$ contains all possible design solutions, where each solution, e_i , includes all relevant design considerations: e.g., geometrical considerations, extent of planned monitoring and design of required contingency action plans (see Section 2). The optimal design solution to prepare is then given by

$$e_{\text{opt}} = \arg \min_{\mathbb{E}} \left\{ \sum_{j=1}^n P(z_j | e_i) C(e_i, z_j, a_j) | p_F^{\{e_i, z_j, a_j\}} \leq p_{FT} \right\}, \quad (4a)$$

where $P(z_j | e_i)$ is the probability of having the measurement outcome z_j when some design solution e_i is applied, and $C(e_i, z_j, a_j)$ describes the expected cost of each possible design outcome and is given by

$$C(e_i, z_j, a_j) = \sum_{k=1}^m C(e_i, z_j, a_j, \theta_k) P(\theta_k | e_i, z_j, a_j), \quad (4b)$$

where $C(e_i, z_j, a_j, \theta_k)$ is the expected cost of event θ_k (including the cost of executing the design and contingency actions that are related to that event), and $P(\theta_k | e_i, z_j, a_j)$ is the probability of event θ_k occurring given the executed design and contingency actions and taking into account the additional information that is expected to be gained. The constraint $p_F^{\{e_i, z_j, a_j\}} \leq p_{FT}$ in Eq. (4) ensures that the probability of failure is less than or equal to the target probability of failure after executing the design and contingency actions, and taking the additional information into account. Goulet et al. [42] recently suggested a similar reliability constraint in an optimisation framework for sequences of measurement and intervention actions for existing structures; though, the concept dates back to the 1980s (e.g., [43]).

If e_{opt} only contains a design decision and no plans for measurements or observations (with accompanying contingency action plans), the optimal design is of a conventional type. A decision tree for the choice between conventional design and the observational method is presented in Fig. 2.

Typically, one of the prepared contingency action plans in e_{opt} will be a drastic change in design that is put into operation for extremely unfavourable monitoring outcomes, because the reliability constraint must be satisfied for all possible monitoring results. An example is replacing the structure with a very robust design solution that satisfies p_{FT} without any need for further actions. This may be seen as a worst-case scenario. However, such costly actions must be sufficiently unlikely to be needed; otherwise, they will not be a part of e_{opt} after evaluating Eq. (4).

The expected cost associated with implementing the design described by e_{opt} is

$$C_{\text{opt}} = \sum_{j=1}^n P(z_j | e_{\text{opt}}) C(e_{\text{opt}}, z_j, a_j), \quad (5)$$

the components of which are evaluated as in Eq. (4), but for the specific design solution e_{opt} .

3.3. Reliability updates from additional information

To find e_{opt} , the probabilities of the respective system states, i.e., $P(\theta_k)$, must be evaluated conditionally on the information, Z , gained from possible outcomes of monitoring. In general, measurements and observations can be used to improve reliability in a number of ways: e.g., direct and indirect measurements of relevant parameters, load tests, and calibration of model errors. More examples are discussed by Goulet et al. [42]. In principle, gaining additional information implies an updated probability of failure:

$$p_{F|Z} = \int_{\Omega_Z} f_{\mathbf{X}|Z}(\mathbf{x}) d\mathbf{x}, \quad (6)$$

where Ω_Z is the failure region of Eq. (2) updated such that the limit state functions are conditional on Z , and $f_{\mathbf{X}|Z}(\mathbf{x})$ is the updated joint probability density function given the information Z . Bayes's rule may be used to update $f_{\mathbf{X}|Z}(\mathbf{x})$:

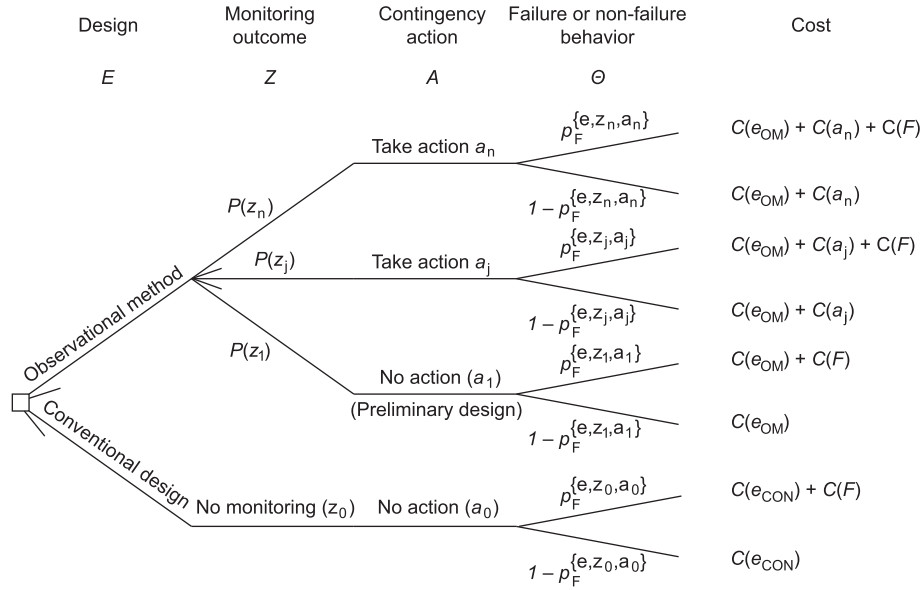


Fig. 2. Decision tree illustrating the observational method versus conventional design.

$$f_{\mathbf{X}|Z}(\mathbf{x}) = \frac{L(\mathbf{x})f_{\mathbf{X}}(\mathbf{x})}{\int L(\mathbf{x})f_{\mathbf{X}}(\mathbf{x})d\mathbf{x}} \quad (7)$$

where $L(\mathbf{x})$ is the likelihood of observing Z , given the variable \mathbf{X} . Bayesian updates have previously been used in observational method applications [9,22,44–46].

When the information, Z , is provided as monitoring data of an equality or inequality type (e.g., [47]), which both are common in observational method applications, the explicit computation of $L(\mathbf{x})$ can be avoided by obtaining $p_{F|Z}$ directly from the definition of conditional probability:

$$p_{F|Z} = \frac{P(F \cap Z)}{P(Z)}. \quad (8)$$

Equality information means that the outcome of \mathbf{X} is known to belong to a subset of the original domain, such that

$$Z = \{h(\mathbf{X}) = 0\}, \quad (9)$$

where the equality $h(\mathbf{X}) = 0$ can be exemplified by measuring a specific deformation, load, or crack size. In contrast, inequality information implies that

$$Z = \{h(\mathbf{X}) \leq 0\}, \quad (10)$$

which can be exemplified by monitoring of a parameter X_i showing that it does not exceed a predefined alarm level, $x_{\text{alarm},i}$. If inequality or equality information is available, Eq. (8) may be computed with structural reliability methods [40,41,47]. A geotechnical engineering application was discussed by Papaioannou and Straub [48].

For example, if we have inequality information showing that a parameter X_1 will not exceed the predefined $x_{\text{alarm},1}$, we know that the outcome of \mathbf{X} will belong to the set

$$\mathbb{H}_Z = \{\mathbf{x} | x_1 < x_{\text{alarm},1}\}, \quad (11)$$

which implies that $p_{F|Z}$ is evaluated with the distribution function of X_1 truncated at $x_{\text{alarm},1}$ (Fig. 3). Note that considering only the inequality information of a measurement does not use the full information of the measured value; however, updating with inequality information may be preferable because of its simplicity. Moreover, the effect of the additional information on $p_{F|Z}$ is dependent on the degree of uncertainty in the observation; for example,

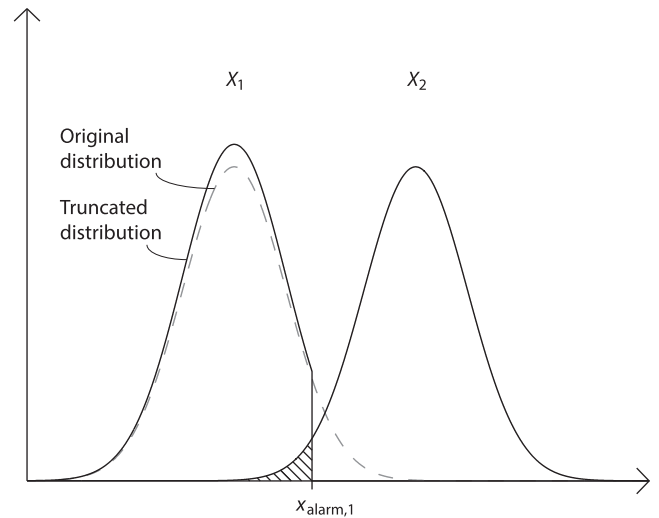


Fig. 3. Knowing that X_1 does not exceed $x_{\text{alarm},1}$, the p_F is significantly reduced.

expecting a large measurement error in the monitoring restrains the improvement of $p_{F|Z}$ [49].

3.4. Establishment of acceptable limits of behaviour

A crucial aspect of the observational method is how to establish the alarm levels, $\mathbf{x}_{\text{alarm}} = [x_{\text{alarm},1}, \dots, x_{\text{alarm},n}]$, referred to as acceptable limits of behaviour in Eurocode 7 (see Section 2). If exceeded, they require that contingency actions be put into operation to ensure safety. Because of the reliability constraint, $P(G < 0 | \mathbf{x} \leq \mathbf{x}_{\text{alarm}}) = p_{FT}$ the $\mathbf{x}_{\text{alarm}}$ must be established such that the potential monitoring outcome of the preliminary design at most corresponds to the target reliability of failure:

$$P(G < 0 | \mathbf{x} \leq \mathbf{x}_{\text{alarm}}) = p_{FT}. \quad (12)$$

Having defined $\mathbf{x}_{\text{alarm}}$, the “probability that the actual behaviour will be within the acceptable limits” (in the Eurocode 7 requirements in Section 2) can be calculated as $P(\mathbf{x}^{(0)} \leq \mathbf{x}_{\text{alarm}})$, which is an example of $P(Z)$ with Z being of the inequality type (Eq. (10)). Finding that

e_{opt} after evaluating Eq. (4) includes a measurement plan with the alarm limits \mathbf{x}_{alarm} , and a corresponding contingency action plan, shows that $P(\mathbf{x}^{(0)} \leq \mathbf{x}_{alarm})$ is “acceptable”; thereby, this Eurocode requirement is satisfied. On the other hand, if contingency actions are expected to be needed with too high probability with respect to their corresponding cost, e_{opt} will consist of the conventional design option. Then, the observational method does not provide the more favourable option. This concept is later exemplified in Section 4.4.

4. Practical example of optimal design method decision-making

To illustrate the proposed methodology for the identification of the optimal design method, we present a practical example in which the performance of an underground rock pillar is evaluated. A decision-making geotechnical engineer evaluates which design method is more favourable; making a conventional design or applying the observational method. The relevant limit state functions are described in Section 4.1. The prior probability of failure based on information available at this stage is evaluated in Section 4.2. The available design options and the decision-making process are discussed in Section 4.3. Measurements of the vertical deformation of the pillar are introduced in Section 4.4. The method for establishing a limit of acceptable behaviour that satisfies the reliability constraint is discussed in Section 4.5. Lastly, calculation results of the predicted costs of the respective design options are presented in Section 4.6.

4.1. Case description: Vertical deformation of a rib pillar

4.1.1. Limit state function

The structure to be excavated is a very long rib pillar (Fig. 4). The considered limit state concerns the vertical strain of the pillar, ε_1 , which may not exceed a maximum allowable strain, $\varepsilon_{1,max}$:

$$G(\mathbf{X}) = \varepsilon_{1,max} - \varepsilon_1. \quad (13)$$

The rib pillar is very long; hence, we assume plane strain conditions [50]:

$$\varepsilon_1 = \frac{1 - \nu^2}{E_r} \sigma_1 - \frac{\nu(1 + \nu)}{E_r} \sigma_3, \quad (14a)$$

$$\varepsilon_3 = \frac{1 - \nu^2}{E_r} \sigma_3 - \frac{\nu(1 + \nu)}{E_r} \sigma_1, \quad (14b)$$

where ε_1 and ε_3 are the strains in vertical and horizontal direction, respectively (defined positive for compression), ν is the Poisson's ratio of the rock mass, σ_1 is the vertical load of the overburden, σ_3 is the confining pressure, which may be provided by rock anchors (see Section 4.1.2), and E_r is the Young's modulus of the rock mass.

The overburden is assumed evenly distributed over a large set of rib pillars [51], such that

$$\sigma_1 = \gamma h \left(1 + \frac{w_o}{w_p} \right) \psi, \quad (15)$$

where γ is the unit weight of the rock, h is the overburden, w_o is the excavation width, w_p is the pillar width, and ψ is the model uncertainty in the load distribution. In this example, we let σ_1 be increased by step-by-step increases of w_o , i.e., making the excavation wider on both sides of the pillar. However, w_p is kept constant.

The maximum allowed vertical strain is defined via the empirical Hoek–Brown failure criterion [52]; specifically, we apply the updated version presented by Hoek et al. [53]:

$$\varepsilon_{1,max} = \frac{\sigma_{1,max}}{E_r} = \frac{\sigma_3 + \sigma_{ci} \left(m_m \frac{\sigma_3}{\sigma_{ci}} + s \right)^a}{E_r}, \quad (16a)$$

where σ_{ci} is the compressive strength of the intact rock. The Hoek–Brown parameters for the rock mass are given by

$$m_m = m_i \exp[(GSI - 100)/(28 - 14D)], \quad (16b)$$

$$s = \exp[(GSI - 100)/(9 - 3D)], \quad (16c)$$

$$a = 0.5 + [\exp(-GSI/15) - \exp(-20/3)]/6, \quad (16d)$$

where m_i is a material constant, GSI is the geological strength index, and D is the disturbance factor. The rock mass is assumed

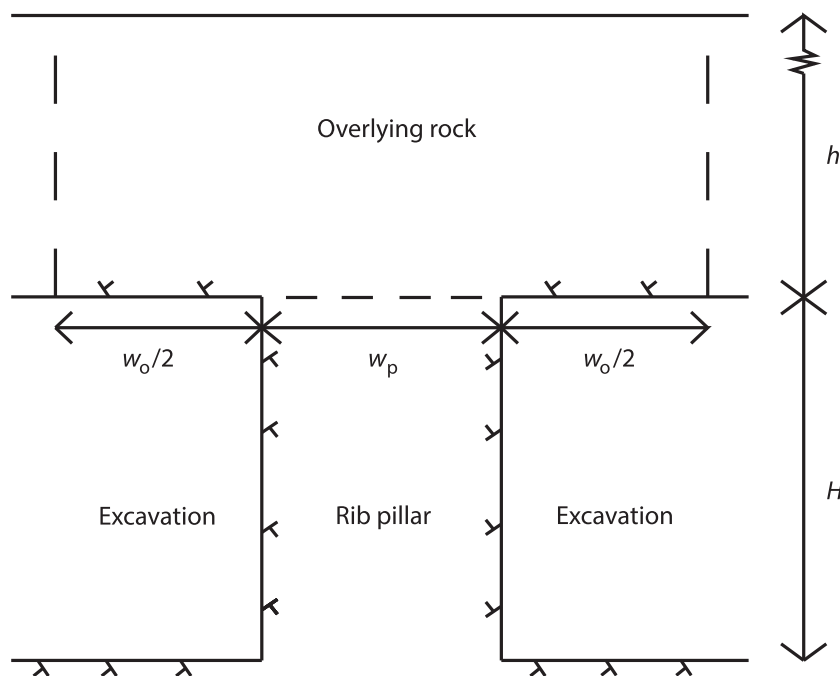


Fig. 4. Cross section of the analysed rib pillar.

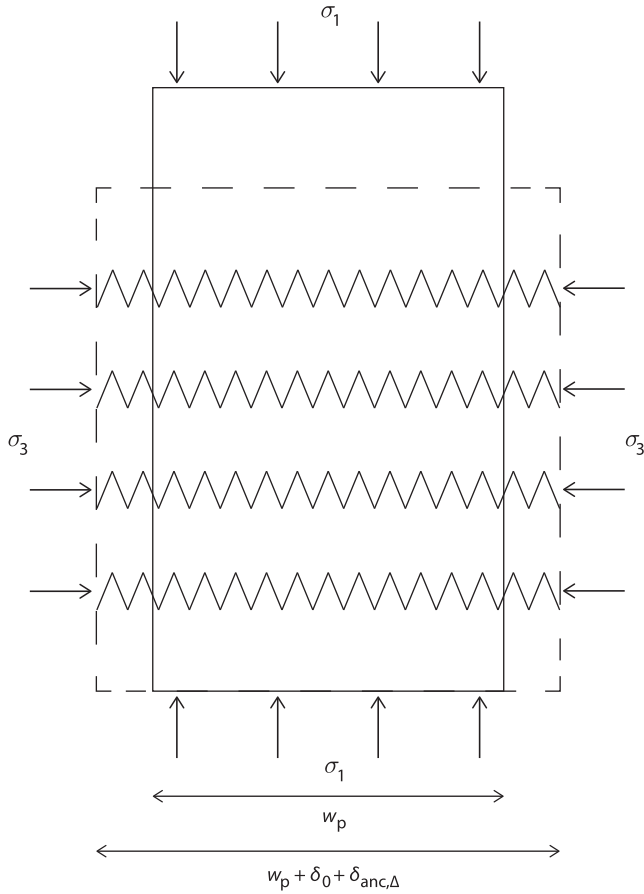


Fig. 5. Conceptual idealisation of the pillar reinforced with rock anchors, which provide confinement pressure.

undisturbed (i.e., the disturbance factor, D , is set to 0). It is also assumed that the $\varepsilon_{1,max}$ of the rock mass increases proportionally to σ_1 as σ_3 increases (i.e., constant E_r), which matches the behaviour of intact rock [50]. However, numerical simulations of fractured crystalline rock by Bidgoli et al. [54] indicate that rock masses also

show a slight increase in E_r as σ_3 increases, but this effect is disregarded here for simplicity. The strain is assumed perfectly elastic as long as $\varepsilon_{1,max}$ is not exceeded.

Inserting Eqs. (14a) and (16a) into the limit state function in Eq. (13) yields

$$G(\mathbf{X}) = \frac{\sigma_3 + \sigma_{ci}(m_m \frac{\sigma_3}{\sigma_{ci}} + s)^a}{E_r} - \left[\frac{1 - \nu^2}{E_r} \sigma_1 - \frac{\nu(1 + \nu)}{E_r} \sigma_3 \right]. \quad (17)$$

4.1.2. Limit states with and without the installation of rock anchors

The rock pillar may be constructed with or without confining rock anchors. If ground conditions are favourable, rock anchors may not be needed, because the unreinforced rock pillar, i.e., $\sigma_3 = 0$, does not violate the limit state with acceptable reliability. Then, Eq. (17) is reduced to

$$G(\mathbf{X}) = \frac{\sigma_{ci}s^a}{E_r} - \frac{1 - \nu^2}{E_r} \sigma_1 \quad (18)$$

If the ground conditions are unfavourable, rock anchors are needed to provide a confining pressure, i.e., $\sigma_3 > 0$, giving the modified limit state function $G_{mod}(\mathbf{X})$. In this case, σ_3 may be provided by two components: the prestress, $\sigma_{3,0}$, applied on the rock anchors at installation, and the additional stress, $\sigma_{3,\Delta}$, caused by the additional anchor elongation, $\delta_{anc,\Delta}$, as the vertical load increases (Fig. 5). Thus,

$$\sigma_3 = \sigma_{3,0} + \sigma_{3,\Delta} = \frac{nF_0}{A_r} + \frac{E_s A_{anc} n \delta_{anc,\Delta}}{A_r L_{anc,0}}, \quad (19)$$

where n is the number of anchors, F_0 is the preload in each anchor, A_r is the area of the pillar wall, E_s is the Young's modulus of the steel anchors, A_{anc} is the total cross-section area of the anchors, and $L_{anc,0}$ is the length of the anchors immediately after prestressing.

To find an expression for how $\delta_{anc,\Delta}$ relates to σ_1 , two equilibria are established and combined (Eqs. (20) and (21)). Immediately after prestressing the anchors, the ε_3 of the pillar will be

$$\varepsilon_{3,0} = \frac{1 - \nu^2}{E_r} \sigma_{3,0} - \frac{\nu(1 + \nu)}{E_r} \sigma_{1,0} = -\frac{\delta_0}{w_p}, \quad (20)$$

where $\sigma_{1,0}$ is the vertical load at the time of anchor installation, and δ_0 is the horizontal expansion of the pillar immediately after prestressing.

Table 1

Prior information available to or judged by the decision maker. All random variables are assumed normally distributed.

Parameter	Symbol	Unit	Mean	Coefficient of variation
<i>Rock mass properties^a</i>				
Poisson's ratio of rock mass	ν	–	0.25	
Unit weight of rock mass	γ	kN/m ³	26	0.10
Model uncertainty in load distribution	ψ	–	1	0.20
Compressive strength of intact rock	σ_{ci}	MPa	80	0.10
Young's modulus of rock mass	E_r	GPa	10	0.10
Hoeck–Brown parameter for intact rock	m_i	–	15	
Geological strength index	GSI	–	55	0.07
<i>Pillar geometry</i>				
Pillar width	w_p	m	4	
Pillar height	H	m	8	
Overburden	h	m	36	
Excavation width at anchor installation	$w_{o,0}$	m	4	
Final excavation width	w_o	m	9	
<i>Anchor properties^b</i>				
Number of rock anchors	n	–	4	
Preload in each anchor	F_0	kN	373 ^c	
Young's modulus of steel anchor	E_s	GPa	210	0.03
Cross-section area of steel anchors	A_s	mm ²	300	

^a Set to simulate average quality rock mass [55].

^b Rock anchor with 3 strands, 13 mm each.

^c Preloaded to 67% of ultimate load.

For vertical loads larger than $\sigma_{1,0}$, the anchors are elongated by $\delta_{anc,\Delta}$ and provide additional confining pressure $\sigma_{3,\Delta}$; thus,

$$\varepsilon_3 = \frac{1-\nu^2}{E_r}(\sigma_{3,0} + \sigma_{3,\Delta}) - \frac{\nu(1+\nu)}{E_r}\sigma_1 = -\left(\frac{\delta_0}{w_p} + \frac{\delta_{anc,\Delta}}{w_p}\right). \quad (21)$$

Combining Eqs. (19)–(21) and rearranging such that $\delta_{anc,\Delta}$ becomes a function of σ_1 gives

$$\delta_{anc,\Delta} = \frac{\nu(1+\nu)}{E_r}(\sigma_1 - \sigma_{1,0}) \left/ \left[\frac{1-\nu^2}{E_r} \frac{E_s A_{anc} n}{L_{anc,0} A_r} + \frac{1}{w_p} \right] \right. \quad (22)$$

The complete $G_{mod}(\mathbf{X})$ is given by inserting Eqs. (19) and (22) into Eq. (17):

$$G(\mathbf{X}) = \frac{\sigma_3 + \sigma_{ci} \left(m_m \frac{\sigma_3}{\sigma_{ci}} + s \right)^a}{E_r} - \left[\frac{1-\nu^2}{E_r} \sigma_1 - \frac{\nu(1+\nu)}{E_r} \sigma_3 \right], \quad (23a)$$

where

$$\sigma_3 = \frac{nF_0}{A_r} + \frac{E_s A_{anc} n \delta_{anc,\Delta}}{A_r L_{anc,0}}, \quad (23b)$$

in which

$$\delta_{anc,\Delta} = \frac{\nu(1+\nu)}{E_r}(\sigma_1 - \sigma_{1,0}) \left/ \left[\frac{1-\nu^2}{E_r} \frac{E_s A_{anc} n}{L_{anc,0} A_r} + \frac{1}{w_p} \right] \right. \quad (23c)$$

The effect of installing prestressed rock anchors is both an increase in $\varepsilon_{1,max}$ and a minor reduction in ε_1 .

4.2. Prior probabilities of failure of the design options

To make a prior assessment of the pillar reliability, the decision maker evaluates the probabilities of failure based on the available prior information for two situations: not installing any rock anchors and installing four rock anchors. The prior information is presented in Table 1. The p_{FT} to be satisfied is set to 10^{-3} , which corresponds to $\beta = 3.1$. The rock mass properties are set to simulate average rock quality with parameter values inspired by the suggestions in Hoek and Brown [55]. For simplicity, the random variables are assigned normal distributions and assumed uncorrelated. The coefficients of variation are assumed either known from the pre-investigation or judged by the decision maker based on the available literature (see e.g. [39,56]).

Using 1.6 million Monte Carlo simulations, the prior probability of exceeding $\varepsilon_{1,max}$ for the case with no anchors, i.e., $p_F^{(0)}\{G(\mathbf{X})\}$, is found to be 0.0046 ($\beta = 2.46$), which exceeds the acceptable p_{FT} . However, for the case with four anchors installed, $p_F^{(0)}\{G_{mod}(\mathbf{X})\}$ is negligible. Consequently, the decision maker may choose to go directly with the reinforced design. The prior probabilities of failure are illustrated in Fig. 6a.

4.3. Design and decision-making processes

There are significant uncertainties related to the prior knowledge. Therefore, applying the observational method to reduce the uncertainties may be beneficial, because additional measurements may show that the ground conditions are favourable enough to satisfy p_{FT} without having to pay for the rock anchor installation. The decision is illustrated in Fig. 7a. The optimal design described by the decision tree is evaluated with Eq. (4). The costs associated with the respective design options are presented in Table 2; the presented costs are for illustrative purposes only in this example.

In the example, the observational method is simplified to one acquirement of more information followed by either acceptance of the preliminary design or installation of a predefined number of rock anchors as a contingency action. In a more complex analysis, the installed number of rock anchors could be adjusted with

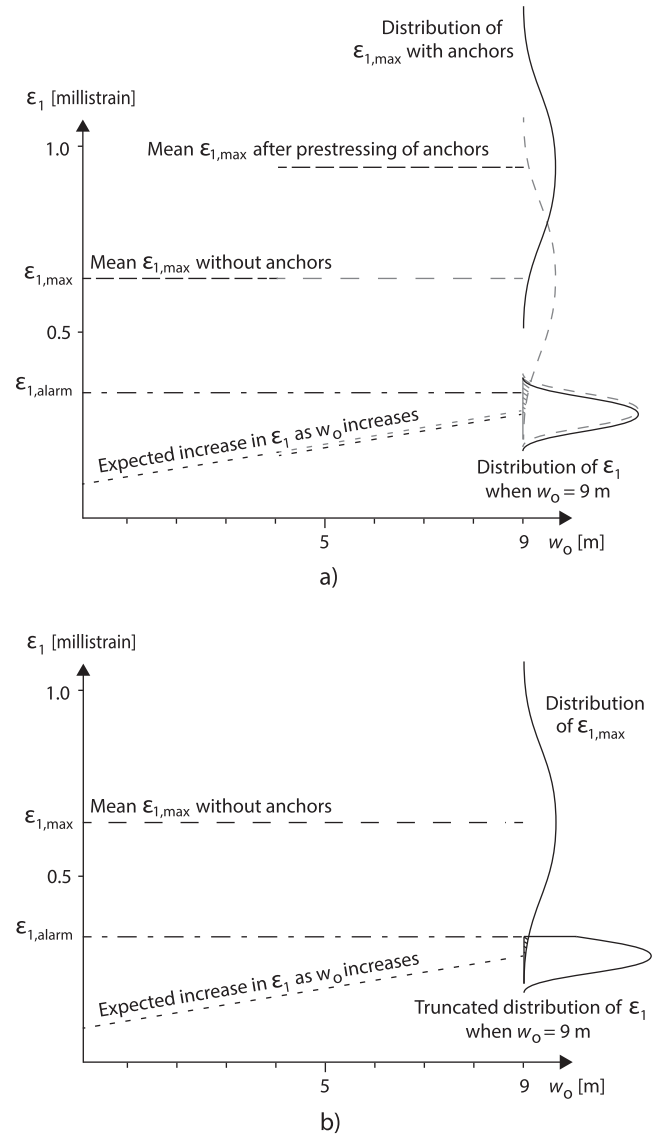


Fig. 6. a) The grey dashed distributions and the continuous black distributions represent the case without anchors and the case with installed anchors, respectively. Based on the prior information, the probability of failure, $p_F^{(0)}\{G(\mathbf{X})\}$, becomes too high when $w_0 = 9$ m, unless prestressed rock anchors are installed. b) Rock anchors are not needed, if it is shown that $\varepsilon_1 \leq \varepsilon_{1,alarm}$, because the failure probability is significantly reduced with this information.

respect to the measurement outcome, so that the cost is further reduced. Note that the reinforced design with four rock anchors is used both for the contingency action and for the conventional design.

4.4. Expected range of measurement outcomes

More information is gathered by measuring the vertical deformation of the pillar each time the excavation width is increased. For simplicity, we assume that the measurement results from the early stages of the sequential excavation are able to predict ε_1 perfectly. In the design phase, the possible outcomes of this perfect prediction are represented by the distribution of corresponding prior knowledge, $\varepsilon_1^{(0)}$, based on Eq. (14a) and Table 1. Measurement errors can straightforwardly be considered [42], but that aspect is disregarded here.

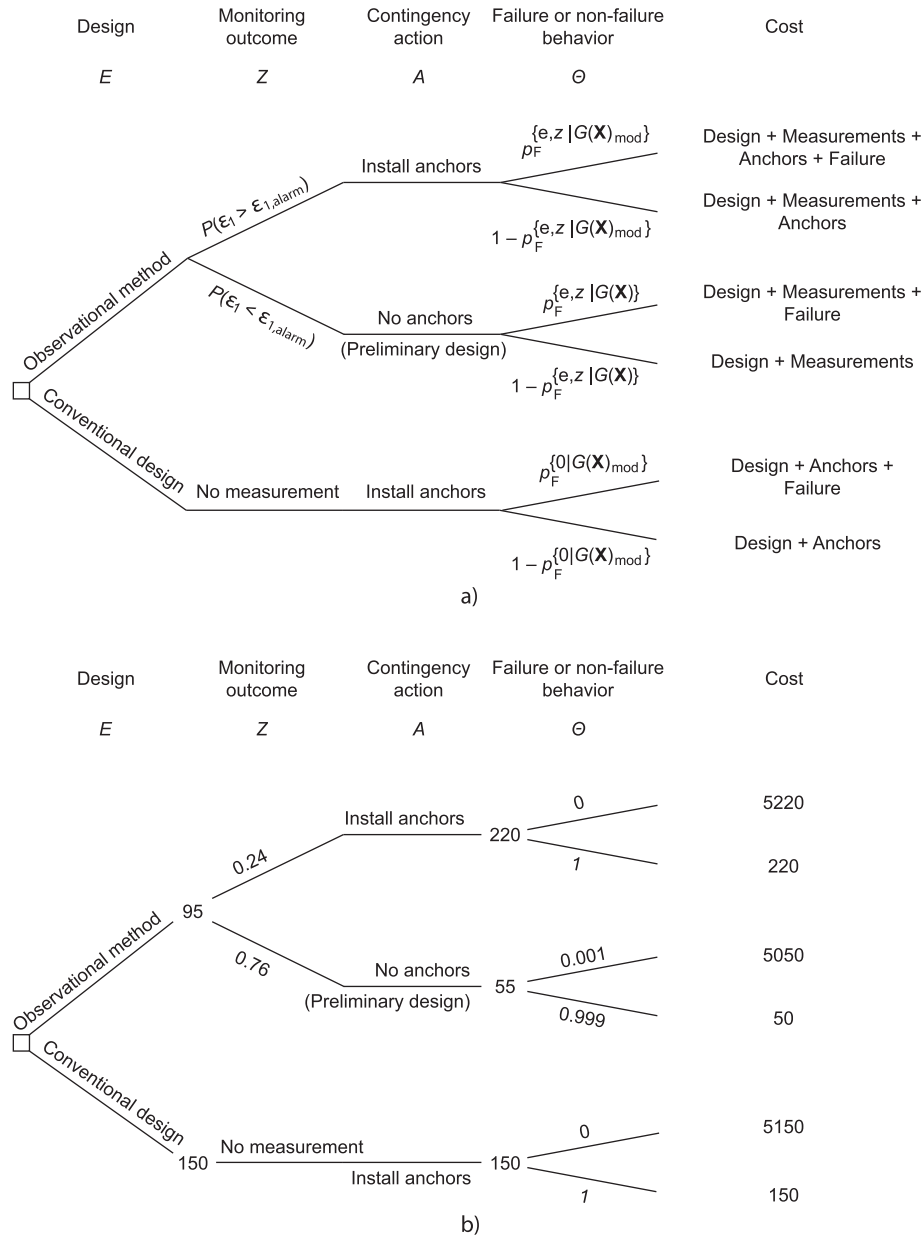


Fig. 7. Decision tree illustrating the choice between a conventional design and applying the observational method for the rock pillar. In a), the tree is described qualitatively; in b), the calculation of the expected cost of each design alternative is visualised. The outcome is that the observational method is found more favourable.

Table 2

Costs associated with the available design options: a conventional design or the observational method that includes a preliminary design and a modification.

Events \ costs [monetary units]	Construction cost	Advanced measurements	Structural failure	Total cost
Conv. design success	150	–	–	150
Conv. design failure	150	–	5000	5150
OM prel. design success	30	20	–	50
OM prel. design failure	30	20	5000	5050
OM mod. design success	200	20	–	220
OM mod. design failure	200	20	5000	5220

4.5. Establishment of limit of acceptable behaviour

An alarm level for the measurement of the vertical deformation, $\varepsilon_{1,\text{alarm}}$, that satisfies the reliability constraint of the completed pillar is found by truncating the probability distribution of its $\varepsilon_1^{(0)}$ at $\varepsilon_{1,\text{alarm}}$ such that $P(G < 0 | \varepsilon_1 \leq \varepsilon_{1,\text{alarm}}) = p_{FT}$, in accordance with

Eq. (12). This gives $\varepsilon_{1,\text{alarm}} = 0.33$ millistrain, which corresponds to a vertical deformation of 2.7 mm. The procedure is illustrated in Fig. 6b. Having defined $\varepsilon_{1,\text{alarm}}$, the probability of observing a behaviour within the acceptable limits is given by $P(\varepsilon_1^{(0)} \leq \varepsilon_{1,\text{alarm}})$. This probability indicates how likely the design is to satisfy p_{FT} without needing to install rock anchors.

4.6. Calculation results

Evaluating Eq. (4) for this example, the decision maker should adopt the observational method and, as a preliminary design, not install any rock anchors: the expected cost of applying the observational method is 95 monetary units (Eq. (5)) and the expected cost of implementing a conventional design is 150. The calculation results are presented in Fig. 7b. The preliminary design is expected to be sufficient with a probability of 76%, which gives a probability of 24% for needing rock anchors. The $p_{FF} = 10^{-3}$ is satisfied for all situations: for the preliminary design, $p_F^{(e,z|G(X))} = P(G < 0 | \varepsilon_1 \leq \varepsilon_{1,alarm}) = 10^{-3}$; for the modified design after the contingency action, $p_F^{(e,z|G_{mod}(X))} = P(G_{mod} < 0 | \varepsilon_1 > \varepsilon_{1,alarm}) \ll 10^{-3}$; and for the conventional design, $p_F^{(0|G_{mod}(X))} = P(G_{mod} < 0) \ll 10^{-3}$ (where the latter is evaluated based only on the prior information in Table 1). The probability of failure for the modified design is negligible; therefore, it is set to 0 in the calculations.

5. Discussion

5.1. On the reliability constraint

The reliability constraint on the completed structure that we have suggested for observational method applications eliminates a conceptual difference between the current use of the observational method and conventional design. While conventional design must satisfy a defined safety level, the observational method does not require this, regardless of the applied definition (Peck's original version [1] or Eurocode 7 [3]). However, considering society's demand for structural safety, we find it reasonable to add a reliability constraint on the observational method in future updates of Eurocode 7. The suggested methodology satisfies such a requirement. A desirable effect of applying the methodology is that the safety concerns previously reported [18,19] can be rejected.

5.2. On the limits of acceptable behaviour

In introducing a reliability constraint in the Eurocode definition of the observational method, we show how acceptable limits of behaviour can be established, so that contingency actions are put into operation only when they are needed to ensure safety: see Eq. (12). Furthermore, the reliability constraint enables showing that, for the preliminary design, "there is an acceptable probability that the actual behaviour will be within the acceptable limits", which Eurocode 7 requires (Section 2). The proposed methodology ensures that this probability is acceptable, as Eq. (4) only finds the observational method favourable, if the acceptable limits are sufficiently unlikely to be exceeded in relation to the cost of the corresponding contingency actions. If the contingency actions are too expensive and too likely to be needed, e_{opt} will contain a conventional design.

5.3. On the practical aspects in finding the optimal design approach

We acknowledge that realistic modelling of the design process and solving the optimization problem in Eq. (4) may seem overwhelming for cases more complex than our example, considering the large number of design alternatives and monitoring options, the possible monitoring outcomes and the corresponding contingency actions. The cost estimation for each event may also prove difficult. However, even simplified decision analyses may provide essential information for engineers working in practice. In our opinion, risk management should permeate decision-making engineers' everyday work [57], and the proposed methodology may

prove to be a valuable tool for improving both the risk awareness of engineers and the quality of the decision-making process. We acknowledge that this paper contains few guidelines for practical application, but we intend to study this further in future research and improve the applicability for different types of geotechnical structures.

6. Conclusions

We have presented a methodology that allows a geotechnical engineer to optimize the choice of design approach, when the observational method is believed to be a viable option. The methodology introduces a reliability constraint on the observational method that proves essential in establishing the acceptable limits of behaviour for the preliminary design in accordance with the requirements in Eurocode 7. We believe that applying the methodology may contribute essential information to the design process, even if the engineer chooses to simplify the analysis. Thus, the methodology may prove to be a valuable tool for decision-making engineers' everyday work with managing risks in geotechnical projects.

Acknowledgements

The presented research was funded and supported by the Swedish Hydropower Centre (SVC), the Rock Engineering Research Foundation (BeFo) and the Swedish Research Council Formas. The research was conducted without any involvement of the funding sources. SVC was established by the Swedish Energy Agency, Elforsk and Svenska Kraftnät together with the Luleå University of Technology, KTH Royal Institute of Technology, Chalmers University of Technology and Uppsala University (www.svc.nu).

References

- [1] Peck RB. Advantages and limitations of the observational method in applied soil mechanics. *Géotechnique* 1969;19:171–87.
- [2] Stille H. Experiences of design of large caverns in Sweden. In: Saari KHO, editor. *Proceedings of the International Symposium on Large Rock Caverns*. Helsinki: Pergamon Press; 1986. p. 231–41.
- [3] CEN. EN 1997-1:2004 Eurocode 7: Geotechnical design – Part 1: General rules. Brussels: European Committee for Standardisation; 2004.
- [4] Nicholson DP. *The observational method in geotechnical engineering*. London: Thomas Telford; 1996.
- [5] Nicholson DP, Tse C, Penny C. *The observational method in ground engineering: principles and applications*. London: Construction Industry Research and Information Association (CIRIA); 1999.
- [6] Chapman T, Green G. Observational method looks set to cut city building costs. *Proc Inst Civil Eng Geotec* 2004;157:125–33.
- [7] Moritz B, Schubert W. Application of the observational method in geotechnical safety management. *Geomech Tunnelling* 2009;2:269–81.
- [8] Hartlén J, Jansson S, Christensen H. Citytunneln, Malmö: geotechnical hazards and opportunities. *Proc Inst Civil Eng Geotec* 2012;165:35–44.
- [9] Wu TH. 2008 Peck lecture: the observational method: case history and models. *J Geotech Geoenviron* 2011;137:862–73.
- [10] Serra JB, Miranda L. Ground uncertainty implications in the application of the observational method to underground works: comparative examples. In: Withiam JL, Phoon KK, Hussein M, editors. *Foundation Engineering in the Face of Uncertainty*. San Diego, CA: ASCE; 2013. p. 254–70.
- [11] Spross J, Larsson S. On the observational method for groundwater control in the Northern Link tunnel project, Stockholm, Sweden. *Bull Eng Geol Environ* 2014;73:401–8.
- [12] Prästings A, Müller R, Larsson S. The observational method applied to a high embankment founded on sulphide clay. *Eng Geol* 2014;181:112–23.
- [13] Bjureland W, Spross J, Johansson F, Stille H. Some aspects of reliability-based design for tunnels using observational method (EC7). In: Schubert W, Kluckner A, editors. *Proceedings of the workshop Design practices for the 21st Century at EUROCK 2015 & 64th Geomechanics Colloquium*. Salzburg: Österreichische Gesellschaft für Geomechanik; 2015. p. 23–9.
- [14] Miranda T, Dias D, Pinheiro M, Eclaircy-Caudron S. Methodology for real-time adaptation of tunnels support using the observational method. *Geomech Eng* 2015;8:153–71.

- [15] Spross J, Johansson F, Uotinen LKT, Rafi JY. Using observational method to manage safety aspects of remedial grouting of concrete dam foundations. *Geotech Geol Eng* 2016;34:1613–30.
- [16] Shen S-L, Cui Q-L, Ho C-E, Xu Y-S. Ground response to multiple parallel microtunneling operations in cemented silty clay and sand. *J Geotech Geoenviron Eng* 2016;142:04016001.
- [17] Buckby RJ, Key CL, Palmer MJ, Penman JG, Swannell NG. An observational approach to design of a new grout curtain at Wimbleball Dam. In: Winter MG, Smith DM, Eldred PJJ, Toll DG, editors. *Proceedings of the XVI ECSMGE: Geotechnical Engineering for Infrastructure and Development*. London: ICE Publishing; 2015. p. 559–654.
- [18] Powderham AJ, Nicholson DP. The way forward. In: Nicholson DP, editor. *The observational method in geotechnical engineering*. London: Thomas Telford; 1996. p. 195–204.
- [19] Powderham AJ. The observational method—learning from projects. *Proc Inst Civil Eng Geotech* 2002;155:59–69.
- [20] Harrison J. Design practices for the 21st Century. In: Schubert W, Kluckner A, editors. *Proceedings of the workshop Design practices for the 21st Century at EUROCK 2015 & 64th Geomechanics Colloquium*. Salzburg: Österreichische Gesellschaft für Geomechanik; 2015. p. 1–10.
- [21] Bozorgzadeh N, Harrison JP. Observational method in the context of rock engineering limit state design. In: Ulusay R, Aydan Ö, Gerçek H, Hindistan MA, Tuncay E, editors. *Proceedings of EUROCK 2016 – Rock Mechanics and Rock Engineering: From the Past to the Future*. Cappadocia: CRC Press; 2016. p. 733–8.
- [22] Spross J, Johansson F, Stille H, Larsson S. Towards an improved observational method. In: Alejano LR, Peruchó A, Olalla C, Jiménez R, editors. *EUROCK 2014: Rock Engineering and Rock Mechanics: Structures in and on Rock Masses*. Vigo, Spain: Taylor & Francis group; 2014. p. 1435–40.
- [23] Raiffa H, Schlaifer R. *Applied statistical decision theory*. Boston, MA: Division of Research, Harvard Business School; 1961.
- [24] Benjamin JR, Cornell CA. *Probability, statistics, and decision for civil engineers*. New York, NY: McGraw-Hill Book Company; 1970.
- [25] Sturk R, Olsson L, Johansson J. Risk and decision analysis for large underground projects, as applied to the Stockholm ring road tunnels. *Tunn Undergr Sp Tech* 1996;11:157–64.
- [26] Einstein HH. Risk and risk analysis in rock engineering. *Tunn Undergr Sp Tech* 1996;11:141–55.
- [27] Karam KS, Karam JS, Einstein HH. Decision analysis applied to tunnel exploration planning. I: Principles and case study. *J Constr Eng M ASCE* 2007;133:344–53.
- [28] Zetterlund M, Norberg T, Ericsson LO, Rosén L. Framework for value of information analysis in rock mass characterization for grouting purposes. *J Constr Eng M ASCE* 2011;137:486–97.
- [29] Zetterlund MS, Norberg T, Ericsson LO, Norrman J, Rosén L. Value of information analysis in rock engineering: a case study of a tunnel project in Äspö Hard Rock Laboratory. *Georisk* 2015;9:9–24.
- [30] van Baars S, Vrijling JK. Geotechnical applications and conditions of the observational method. *Heron* 2005;50:155–72.
- [31] Der Kiureghian A. First- and second-order reliability methods. In: Nikolaidis E, Ghiocel DM, Singhal S, editors. *Chapter 14 in Engineering design reliability handbook*. Boca Raton, FL: CRC Press; 2005.
- [32] Melchers RE. *Structural reliability analysis and prediction*. 2 ed. Chichester: John Wiley & Sons; 1999.
- [33] Faber MH. On the treatment of uncertainties and probabilities in engineering decision analysis. *J Offshore Mech Arct Eng* 2005;127:243–8.
- [34] Der Kiureghian A, Ditlevsen O. Aleatory or epistemic? Does it matter? *Struct Saf* 2009;31:105–12.
- [35] Vrouwenvelder ACWM. Developments towards full probabilistic design codes. *Struct Saf* 2002;24:417–32.
- [36] Baecher GB, Christian JT. *Reliability and statistics in geotechnical engineering*. Chichester: John Wiley & Sons; 2003.
- [37] Fenton GA, Griffiths DV. *Risk assessment in geotechnical engineering*. Hoboken, NJ: John Wiley & Sons; 2008.
- [38] Phoon K-K, Ching J. *Risk and reliability in geotechnical engineering*. Boca Raton: CRC Press; 2014.
- [39] Phoon KK, Retief JV. *Reliability of Geotechnical Structures in ISO2394*. London: CRC Press/Balkema; 2016.
- [40] Ditlevsen O, Madsen HO. *Structural reliability methods*. 2.3.7 ed. Kgs. Lyngby: Coastal, Maritime and Structural Engineering, Department of Mechanical Engineering, Technical University of Denmark; 2007.
- [41] Straub D. Value of information analysis with structural reliability methods. *Struct Saf* 2014;49:75–85.
- [42] Goulet J-A, Der Kiureghian A, Li B. Pre-posterior optimization of sequence of measurement and intervention actions under structural reliability constraint. *Struct Saf* 2015;52:1–9.
- [43] Thoft-Christensen P, Sørensen JD. Optimal strategy for inspection and repair of structural systems. *Civil Eng Syst* 1987;4:94–100.
- [44] Stille H, Holmberg M. Observational method in rock engineering. In: Mahdi A, Ghazvinian A, editors. *5th Asian Rock Mechanics Symposium*. Teheran: ISRM; 2008. p. 157–66.
- [45] Stille H, Holmberg M. Examples of applications of observational method in tunnelling. *Geomech Tunnelling* 2010;3:77–82.
- [46] Wu S-H, Ching J, Ou C-Y. Probabilistic observational method for estimating wall displacements in excavations. *Can Geotech J* 2014;51:1111–22.
- [47] Straub D. Reliability updating with equality information. *Probab Eng Mech* 2011;26:254–8.
- [48] Papaioannou I, Straub D. Reliability updating in geotechnical engineering including spatial variability of soil. *Comput Geotech* 2012;42:44–51.
- [49] Hall WB. Reliability of service-proven structures. *J Struct Eng* 1988;114:608–24.
- [50] Jaeger JC, Cook NGW, Zimmerman R. *Fundamentals of rock mechanics*. John Wiley & Sons; 2009.
- [51] Hoek E, Brown ET. *Underground excavations in rock*. Abingdon: Taylor & Francis; 1980.
- [52] Hoek E, Brown ET. Empirical strength criterion for rock masses. *J Geotech Eng-ASCE* 1980;106:1013–35.
- [53] Hoek E, Carranza-Torres C, Corkum B. Hoek-Brown failure criterion – 2002 edition. *Proceedings of NARMS-Tac*. Toronto: University of Toronto; 2002. pp. 267–273.
- [54] Bidgoli MN, Zhao Z, Jing L. Numerical evaluation of strength and deformability of fractured rocks. *J Rock Mech Geotech Eng* 2013;5:419–30.
- [55] Hoek E, Brown ET. Practical estimates of rock mass strength. *Int J Rock Mech Min Sci* 1997;34:1165–86.
- [56] Aladejare AE, Wang Y. Evaluation of rock property variability. *Georisk* 2016;11:22–41.
- [57] Spross J, Olsson L, Hintze S, Stille H. Would risk management have helped? – A case study. In: Schweckendiek T, van Tol AF, Pereboom P, van Staveren MT, Cools PMCBM, editors. *International Symposium on Geotechnical Safety and Risk 2015*. Rotterdam: IOS Press; 2015. p. 745–51.