Optimization of LCC for soil improvement using Bayesian statistical decision theory

J. Spross\textsuperscript{1, 2}, S. Hintze\textsuperscript{2}, S. Larsson\textsuperscript{1}

\textsuperscript{1}Division of Soil and Rock Mechanics, KTH Royal Institute of Technology, Stockholm, Sweden
\textsuperscript{2}NCC Infrastructure, Stockholm, Sweden

Abstract

Design decisions in geotechnical engineering typically need to be made under considerable uncertainty, both regarding present geotechnical conditions and future events occurring during the service life of the structure. To optimize the utility of societal investments, design decisions should consider the life cycle cost (LCC) and not only the construction cost. This paper investigates the applicability of Bayesian statistical decision theory to assist in this decision making. The paper illustrates the concepts with a practical example, where a geotechnical engineer considers three design alternatives for the foundation of a road embankment: pre-fabricated vertical drains with a surcharge, end-bearing and floating dry deep mixing columns. The effect of a potential extreme groundwater drawdown event on the LCC of these alternatives is analyzed and discussed. Concluding remarks are made on the relevance of such design tools in a structured risk management in geotechnical engineering projects.

Keywords: life cycle cost; embankment; observational method; PVD; dry deep mixing

1. Introduction

Design and construction of infrastructure projects always involves important decisions to find the most cost-effective foundation method for the actual ground situation. This situation is very common when designing embankment foundations on soft soil. If the soil condition under the embankment is soft clay, soil improvement will be needed, using for example preloading to induce consolidation settlement with a temporary surcharge load, often in combination with prefabricated vertical drains (PVDs) or Press-To-Drain (PTD). Consolidation techniques to improve the soil take place over a longer period. A quicker soil improvement option is dry deep mixing columns, where a large number of columns are installed in the soft soil.

The observational method has successfully been used to verify limit states of different applications since it first was presented by Peck [1]; see e.g. the textbook by Powderham and O’Brien [2]. Recently it has been shown to be relevant for serviceability limit states (i.e. settlements) when using preloading and deep mixing [3, 4, 5]. Using the observation method can often effectively manage the uncertainty in geotechnical projects to avoid delay and therefore minimize unexpected circumstances.

The decision of which soil improvement method that is most cost-effective shall not only be based on the cost for design and construction, as the performance during operation and maintenance phase often is even more important for the final life cycle cost (LCC). Examples are the long-term effects of vibrations from traffic and change of the ground water level. The influence effects vary for different foundation methods. The decision of foundation method for embankments should therefore also consider potential negative events occurring during its service life, which in Sweden today normally is between 50 and 120 years.

Samuelsson et al. [6] made a review of the presented result in the literature of environmental impacts and monetary costs from construction works regarding LCC. They found that only limited research has been published in applying life cycle analysis methods in geotechnical engineering. An exception is Praticò et al. [7], who proposed a method to analysis of LCC in selection of stabilization alternatives for better performance of low-volume roads.

In early design phases, it is often very difficult to evaluate the risks and LCC of different design alternatives. Here Bayesian statistical decision theory can be a tool for the optimal decision, see e.g. the excellent textbook for civil engineers by Benjamin and Cornell [8], and application examples by Wu [9, 10]. This paper therefore investigates the potential use of such a tool for the selection of embankment foundation method. Here we account for the respective construction costs, as well as the monetary risk associated with the effects of possible future permanent groundwater drawdown under the embankment.

Three foundation options are compared in a calculation example:

- PVDs with surcharge and the observational method
- End-bearing dry deep mixing columns
- Floating dry deep mixing columns (that do not reach a bearing layer)

In the following, we first introduce Bayesian statistical decision theory and an assessment method for risk costs of future negative events. The application of these concepts are then illustrated in a calculation example, which is followed by concluding remarks.

2. Bayesian statistical decision theory

To analyze which design alternative that is more favorable over the expected service life of the embankment, we apply Bayesian statistical decision theory. This means that the more favorable alternative is the one that minimizes the expected cost (or, more generally, maximizes the expected utility). Although such analyses are rare in geotechnical engineering research and practice, formal decision-theoretical analysis is a straightforward addition to reliability-based design. In a geotechnical context, the analysis comprises (at least) four parts, which often are illustrated with a decision tree (see Fig. 2 in the calculation example for illustration):

1. A set of main design alternatives, being the three foundation options (here denoted $E = [e_1, e_2, e_3]$, to align with the general term “Experiment”), with
or without possibility to gain additional information during construction through measurements.

2. An outcome (Z) of a performed measurement (if relevant).

3. A set of pre-planned actions (A = [a1, ..., an]) based on the measurement result.

4. The occurrence of a structural failure event (Θ = [θ1, θ2]; i.e., non-failure or failure) under the assumed design conditions.

5. The later occurrence, or not, of an additional structural failure event (Z = [z1, z2]) under other, more extreme, conditions, given that the structure has withstood previous loadings.

This setup allows a pre-posterior analysis, which means that the foundation design is decided under uncertainty, i.e., before the “state of nature” (“failure or non-failure structural behavior in Θ and Z) is known. A specific design outcome is a combination of a design alternative in E and any executed pre-planned actions in A. In the context of Bayesian decision analysis, this implies a decision rule where α is a function of α. To determine the optimal design alternative (eopt), which is associated with the minimum expected cost of unfavorable conditions and events are weighted together from each branch in the decision tree. The calculations are easy in practice, as shown in the example, although the mathematical expressions needed to cover all possible outcomes in the general case may seem daunting; the optimal design alternative in a decision tree is obtained by [11]:

\[
e_{\text{opt}} = \arg \min_{e} \left\{ \sum_{j=1}^{n} P(z_j | e_j) C(e_j, z_j, a_j) \right\}
\]

where \(P(z_j | e_j)\) is the probability of having the measurement outcome \(z_j\) when design alternative \(e_j\) is chosen, and \(C(e_j, z_j, a_j)\) is the weighted expected cost of each \(\theta\)-branch, given by

\[
C(e_j, z_j, a_j) = \sum_{k=1}^{2} \sum_{l=1}^{2} \left\{ P(\theta_k | e_j, z_j, a_j) \cdot \left( 1 + \frac{1}{r} \right) \cdot C(e_j, z_j, a_j, \theta_k, \xi_{k,l}) \right\}
\]

where \(C(e_j, z_j, a_j, \theta_k, \xi_{k,l})\) is the expected cost of event \(\xi_{k,l}\) when followed by event \(\xi_{k,l}\) (including the cost of executing the design, contingency actions, and repairs that are related to those events). \(P(\theta_k | e_j, z_j, a_j)\) is the probability of event \(\theta_k\) occurring given the executed design, and \(P(\xi_{k,l} | e_j, z_j, a_j, \theta_k)\) is the probability of event \(\xi_{k,l}\) occurring given the executed design and occurrence of \(\theta_k\). The constraint \(p_{\text{FT}}(e_j, z_j, a_j) \leq p_{\text{FT}}\) in Eq. (1) ensures that the original design satisfies the acceptable failure probability, \(p_{\text{FT}}\), for all design options. The expected cost associated with implementing the design described by \(e_{\text{opt}}\) is

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

which is evaluated similarly to Eqs. (1-2), but for the specific design solution \(e_{\text{opt}}\) only. (Fig. 2 in the illustrative example visualizes the calculation procedure in a decision tree.)

3. Risk costs for future extreme loading events

To assess the risk related to a future extreme loading event (\(\xi_{3z}\)) causing failure, the probability of the extreme event needs to be estimated. We used the Poisson distribution to model this; here specifically we let \(\lambda\) be a Poisson-distributed random variable describing the number of extreme ground drawdowns during the expected service life of the structure (\(T_{\text{ser}}\)), such that \(\lambda \sim \text{Pois}(\lambda T_{\text{ser}})\), where \(\lambda = 1 / T\) is the annual probability of experiencing extreme groundwater drawdown when it has a return period \(T\). The probability of experiencing at least one groundwater drawdown event during the service life, i.e. \(p_{\text{GW}} = P(\xi_{3z} | \xi_{3z})\), is then

\[
p_{\text{GW}} = 1 - P(\lambda = 0) = 1 - e^{-\lambda T_{\text{ser}}}
\]

Regarding the costs \(C_{\text{GW}} = C(e_{\text{wa}}, z_{\text{wa}}, a_{\text{wa}}, \xi_{3z})\) related to event \(\xi_{3z}\), they need to be discounted to attain the present value, \(C_{\text{GW}, r}\), of this future risk cost. Following the Swedish Transport Administration’s guidelines [12], we applied a discount rate of \(r = 3.5\%\). The discounting was performed assuming that \(C_{\text{GW}}\) is imposed after \(T_{\text{ser}}\) years, which is true on the average for a Poisson-distributed event; though the discounting time was limited to not exceed \(T_{\text{ser}}\):

\[
C_{\text{GW}, r} = C_{\text{GW}}(1 + r)^{-\min[r, T_{\text{ser}}]}
\]

The calculation procedure implies that the discounted one-time groundwater drawdown cost \(C_{\text{GW}, r}\) is imposed with the probability of having one or more extreme drawdown events during the service life. (Additional drawdown events are expected to cause considerably less damage, since the soil would already be consolidated.)

4. Illustrative example calculation

4.1 Overview of the three considered design alternatives

A 1.2-m high, 500-m long, and 23-m wide road embankment is to be constructed on 15.5 m of very soft clay. The design criterion investigated in this paper is to limit the probability of the embankment experiencing excessive residual settlement (i.e. larger than some acceptable settlement \(\Delta S_{\text{allow}}\)) after completion:

\[
P(G(\mathbf{X}) > \Delta S_{\text{allow}} - \Delta S \leq 0) \leq p_{\text{FT}}
\]

where \(\Delta S\) is the occurring residual settlement, and \(p_{\text{FT}}\) is the target failure probability for the limit state \(G(\mathbf{X}) = 0\), with \(\mathbf{X}\) being a vector of the involved variables. A \(p_{\text{FT}} = 5\%\) is used. Three design alternatives to satisfy this criterion are considered (Fig. 1):
The decision analysis provides support to determine the more favorable embankment foundation design with respect to the extreme groundwater drawdown event $\zeta_2$ in the decision tree. Key site conditions are described in Table 1. The clay is assumed normal-consolidated and any statistical trend with depth for the geotechnical parameters has been disregarded for simplicity. As the purpose of this paper is to discuss the decision-theoretical aspects in the selection of different design alternatives, the details of the analyzed limit states and applied calculation models are only briefly outlined; for further details, we refer to the provided references for each design alternative.

The construction cost for only the 1.2-m high embankment was estimated to 4.14 million SEK (MSEK) and the establishment of machinery to 50,000 SEK. For all three alternatives, the cost induced by unsatisfactory performance (i.e. “failure”) was set to 20 MSEK (before discounting).

### Table 1. Geotechnical parameters with probability distributions considered in the three design options. Values are based on the examples in [3, 4, 5], with some simplifications and adjustments to facilitate an illustrative example calculation.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Distribution</th>
</tr>
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<tbody>
<tr>
<td>Unit weight of soil, $\gamma_s$</td>
<td>Log-normal, mean of 18 kN/m$^3$ and COV of 5%</td>
</tr>
<tr>
<td>Unit weight of embankment, $\gamma_{emb}$</td>
<td>Log-normal, mean of 21 kN/m$^3$ and COV of 5%</td>
</tr>
<tr>
<td>Unit weight of dry crust, $\gamma_{dry}$</td>
<td>Set to 18 kN/m$^3$ (constant)</td>
</tr>
<tr>
<td>Soil modulus, $M_s$</td>
<td>Log-normal, mean of 1.0 MPa and COV of 16%</td>
</tr>
<tr>
<td>Column modulus, $E_{c, 28}$</td>
<td>Log-normal, mean of 24 MPa and COV of 25%</td>
</tr>
<tr>
<td>Effective cohesion of columns, $c', c_{28}$</td>
<td>Log-normal, mean of 45 kPa and COV of 25%</td>
</tr>
<tr>
<td>Effective friction angle of columns, $\phi'$</td>
<td>Log-normal, mean of 32° and COV of 5%</td>
</tr>
<tr>
<td>Vertical consolidation coefficient, $c_v$</td>
<td>Log-normal, mean of 0.2 m$^2$/year and 50% COV</td>
</tr>
<tr>
<td>Hydraulic conductivity of soil ($k_s$) and columns ($k_c$)</td>
<td>Assumed equal; calculated from $c_v$ and $M_s$</td>
</tr>
<tr>
<td>At-rest earth pressure coefficient, $K_0$</td>
<td>Set to 0.5</td>
</tr>
<tr>
<td>PVD properties and geometries</td>
<td>Triangular pattern with 0.7 m centre-to-centre drain spacing; see example in [4] for remaining PVD design assumptions.</td>
</tr>
</tbody>
</table>

We have here for simplicity assumed a normally consolidated clay and adjusted the assumed $M_s$ to compensate for a disregarded expected increase in stiffness ($M'$) for stresses considerably larger than the preconsolidation stress, which normally is observed in soft clay in Scandinavia. See details on the Swedish settlement calculation method in Larsson and Sällfors [13].

1. PVDs and 13 months of preloading with a surcharge that is monitored with the observational method.
2. End-bearing dry deep mixing columns.
3. Floating dry deep mixing columns penetrating 75% of the clay layer.

4.2 Design alternative 1: vertical drains with surcharge and the observational method

For the PVDs with a surcharge, the probabilistic design procedure proposed by Spross and Larsson [4] is applied, using the same layout of PVDs and properties as in their design example. To summarise this design procedure: Using Monte Carlo simulation, the uncertainty in the total, long-term primary consolidation settlement, $S$, is first assessed for the final embankment load. A surcharge height is then selected. The surcharge needs to be heavy enough to ensure that sufficient settlement (a calculable $s_{\text{target}} = 0.6$ m) and overconsolidation ($OCR_{\text{target}} = 1.1$) occurs in the compressed soil within the available preloading time. Following Spross and Larsson’s procedure, it is found that a surcharge of 1.2 m gives a probability of approximately 70% to meet $S_{\text{target}}$ within the available 13 months preloading time. To mitigate the risk of not meeting $S_{\text{target}}$, an observational approach is used during the preloading: If measurements indicate that the settlement rate is to slow to meet $S_{\text{target}}$ and $OCR_{\text{target}}$ within the preloading time, the pre-planned action (cfr. $a_1$ in the decision tree) to increase the surcharge height is put into operation. Thereby, the main design criterion $e_1$ in Eq. 1 will be achievable for all predictable soil conditions.

In addition to the cost for the original embankment and machinery, we have estimated the alternative-specific cost...
for PVDs with surcharge to be 31,500 SEK per running meter of embankment, including 18,000 SEK for installing a total of 525 m of PVDs per running meter, and 13,500 SEK per running meter for the added surcharge material, berms, and settlement compensation (see Fig. 1a). This gives a total construction cost of 19.9 MSEK for the 500-m long embankment. The cost to perform a surcharge increase (action \( a_i \)) was estimated to be an additional 3 MSEK.

4.3 Design alternative 2: end-bearing dry deep mixing columns

For end-bearing dry deep mixing columns, we used the probabilistic design procedure proposed by Spross et al. [5]. To avoid excessive residual settlement of the embankment, two limit states need to be analyzed: a limit state, \( G_1 \), for residual embankment consolidation settlement and an auxiliary limit state, \( G_2 \), to capture excessive settlement caused by column yielding, which is not captured by \( G_1 \). For \( G_2 \), we assume the common equal-strain, area-weighted model for the properties of the soil volume that is penetrated by columns. Wijerathna’s et al. [14] proposed composite vertical coefficient of consolidation is used to account for the rate of consolidation in the improved soil under the embankment. The effect of curing time of the columns is also considered. For \( G_2 \), the maximum increase in vertical column stress \((\Delta\sigma_{v,c})\) caused by the embankment is compared against the maximum allowable increase in column stress \((\Delta\sigma_{c,max})\) before yielding occurs. Because of expected load distribution, column yielding is evaluated at the top of the columns. All considered, Eq. (6) can be reformulated into a reliability system, where \( \Delta S \) and \( \Delta\sigma_{c,max} \) both are functions of the horizontal area ratio, \( R_{end} \), between soil and columns:

\[
P\left(\{\Delta S_{allow} \leq \Delta S\} \cup \{\Delta\sigma_{c,\text{max}} - \Delta\sigma_{v,c} \leq 0\}\right) \leq p_{\text{FT}}.
\]

Inserting the relevant variables of Table 1, this equation allows the computation of the \( R_{end} \) that is required to satisfy the assigned \( p_{\text{FT}} \), as \( R_{end} \) is the only unknown. For \( p_{\text{FT}} = 5\% \), an \( R_{end} = 0.13 \) was obtained, which gives a center-to-center distance of 1.5 m between columns of 600-mm diameter.

We have estimated the alternative-specific cost for the end-bearing columns to be 14,500 SEK per running meter of embankment for installing 245 m of deep mixing columns per running meter of embankment. With embankment and machinery, this gives a total cost of 11.4 MSEK.

4.4 Design alternative 3: floating dry deep mixing columns

For the floating 12-m-long dry deep mixing columns, we use the traditional simplified Swedish calculation model [15], where the improved soil is considered a block, which transfers most (80%) of the load through the columns to the soil under the bottom of the columns, and the remaining 20% is assumed to be transferred to the surrounding soil. Column yielding is calculated like that of the end-bearing columns. Residual settlement is calculated as the sum of the settlement in the improved block and the primary consolidation settlement under the block, which is calculated with an approximate 2:1 load distribution applied to the underlying 4 m of clay (Fig. 1c). Applying these adjustments to Eq. (7) gives for this design alternative a required area ratio \( R_A = 0.155 \), which corresponds to a center-to-center distance of 1.35 m between columns of 600-mm diameter.

We have estimated the alternative-specific cost for the floating columns to be 12,000 SEK per running meter of embankment for installing a total of 200 m of deep mixing columns per running meter of embankment. With embankment and machinery, this gives a total cost of 10.2 MSEK.

4.5 Susceptibility of design alternatives to damage from groundwater drawdown

Extreme groundwater pressure decrease (that exceeds initial design assumptions) in the till stratum will affect the design alternatives differently. The vertical drain alternative is judged to be damaged by such an event, because it would induce further consolidation settlement of the clay, in addition to that caused by the surcharge during the construction phase. Even though a margin for increased load is achieved through the slight overconsolidation \((OCR_{\text{target}} = 1.1)\), this margin by definition corresponds to only 10% of the preconsolidation pressure, which for soft clay corresponds to a margin of around 5 kPa. Thus, considerable primary consolidation settlement can be expected also for a moderate groundwater drawdown (> 0.5 m), followed by increased susceptibility to secondary consolidation processes as the clay no longer is overconsolidated.

The end-bearing dry deep mixing columns should be robust to a groundwater drawdown event. The columns stand on the firmer till stratum, so the whole soil volume under the embankment can be considered an improved composite material. Its stiffness is considerably larger than that of the clay, especially since the governing failure mode for these short columns turns out to be yielding (exceeded \( \Delta\sigma_{c,\text{max}} \)) rather than settlement, so the columns will therefore be overdesigned with respect to settlement. Moreover, the design stiffness of the columns does not account for the stiffness increase that is expected with time for cementitious composite materials.

The alternative with floating columns does, in contrast to the end-bearing alternative, have a layer of unimproved clay below the columns, where groundwater drawdown could cause consolidation settlement. Additionally, the groundwater drawdown may induce negative skin friction on the floating columns, adding a downward force to the columns. In this paper, we have assumed that the degree of damage and related risk cost a similar between alternative 1 and 3. In reality, the duration of the drawdown is important; due to the enhanced drainage of the PVDs, alternative 1 may be sensitive even if the duration of the event is rather short.

4.6 Result of decision-theoretical analysis of design alternatives

To find the alternative that is associated with the lowest life-cycle cost with respect to the risk for additional loading from groundwater drawdown, a decision tree can be set up...
to model this decision-theoretical problem. The branches of the tree illustrate the possible outcomes with associated probabilities. The most favorable design alternative is the one with the minimum expected life-cycle cost, having weighted the costs of the outcomes with respect to their probability of occurrence (Eq. 1). Fig. 2 shows the decision tree with costs and outcome probabilities for an assumed return period $T = 50$ years for the extreme groundwater drawdown, which gives a probability $p_{GW} = 0.7$ (Eq. 4) (incidentally similar in value to the probability of having sufficient surcharge in alt. 1).

Since $T$ may be difficult to determine, a sensitivity analysis was performed. Fig. 3a shows how the expected life-cycle cost is affected by the return period of the extreme event, with and without accounting for a discount rate. Fig. 3b shows the probability ($p_{GW}$) of experiencing at least one such event during $T_{ser}$. For this illustrative example, the following observations can be made:

- Even for long return periods, $p_{GW}$ is considerable (as large as 45% for $T = 100$ years).
- The applied discount rate, $r = 3.5\%$, does majorly reduce the associated risk cost (Eq. 5).
- Comparing the end-bearing and the floating columns, the floating columns may seem more favorable, if only construction costs are considered. However, if the discounted risk cost associated with a groundwater drawdown event is considered, the return period of the failure event needs to be around 60 years or longer for the expected life-cycle cost to be similar between these two alternatives.
- The design alternative with surcharge on vertical drains has from this perspective two disadvantages in this example: having the largest investment cost and being damageable by the groundwater drawdown event.

5. Concluding remarks

With an expected increased focus on sustainability and life-cycle analyses in geotechnical engineering, the construction industry needs to improve its tools for design analyses. Probabilistic analyses are often considered advanced, but they do offer a framework to account for the uncertainties that naturally arise when considering time spans far into the future. The first, important step is however to identify potential hazards and the related consequences – what is not recognized as an issue will never be accounted for in the design. This emphasizes the need to implement life-cycle analyses within a structured risk management framework, such as the ISO 31000. The calculation example of this paper could for example serve as a basis for the decision in the risk evaluation step of this framework: how well do the considered design alternatives satisfy the client’s risk acceptance of potential future serviceability issues? The presented calculation method visualizes the risk clearly, thereby facilitating effective risk communication in the project.

<table>
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<tr>
<th>Design alternative</th>
<th>Monitoring outcome</th>
<th>Contingency action</th>
<th>Failure or non-failure behavior [probability]</th>
<th>Additional failure event [probability]</th>
<th>Construction cost [kSEK]</th>
<th>Failure cost [kSEK]</th>
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Figure 2. Decision tree for analysis of LCC with respect to robustness to groundwater drawdown event, assuming a return period of 50 years and a discount rate of 3.5%. End-bearing columns has the smallest present cost, 12.44 MSEK per running meter of embankment (in boldface font).
Figure 3. a) LCC dependence on return period of the groundwater drawdown event for the three design alternatives. Filled lines (––) use 3.5% discount rate. Dashed lines (– –) use 0% discount rate. Dotted lines (…) are initial construction costs for reference. 
b) Probability of experiencing one or more groundwater drawdown events within $T_{ser} = 60$ years for different return periods.

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References