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# Finite element-based geotechnical risk analysis for anchor-supported deep excavations

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## Abstract

Evaluation of the reliability of deep excavation support systems requires the inclusion of the soil variability in conjunction with probabilistic analysis. This is often considered a complicated process; therefore, the method is usually not followed in routine engineering practice. Recent developments in finite element modelling now enable probabilistic analyses to be performed more conveniently. As an illustrative example, the case history of a 20 m-deep excavation supported by anchored drilled shafts in the Sogutozu district of Ankara was investigated. Experimental results indicate that saturated sand-gravel bands, which are common within Ankara Clay, have the potential to cause a significant decrease in the anchor capacity. Ergo the excavation performance, including structural capacity and wall lateral displacements, was evaluated taking into regard the probabilistic nature of the effect of these bands and the variability of the soil parameters. Back-analysis was performed and compared with inclinometer readings to examine the capability of the numerical model to simulate the field behaviour. Later, additional finite element analyses were carried out through Python scripting software to investigate the reliability of the excavation. Geotechnical parameters of the Ankara Clay produced by Monte Carlo simulation method were used in these analyses. The results were evaluated within the context of recommended acceptable reliability levels. The reliability of the system was determined to be relatively low when taking into account the adverse impact of sand and gravel bands.

**Keywords** Deep excavations · Risk analysis · Monte carlo simulation · PLAXIS · Python

## Introduction

Probabilistic analysis of geotechnical structures with conventional deterministic methods can not explicitly consider the effect of soil variability in excavation-induced deformations. Current semi-probabilistic reliability-based design (RBD) methods rely on similar traditional allowable stress design (ASD) except the replacement of the factor of safety (FS) with a combination of load and resistance factors (material partial factors) (Wang et al. 2008). It has been argued that even if the recommendations of the current LRFD codes pro-

vide load and resistance factors for a given target probability of failure, for practitioners it might be hard to incorporate site-specific uncertainties which are not explicitly defined in the specifications during the design phase. Although semi-probabilistic RBD methods have long been successfully applied to foundation design where load and resistance are usually independent, the applications of earth retaining structures have been less satisfactory considering the same source affecting both load and resistance (Christian et al. 2008; Wang 2013; Wang et al. 2016). Difficulties in conducting a comprehensive high-quality laboratory testing program, limited extent of geotechnical site investigations, natural (inherent) soil variability and uncertainties specific to design should encourage the designer incorporating a probabilistic framework in the design phase. This is particularly true for deep excavation projects in urban areas due to the catastrophic failure risk. On the other hand, incorporating probabilistic concepts into numerical analyses of complex geotechnical structures generally leads to a computational effort often not feasible from a practical point of view (Schweiger and Peschl 2005). However, recent technological developments in numerical methods and modelling have made the

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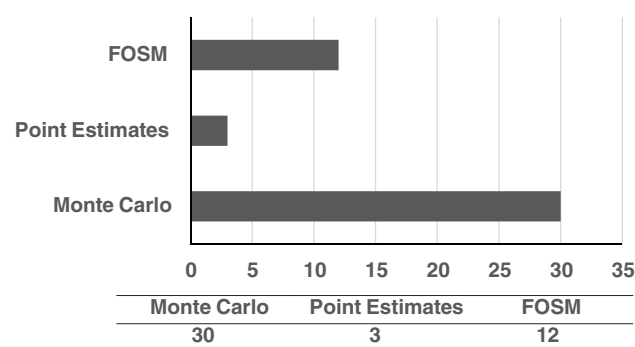
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reliability-based design easier to be incorporated, and have allowed much more convenient consideration of soil variability during the calculation phase. Hence, this study illustrates a simple methodology for the calculation of reliability considering the variabilities in geotechnical parameters using conventional finite element method through a deep excavation case history.

## Methodology

A number of methods are being used to determine the reliability of geotechnical designs based on probability theory. The most commonly used methods in the reliability-based design include the first-order reliability method (FORM), point estimation method (PEM), first order second moment (FOSM) and Monte Carlo simulation (MCS). The studies on the reliability-based design of deep excavations and slopes using PEM, FORM and MSC have shown a significant increase (e.g. Goh and Kulhawy 2005; Rippi 2015; Janssen 2016; Akbas and Kulhawy 2010). Recently, applications of probabilistic design methodologies were described for deep excavation case histories to investigate the effect of soil variability on the distribution of the wall displacements and settlement profile of the soil strata (Momeni et al. 2018; Bozkurt 2019; Nguyen et al. 2021; Arabaninezhad and Fakher 2021). In these studies, risk assessment of the designs associated with failure has been determined using different probabilistic methods.

Today, Monte Carlo simulation is the most prominent method among these three methods used in the design phase (Fig. 1). MC simulation is a numerical method in which a mathematical or an empirical parameter is randomly calculated in a given distribution (Wang et al. 2016). Simultaneous calculation of multiple input data of random variables without physical testing and computer simulations can be performed relatively easily. A typical MCS flow chart is presented in Fig. 2.



**Fig. 1** Simulation methods used on different papers in reliability-based designs in Geotechnics (Adopted from Viviescas et al. 2017)

Computer-aided statistical analyses and calculations are needed to understand the failure mechanism and determine conditions beyond the strength and serviceability limits and finalize geotechnical designs under engineering decisions. In this context, firstly, the statistical distribution of the variables constituting the problem should be decided. Then, for reliability problems, a limit equilibrium function ( $P = R - L$ ) should be checked. The system failure mechanism can be defined by the load ( $L$ ) and the resistance ( $R$ ) operators. Failure can be described not only as the yielding of all the basic elements in geotechnical design but also as the exceedance of the resistance effect ( $L > R$ ) or the limit conditions. If the limit equilibrium function ( $P$ ) is smaller than zero, the system will fail. According to  $n$  sets of simulations performed, the probability of failure ( $p_f$ ) can then be determined simply by calculating the number of failed analyses ( $n_f$ ) with Eq.(1):

$$p_f = \frac{n_f [P = (R - L) \leq 0]}{n} \quad (1)$$

After the calculation of  $p_f$ , the system reliability index,  $\beta$  can be evaluated by Eq. (2).  $\Phi$  operator is the standard normal probability distribution function.

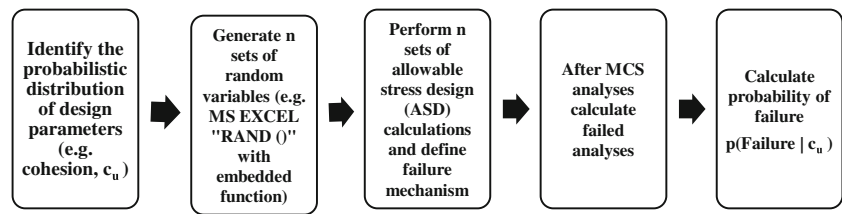
$$\beta = -\Phi^{-1}(p_f) \quad (2)$$

The required number of the simulations ( $N$ ) can be determined by converging towards a desired level of accuracy ( $1-\delta$ ). The use of the Central Limit Theorem allows the estimation of an optimal number of trials. The theorem states that regardless of the distribution of the variables from where samples were taken if  $N$  becomes large, the distribution of the sum of independent variables asymptotically approaches a Normal distribution (Baecher and Christian 2005). The required number of simulations can be defined using Eq. (3) provided that the probability of exceedance,  $p$  is equal to 0.5 for the upper limit (Fishman 1986).

$$N(\varepsilon, \delta) = \left[ \Phi^{-1}(1 - \delta/2)/(2\varepsilon) \right]^2 \quad (3)$$

where  $\varepsilon$  is the error margin of the calculation accuracy,  $(1-\delta)$  is the desired confidence level and  $\Phi^{-1}$  expresses the inverse of CDF for the standard Normal distribution. It was observed that the number of analyses was selected as 1000 in many geotechnical studies that employed MCS (e.g. Akbaş 2015; Tang 2011). The number of simulations required for a 95% confidence level will be 1000 with an error margin of 0.03 (Baecher and Christian 2005). Due to the fact that the probability of failure is an unknown prior to simulations, as a common procedure the number of analyses to be carried out could be determined by considering the situation in which the change in the probability of failure approaches a fixed value. The required number of analyses can be controlled through

**Fig. 2** MCS flow chart (Modified from Wang et al. 2011)



the convergence of  $\beta$  and  $p_f$  (e.g. Ahmed and Soubra 2011; Li et al. 2016). The ability to reflect the desired distribution of the probability density function (PDF) of the system variables can only be achieved by performing a sufficient number of random analyses. This can be seen as the main disadvantage of the MCS, since it can be a time-consuming process. In this study the MCS analyses of the finite element model were performed using Python scripting software. The main purpose is to demonstrate that the probabilistic analyses can be conducted quite easily following the methodology shown herein. The calculation procedure is illustrated in Fig. 3. Defined geotechnical variables associated with the failure mechanism are randomly generated for the given probabilistic distribution. The following steps would simply be performing finite element analyses and obtaining the results. PLAXIS users can manage and control the analysis sequence via built-in Python Interpreter in PLAXIS Finite Element (FEM) code.

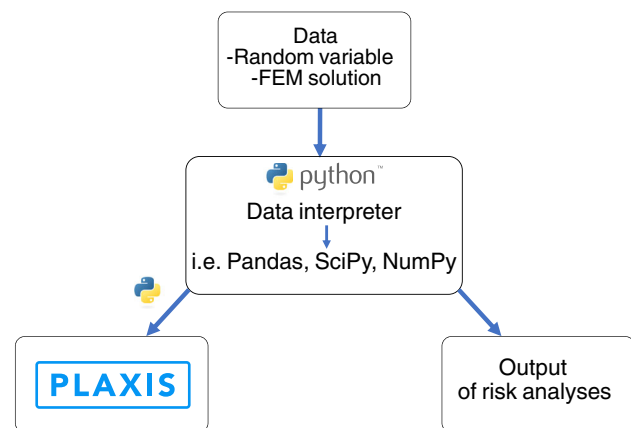
The research focuses on the role of soil variability in the failure mechanism of an excavation case history, specifically examining two main situations: the effect of sand-gravel bands on anchor capacity, and the soil inherent variability. The effect of soil variability on the lateral displacement and on the moment distribution of the anchored drilled shaft support was investigated. Initially, a back-analysis of the anchored drilled shaft support was performed, resulting in the acquisition of deformation characteristics comparable to those that were monitored in the field. Subsequently, the finite element analyses using PLAXIS 2D were carried out through Python scripting software, considering the soil variability

in the Ankara Clay. The soil parameters used in the back-analysis were chosen as the mean values for the reliability analyses, whereas the coefficient of variation (COV) of the strength and the stiffness parameters of the Ankara Clay was estimated utilizing the precedent investigations in the Ankara Clay (Akbas and Kulhawy 2010). Finally, the reliability of the deep excavation designed with the conventional method was examined by the MC simulation method.

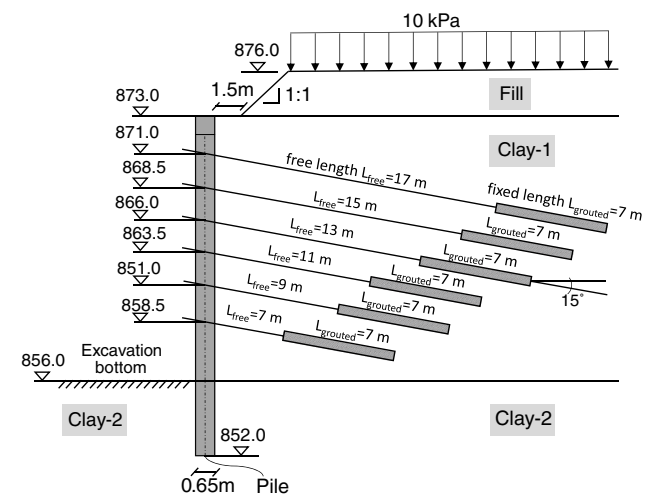
**Case study**

FEM analyses of a 20 m-deep excavation with an anchored drilled shaft support system designed for a temporary excavation in Ankara were conducted. The support system consists of 17 m long, 65 cm diameter drilled shafts supported by six rows of prestressed ground anchors, with an inclination of 15 degrees. The drilled shafts were constructed after the removal of a three-meter deep fill layer. Six rows of prestressed anchors were installed with 2.5 m and 2.0 m vertical and horizontal spacing, respectively. The fixed length of the grouted anchors was 7 m (Fig. 4).

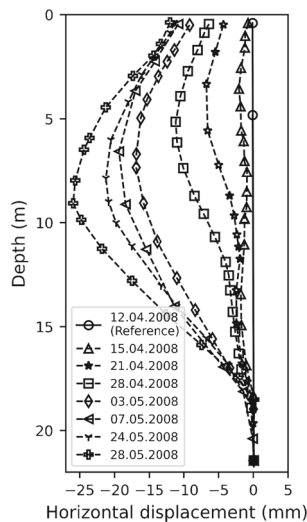
During the construction period, ten inclinometers were placed at specific coordinates to monitor the performance of the excavation support system. The maximum lateral displacement was recorded as 26 mm (Fig. 5). Horizontal



**Fig. 3** Reliability analyses FEM-Python procedure



**Fig. 4** Cross-section of the drilled shaft support and the application levels of prestressed anchors in Sogutozu, Ankara

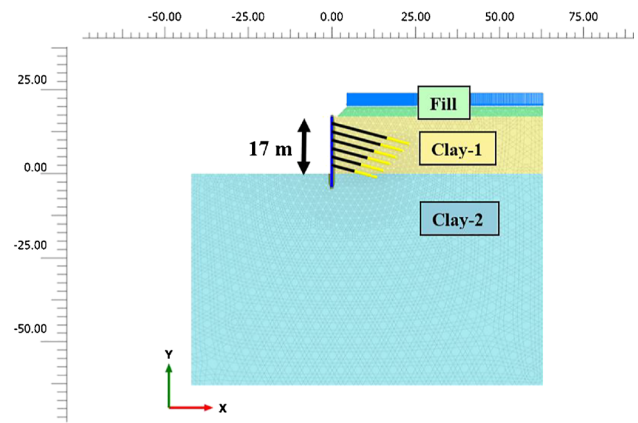


**Fig. 5** Inclinometer readings

deformation profile of the earth retaining systems can be predicted in accordance with depth-specific relative magnitudes of field monitoring data. As the stress–strain distribution of the soil during unloading is linked to the displacement profile. However, field monitoring data are significantly affected by the rigidity difference depending on whether the location of the inclinometers coincides with soil or drilled shaft. Due to the fact that the installation procedure of the inclinometers, and their coordinates were not available, the authors used the maximum magnitude recorded through available inclinometer data. In this study, the comparisons of lateral displacement predictions with the field performance have been limited only to quantitative comparison using the maximum lateral displacement value of the available inclinometer.

The critical condition for the excavations performed in overconsolidated clay is considered to be the long-term behaviour in the course of unloading (Lambe and Whitman 1969). During the geotechnical site investigations, the borehole survey gave no indication of a water table. Therefore, the numerical analyses were carried out for drained conditions. The earth retaining system was subjected to back analysis with the finite element method (PLAXIS) to determine whether the maximum lateral displacement value recorded in the field could be matched. The geometry used in the analyses is presented in Fig. 6.

The in situ soil profile is idealized by dividing the stratigraphy into three layers: Fill, Clay-1, and Clay-2. Clay layers are assumed to extend throughout the depth of the numerical analysis, as in some areas the Ankara Clay deposit can lie as deep as 200m (Erol 1993). Soil behaviour was modelled using the Hardening Soil (HS) material model (Schanz 1998; Schanz et al. 1999). This isotropic hyperbolic soil model accounts for the stress-dependent soil modulus. Un/reloading stress condition is assumed to be purely elastic. In



**Fig. 6** PLAXIS 2D numerical model of the drilled shaft support in Sogutozu, Ankara

the current PLAXIS code, the HS model is implemented for Mohr-Coulomb yield criterion. However, further extended versions of this model with different yield criteria and taking account of small strain stiffness which has a significant influence on the results can also be seen in Benz (2007); Benz et al. (2008). HS model has been validated for different soil types including soft to stiff clays (Surarak et al. 2012). It is worthwhile mentioning that, the effective stiffness and strength parameters were calculated using in situ test results and lower boundary of empirical relationships due to the scarcity of comprehensive laboratory testing. Soil strength parameters ( $c'$  and  $\phi'$ ) were estimated through SPT-N and plasticity index (PI) correlations (Stroud 1974), effective stiffness was defined using Duncan and Buchignani (1976); Craig (2004); Sorensen and Okkels (2013) to simulate the field behaviour (Table 1).

The use of empirical correlations during the risk assessment of the earth retaining system would lead to transformation uncertainty (Kulhawy 1992) which is a common shortcoming when high-quality laboratory test data are not available. Soil parameters, which later will form the basis for risk analyses, are presented in Table 2.

The stiffness moduli of the clay layers were calculated taking into consideration conventional design suggestions. Secant modulus ( $E_{50}^{ref}$ ) and oedometric modulus  $E_{50}^{oed}$  were taken as identical and un/re-loading modulus was calculated

**Table 1** Drained modulus calculation empirical equations

Parameter	Empirical equation	Reference
$E'$	$0.75 E_u$	Craig (2004)
$E_u$	$375 c_u$	Duncan and Buchignani (1976)
$c'$	$c_u 0.1$	Sorensen and Okkels (2013)

**Table 2** Soil parameters

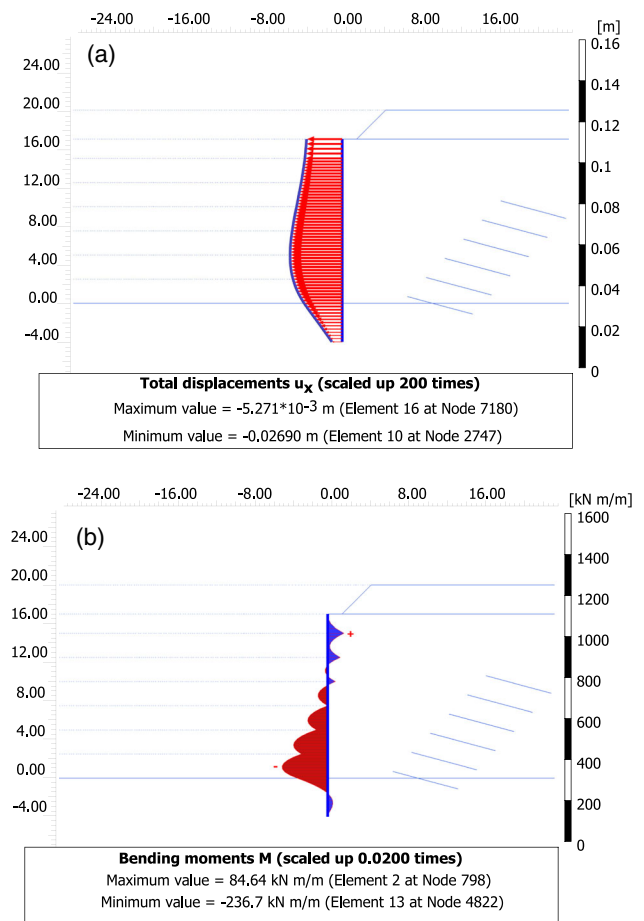
Parameter	Fill	Clay-1	Clay-2	Unit
Material Model	HS model	HS model	HS model	–
Type	Drained	Drained	Drained	–
$E_{50}^{ref}$	12000	55000	110000	$kN/m^2$
$E_{oed}^{ref}$	12000	55000	110000	$kN/m^2$
$E_{ur}^{ref}$	36000	165000	330000	$kN/m^2$
Power (m)	0.5	0.5	0.5	–
$c'$	3	20	25	$kN/m^2$
$\phi'$	25	25	25	$\hat{A}^\circ$
$p^{ref}$	100	100	100	$kN/m^2$
$R_{int}$	0.70	0.70	0.80	–

with  $E_{ur}^{ref} = 3E_{50}^{ref}$  as suggested in Plaxis Reference Manual (Plaxis 2021). The exponent of  $m$  defines the shape of the yield loci. During the analyses owing to limited information about laboratory testing  $m$  value and reference pressure,  $p^{ref}$  was assumed to be 0.5 and 100 kPa, respectively.  $m$  and  $p^{ref}$  should be calibrated using element-level test simulations that ensure the comparisons with respect to laboratory tests (e.g. pressuremeter, triaxial). In case the model is to be used for soft clays, the exponent of the power law can be selected as 1 (Schanz et al. 1999). For granular materials, the value ranges between 0.5–1.0 (Benz 2007). Interface,  $R_{int}$  describing the relative displacement between the drilled shaft and the soil has been estimated using Plaxis Reference Manual (Plaxis 2021). Finite element analyses were performed using 15 noded triangular elements. The contribution of the traffic surcharge load was taken as 10 kPa. According to the deterministic analysis performed with PLAXIS finite element software, maximum lateral displacement and moment values were calculated as 26 mm and 236.7 kNm/m, respectively (Fig. 7). Given the good agreement with the field measurements in terms of the amount of horizontal deformation of the wall, the finite element model was deemed successful in realistically simulating the behaviour of the excavation support system.

### Failure mechanisms and limit state function

In the context of a risk assessment of an anchored deep excavation, at least three main mechanisms should be examined:

- Support structural failure (drilled shaft, waling beams etc.)
- Exceedance of anchor capacity
- Global soil failure



**Fig. 7** The results of the 2D FE analysis. (a) Maximum lateral displacement and (b) moment profile of the drilled shaft support

The system failure is considered to be the exceedance of the wall lateral displacement or drilled shaft structural capacity. When carrying out multiple Monte Carlo Simulations sometimes numerical errors can occur that stop the analysis. These analyses were removed from the data set. During analyses, if the plastic limits are exceeded or soil fails, the result screen displays “soil collapse” or “load advancement” warnings as error codes indicating that the system has failed. Computational failures caused by soil failure and exceedance of plastic limits need to be stored separately during the iterative analyses. As a result of the PLAXIS 2D analyses, if the surficial three m-deep fill layer does not collapse or the plastic limits are not exceeded, the analyses are considered to be computationally successful, and the total multiplier ( $\sum M_{stage}$ ), describing the load advancement ultimate level, reaches 1.

$$P = \begin{cases} -1 & \text{last stage unsuccessful } M_{stage} \neq 1 \\ 1 & \text{last stage successful } M_{stage} = 1 \end{cases} \quad (4)$$

**Table 3** Suggested coefficient of variation values for clay (Extended from Phoon and Ching 2015)

Parameter	Coefficient of variation (COV)	Reference
Undrained shear strength	%13 – %40	Harr (1984); Kulhawy (1992)
$s_u$	%23	Lacasse and Nadim (1996) Akbas and Kulhawy (2010)
Effective friction angle, $\phi'$	%2 – %13	Harr (1984); Kulhawy (1992)

This success statement is not associated with reliability. It just means that the results of the analyses can be obtained for risk assessment whether the failure mechanism is within the acceptable limits or not. The earth retaining structure was intended to be used for a temporary excavation and the unacceptable conditions concerning the soil response was considered as the exceedance of limit lateral displacement and the design capacity of the drilled shafts. The calculations of the reinforcement of the drilled shaft support were done in accordance with TS-500 (2000) and Specifications, LRFD Bridge Design (2012). The moment capacity of the drilled shaft was calculated as 1.5 times the moment value ( $M_{deterministic}$ ) obtained from deterministic analysis (FEM back analysis). According to a comprehensive study all around the world in Long (2001), it has been reported that problems have been recorded when the ratio between lateral displacement and excavation depth is higher than 0.3%. Wang et al. (2008) has presented the deformation control criteria for excavations based on PSCG (2000) database. Depending on the presence of significant infrastructure in the vicinity and/or buildings in use within a distance of 1-2 excavation depth (H), allowable displacement has been defined as 0.3%H. The same value for lateral displacement to excavation depth ratio of 1/333 has been suggested for the horizontally supported walls constructed on fine-grained soils using soil nails in Federal Highway Administration (FHWA) Design Guideline (Lazarte et al. 2015). Owing to the aforementioned field experiences and design guidelines, in the FEM analyses Eq. (5) was used as the limit equilibrium function for lateral displacement.

$$P = \begin{cases} -1 & \delta_{xmax} \geq 1/333 * H_{excavation} \\ 1 & \delta_{xmax} < 1/333 * H_{excavation} \end{cases} \quad (5)$$

### Soil variability in Ankara clay

Risk assessment of the excavation support system requires realistic definitions of the statistical parameters describing the inherent variability of the underlying soil. The construction of the anchored drilled shaft supported deep excavation was conducted mainly in the Ankara Clay. Stiff Ankara Clay tends to show medium to high plasticity with plasticity indices ranging between PI=21-52% (Akbas and Kulhawy

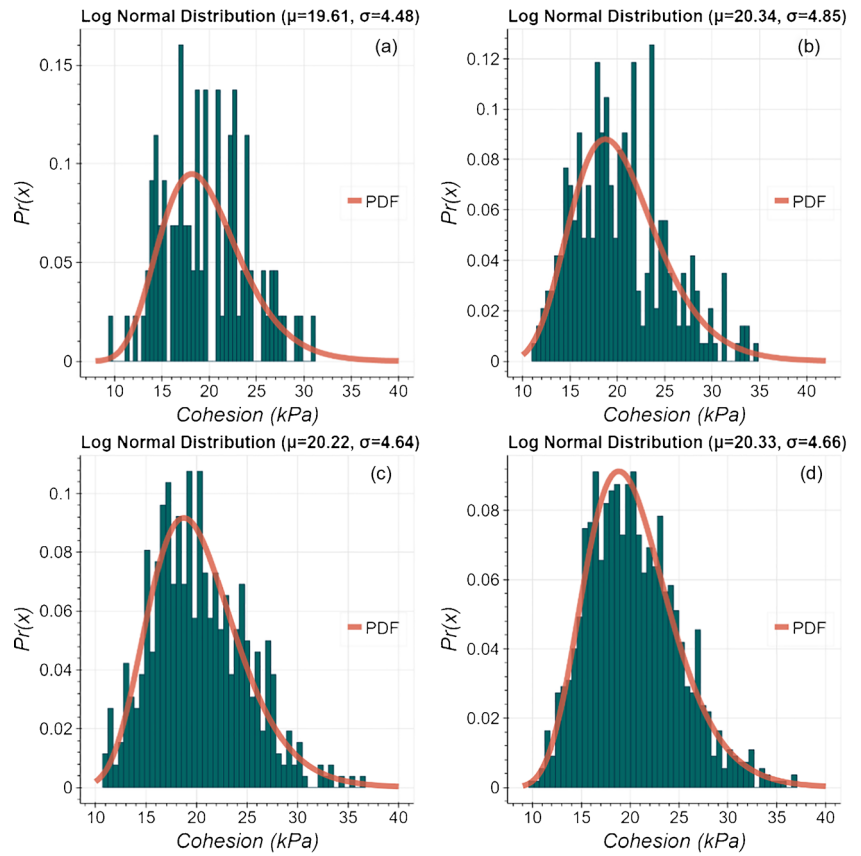
2010). The Ankara Clay, which exceeds 200m in places, commonly includes sand and gravel bands with varying thicknesses (Erol 1993). Highly heterogeneous structure results from the silt, sand and gravel content in the form of layers and lenses (Akgun et al. 2017). The variability of strength parameters of this unique inhomogeneous clay has been investigated comprehensively (e.g. Akbas and Kulhawy 2010; Akgun et al. 2017). Also, many studies present recommended values for the variability of geotechnical parameters for clays in general. The coefficient of variation (COV) of the strength parameters is summarized in Table 3.

The soil profile of the project area can be idealized to consist of three main layers: Fill, Clay-1, and Clay-2. Note that the variability of Ankara Clay was considered only for Clay-1 layer, in which the majority of the excavation took place, and thus includes all of the fixed lengths of ground anchors. It can be argued that the Clay-2 layer might also affect the lateral displacement profile (mostly passive resistance along the embedment depth), pile moment distribution marginally, and bottom heave mechanism significantly. Drilled shaft support would mainly work as a floating pile owing to the non-existence of stiff bearing strata and the shaft axial capacity would mostly be governed by the side friction from Clay-1 layer. This study is limited to the effect of soil variability of Clay-1 on the earth retaining structure and moment distribution. Often soil variability is reflected using the assumption of lognormal distribution especially for the parameters that can only take positive values (Phoon et al. 1999; Spry et al. 1988). Therefore, the assumed probabilistic characteristic of the strength parameters was decided to follow the lognormal distribution, and the mean statistical parameters of this layer were assigned through back-analysis. COV for Clay-1 parameters were estimated using Akbas and Kulhawy (2010) and Harr (1984). Considering the large deviation in values suggested for effective

**Table 4** Statistical parameters used in risk analyses

Parameter	$c_u$ (kPa)	$c'$ (kPa)	$\phi'$ (°)
Soil variability of Clay-1 layer			
Mean, $\mu$	200	20	25
Coefficient of variation, COV		23%	13%

**Fig. 8** M-C FEM (a-d) lognormal distribution cohesion histogram for 100, 300, 500 and 1000 simulations respectively

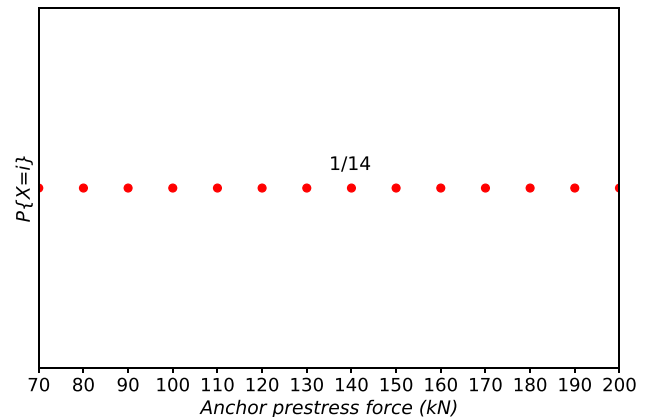


stress friction angle, COV was selected conservatively as 13%. The estimated statistical parameters for Ankara Clay are presented in Table 4.

Using  $\mu$  and  $COV$  parameters in Table 4, the values of effective cohesion, friction angle and elasticity modulus as the output of MC simulation have been generated and histograms for a different number of random simulations are plotted in Fig. 8. Even though 500 simulations are sufficient to reflect the assumed lognormal distribution for  $c'$ , an adequate number of simulations can be designated to reach the convergence in terms of the output of reliability analyses (i.e. lateral displacement and structural force distribution.)

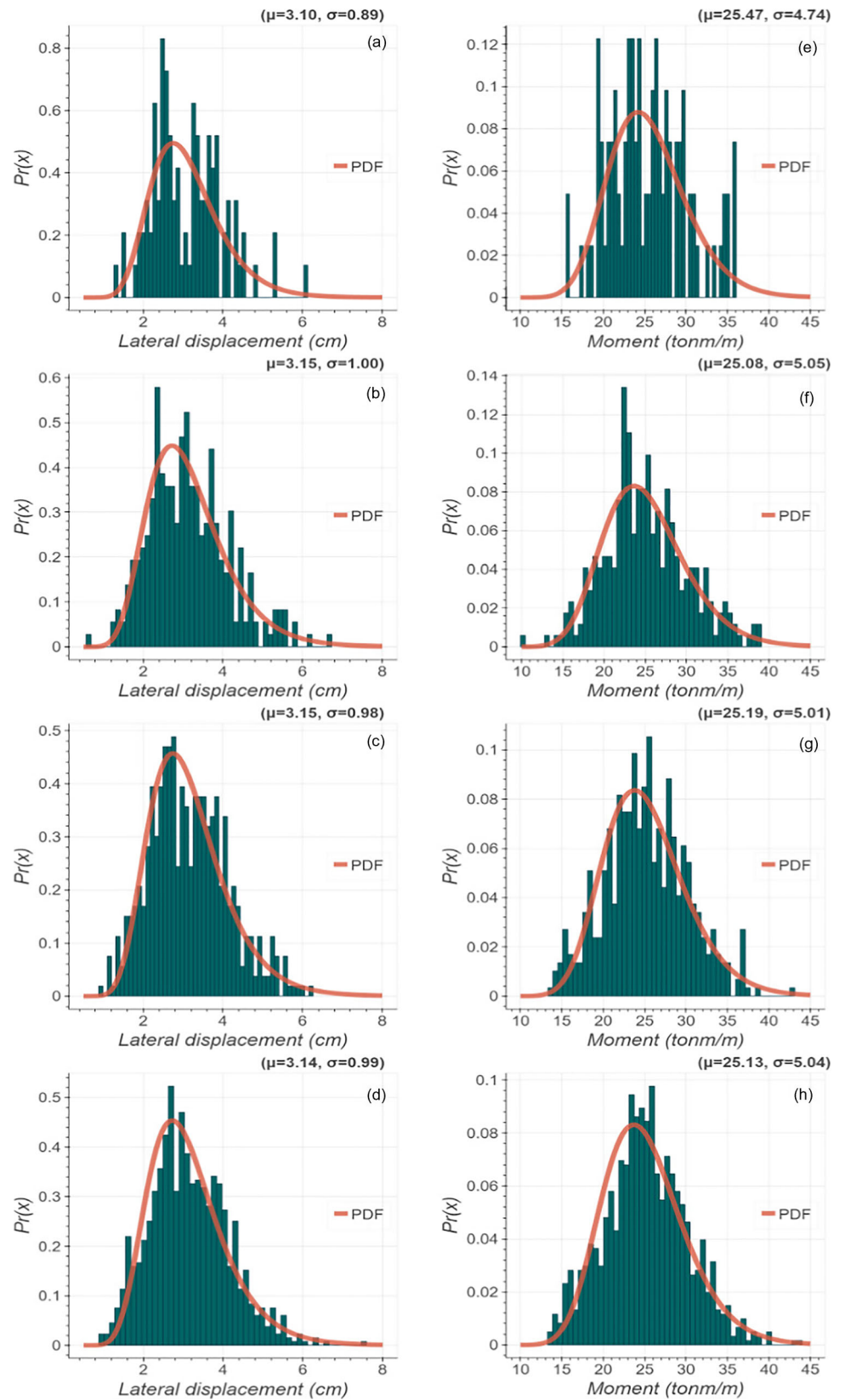
During the analyses, the water-bearing sand-gravel bands, which are commonly observed in the Ankara Clay were also taken into consideration to investigate the adverse effect of these bands on the excavation system behaviour. Ergun (1987) states that these bands cause a rapid decrease in anchor capacity with anchor test failure loads that range between 70-200 kN, which is less than half of what is commonly observed in Ankara clay. In the light of the field evidence indicating incidents of anchor failure due to the presence of sand and gravel in Ankara Clay, as reported by Ergun (1987), and the absence of anchor proof test results specific to the excavation site available, the risk assessment of the excavation considered the probable reduction in anchor prestress. For probabilistic modelling purposes, the sand-gravel band

content of Ankara Clay was investigated using data from 37 borehole logs in the Ankara Clay (Bozkurt 2019). According to this data, with a uniform probability of 15%, an anchor will be in contact with the sand-gravel band, otherwise, the grouting will be conducted in clay layer with a 400 kN prestress capacity, which is the assumed design value for the excavation considering field experiences in the area. Utilizing the probability of the presence of sand-gravel bands, the prestress load values varying between 70-200 kN were randomly assigned to anchors iteratively. Under the assumption



**Fig. 9** Anchor prestress force probability mass function

**Fig. 10** M-C FEM (a-d) lateral displacement for 100, 300, 500 and 1000 simulations respectively and (e-h) pile moment histogram for 100, 300, 500 and 1000 simulations respectively

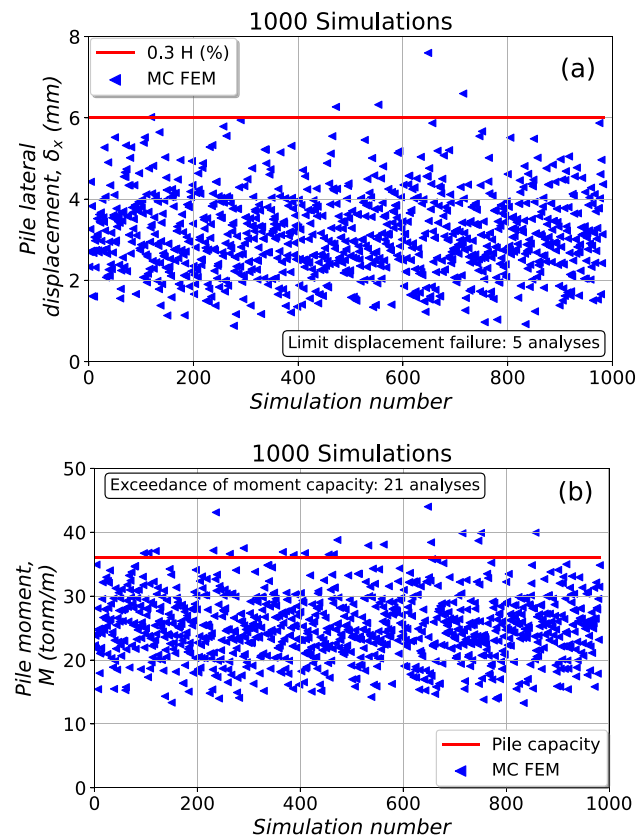


of a discrete uniform probability distribution, probability mass function,  $P(X = i)$  would correspond to 1/14 for every 10 kN increment of 70-200 kN range (Fig. 9). It is worth noting that this study did not primarily focus on the existence of water in sand-gravel bands per se, although low anchor capacity appears to be closely linked to the presence of sand and gravel bands within the Ankara Clay as stated in Ergun (1987).

### Reliability analyses

In order to estimate the required number of iterations with the aim of achieving a nearly constant reliability index value, 100, 300, 500, 750, and 1000 simulations were conducted. Note that the simple Python code for initialization and reading data from Plaxis output is presented in Appendix A.

Figure 10 shows the distribution of the maximum lateral displacement and moment values of drilled shaft support for varying numbers of finite element simulations. MCS analyses were conducted to examine mainly two issues affecting the system reliability: the exceedance of acceptable wall lateral displacement (Fig. 11a) and the pile structural capacity limits (Fig. 11b).



**Fig. 11** M-C FEM failed analyses (a) exceedance of acceptable lateral displacement and (b) exceedance of limit pile moment capacity

Within the scope of the reliability assessment of the anchored drilled shaft support, the results of the analyses are presented in Table 5 for different numbers of simulations. Taking into consideration the aforementioned points, a sufficient simulation number for this case history can safely be chosen as 500. Even after 300 simulations, the probability of failure and reliability index approach a nearly constant value for the examined case history.

The results of risk analyses have shown that in the Ankara Clay, considering the adverse impact of the sand-gravel bands on anchor capacity resulted in significantly low reliability.  $\beta$  of the realized design was calculated as approximately 1.9.

Risk analyses have also been conducted without the inclusion of sand-gravel bands. The comparison of both cases (with/without sand-gravel bands) depicts a clear distinction for the reliability assessment. Even though considering sand-gravel existence results in a rather low performance, ignoring sand-gravel bands leads to an overestimation of the reliability with the values of  $\beta_{SLS}$  calculated in a range of 2.3–2.5. The comparison of the results is presented in Table 6.

With regard to failure mechanism related to soil collapse, it can be argued that errors of the soil collapse in three m-depth fill layer and load advancement failures can be neglected considering that the construction of the structure has been completed without failure by a large scale of displacement in the short term (without drainage) condition (Fig. 12). The unrealistic failure mechanism within the fill layer was most likely triggered by numerical problems due to the load application and corner effects. Therefore, the failed analyses with soil collapse error led by the excessive deformation within the fill layer were neglected.

### Discussions

Current reliability-based design codes present methodologies for ultimate limit-state (ULS) designs with load and resistance partial factors which are calibrated for a target reliability index (e.g. Allen et al. 2005; Canadian Highway Bridge Design Code 2014; European Committee for Stan-

**Table 5** Results of risk analyses including sand-gravel bands

Max. number of iteration, (n)	Limit lateral displacement and moment capacity exceedance, (n)	$p_f$ (%)	$\beta$	Elapsed time (hr)
100	2	2.0	2.05	3
300	8	2.7	1.93	16
500	15	3.0	1.88	32
750	23	3.1	1.87	45
1000	26	2.6	1.94	65

**Table 6** Comparison of risk analyses including and excluding sand-gravel bands

Max. number of iterations, (n)	Inclusion of sand-gravel bands			Exclusion of sand-gravel bands		
	Limit lateral displacement and moment capacity exceedance, (n)	$p_f$ (%)	$\beta$	Limit lateral displacement and moment capacity exceedance, (n)	$p_f$ (%)	$\beta$
100	2	2.0	2.05	1	1	2.33
300	8	2.7	1.93	3	1	2.33
500	15	3.0	1.88	3	0.6	2.52

standardization, EN1997-1:2004 2004; European Committee for Standardization, EN1990:2002 2002; The Joint Committee on Structural Safety, JCSS2001 2001; International Standards Organization, ISO 2394: 2015 2015). However, deterministic analyses are still being used for conventional excavation designs and there is no design guideline specifically for the reliability assessment of deep excavations for SLS design. SLS target failure probability should be consistent with the objective to limit the loss of functionality and/or the occurrence of damage to economically acceptable levels (ISO 2394: 2015 2015). Although there are suggested design recommendations on the target reliability index for SLS design in specifications (i.e. EN1990:2002 2002; ISO 2394: 2015 2015; JCSS2001 2001), there is neither a clear distinction for temporary designs nor excavations. Suggested values in some of the specifications are given in Table 7.

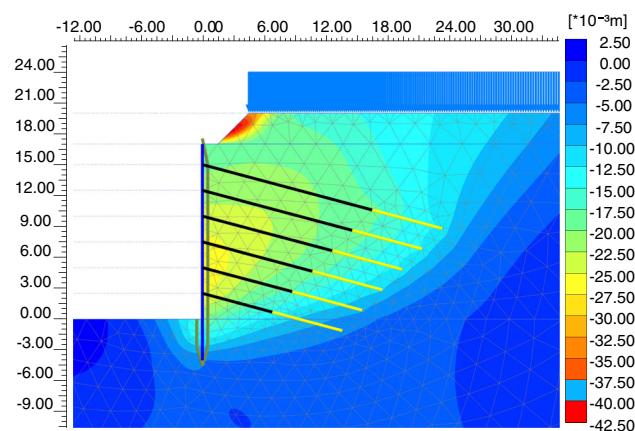
Semi-probabilistic methods to designate risk assessment with SLS considerations usually reduce random variables to a set of design values with a specific use (ISO 2394: 2015 2015). During the reliability analyses, incorporating project-specific load or resistance variability (e.g. soil inherent variability) and performing a set of random Monte Carlo simulations result in  $\beta_{SLS}$  with different order of partial factors that of ULS design. As in contrast to SLS design, ULS partial factors are fixed and calibrated for a target  $\beta_{ULS}$ . Interpretation of the reliability analyses with the use of a predefined limit state function necessitates comparing the

resulting reliability index with respect to a target  $\beta_{SLS}$ . Reliability analyses for SLS have been studied for a feasible range of target reliability index for deep excavations, shallow foundations (i.e. Akbas and Kulhawy 2009; Zhang and Liu 2022; Wu et al. 2015). As aforementioned above, there is no design guideline for target  $\beta_{SLS}$  specific to deep excavations, however, Wang and Kulhawy (2008) correlated target  $\beta_{SLS}$  with respect to  $\beta_{ULS}$  for augered cast-in-place piles. The authors highlight current specifications need to be enhanced with regard to SLS designs for retaining structures. In this context, a similar relationship between  $\beta_{SLS}$  and  $\beta_{ULS}$  might be utilized for deep excavation designs.

In this study, analyses were performed to assess the reliability of a temporary excavation with and without the effect of sand-gravel bands on the anchor prestress force. The probability of failure and the corresponding  $\beta_{SLS}$  was found to be relatively lower for the risk analyses with the inclusion of the adverse impact of sand-gravel bands. Considering important infrastructure facilities and the proximity of the buildings to the excavation area, the inherent variability of the natural soil and the adverse effect of sand-gravel bands on the anchorage capacities necessitate precautionous actions such as simultaneous field monitoring due to the failure risk.

## Conclusions

The reliability of an anchored deep excavation support system was investigated considering both the inherent soil variability in Ankara Clay and the existence of sand-gravel bands, which significantly decreases the anchor capacity. With the back analysis, the maximum lateral wall displacement measured as 26 mm in the field was recalculated. Subsequent to the back analysis, a large number of iterative analyses were performed using a simple Python scripting code within PLAXIS to assess the probability of failure. The reliability of the system by the inclusion of the adverse impact of sand-gravel bands was determined to be relatively low. The results indicate the strong need for pull-out tests on each anchor in the field for the confirmation of the design capacity. This assessment is based on the assumption that anchorage capacities are not field-tested and anchors are not replaced when they fail to achieve design capacities.



**Fig. 12** Lateral displacement profile of Plaxis FEM analysis and, soil collapse mechanism within 3 m-depth fill layer

**Table 7** Target reliability index,  $\beta_t$  for SLS in geotechnical RBD codes

Reliability class	Relative cost of safety measure	1 year reference period	50 years reference period	Short service life/obsolescence
RC3	High	–	–	1.3
RC2	Medium	2.9	1.5	1.7
RC1	Low	–	–	2.3
		European Committee for Standardization, EN1990:2002 (2002)	European Committee for Standardization, EN1990:2002 (2002)	The Joint Committee on Structural Safety, JCSS2001 (2001), International Standards Organization (2015)

[\*] Refer to Reliability Class 2 (RC2)

It was shown that the designs based on deterministic methods do not necessarily guarantee safety. Traditional methods include uncertainties that clearly can not be taken explicitly into account and this should encourage the designer to perform probabilistic analyses to determine the system failure probability as illustrated herein. Despite the disadvantage of increased computation time, performing an adequate number of analyses in MCS is crucial for accurately interpreting the results of reliability analyses. To specify the required number of analyses, the reliability index should converge for the specified failure mechanisms.

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## Declarations

**Declaration of competing interest** The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

**CRedit authorship contribution statement** **Sinem Bozkurt:** Conceptualization, Methodology, Formal analysis, Software, Writing - original draft, Visualization. **Sami Oguzhan Akbas:** Supervision, Writing - review & editing.

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## Appendix A: Python codes for remote scripting

Complete codes used during the analyses can be found in Bozkurt (2019). As an illustrative example, the codes written for Python (Version 3.4) - Plaxis2D connection, the definition of soil parameters for Clay-1 layer using soil variability, the calculation of randomized prestress anchor force and reading the maximum values of moment and displacement of the drilled shaft support are presented.

### *#Python-Plaxis communication:*

```
localhostport_input = 10000
localhostport_output = 10001
plaxis_path = input("Enter the installation location of Plaxis 2D")
found_module = imp.find_module('plxscripting', [plaxis_path])
plxscripting = imp.load_module('plxscripting', *found_module)
from plxscripting.easy import*
```

### *# Definition of the soil parameters for Clay-1 layer using soil variability:*

```
material_clay1 = g_i.soilmat()
cohesion = 200
cov_cohesion = 23
cohesion = random.lognormvariate(log(cohesion) - 0.5 * \
    pow(sqrt(log(1 + pow(cov_cohesion / 100.0, 2))), 2), \
    sqrt(log(1 + pow(cov_cohesion / 100.0, 2))))
phi = 25
cov_phi = 13
```

```

phi = random.lognormvariate
(log(phi) - 0.5*\
  pow(sqrt(log(1 + pow(cov_phi/100.0,
2))), 2),\
  sqrt(log(1 + pow(cov_phi / 100.0,
  2))))
countercohesion.append(cohesion*0.1)
counterfrictionangle.append(phi)
material_clay1.setproperties(
  "MaterialName", "Clay_1_" +
  str(data),
  "SoilModel", 3,
  "DrainageType", 0,
  "K0Determination", 1,
  "K0PrimaryIsK0Secondary", True,
  "gammaUnsat", 18,
  "gammaSat", 18,
  "E50ref", 262.5 * cohesion,
  "Eoedref", 262.5 * cohesion,
  "Eurref", 787.5* cohesion,
  "cref", cohesion * 0.1,
  "phi", phi,
  "psi", 0,
  "InterfaceStrength", 1,
  "rinter", 0.7,
  "K0nc", 1-sin(phi*pi/180),
  "K0Primary", 1-sin(phi*pi/180),
  "K0Secondary", 1-sin(phi*pi/180))
g_i.Soils[1].Material = material_clay1

```

**# Definition of the sand-gravel band presence and associated prestress force:**

```

while data < Iterationnumber:
  g_i.gotosoil()
  dataprestress = []
  for i in range(len(g_i.NodeTo
  NodeAnchors)):
    choices_layer = ['Clay',
  'Sandband']
    weights_layer = [0.85, 0.15]
    random_layer_specific = np.
    random.choice(choices_layer,
    p=weights_layer)
    if random_layer_specific ==
    'Clay':
      prestressforce = 400
    else:
      choices = [70, 80, 90, 100,
        110, 120, 130, 140,\
        150, 160, 170, 180, 190,
        200]
      weights = [1.0/14.0, 1.0/

```

**# Reading the maximum values of displacement and moment of the drilled shaft:**

```

if g_i.Phase_14.Reached.SumMstage == 1:
  g_i.view(g_i.Phase_14)
  x_left = 0
  momentlist = []
  lateraldisplacementlist = []
  for phase in g_o.Phases[1:]:
    maxM = 0.0
    minM = 0.0
    maxUx = 0.0
    minUx = 0.0
    plateX = g_o.getresults
    (phase,\
    g_o.ResultTypes.
    Plate.X, 'node')
    plateM = g_o.getresults
    (phase,\
    g_o.ResultTypes.
    Plate.
    M2D, 'node')
    plateUx = g_o.getresults
    (phase,\
    g_o.ResultTypes.
    Plate.Ux, 'node')
    for x, M in zip(plateX,
    plateM):
      if abs(x - x_left) <
      1E-5:
        if M > maxM:
          maxM = M
          xAtMaxM = x
        momentlist.append
        (maxM)
        if M < minM:
          minM = M
          xAtMinM = x

```

```

momentlist.append
(abs(minM))
for x, Ux in zip(plateX,
plateUx):
if abs(x - x\_left) <
1E-5:
if Ux > maxUx:
maxUx = Ux
xAtMaxUx = x
lateraldisplacementlist.
append(maxUx)
if Ux < minUx:
minUx = Ux
xAtMinUx = x
lateraldisplacement.
append(abs(minUx))
countermoment.append(max
(momentlist))
counterdisplacement.append(max
(lateraldisplacementlist))
g_o.close()
else:
countermoment.append("NA")
counterdisplacement.append("NA")

```

### Appendix B: Structural element properties

In this section, the parameters of the structures supporting the earth retaining system are provided.

**Table 8** Material properties of the structural elements used in the analyses

Property	Value
Excavation height	20 m
Reinforced drilled shaft diameter	Φ65 cm
Drilled shaft length	17 m
Spacing	1 m
Material model	Linear elastic
Anchor row number	6
Anchor grouted length	7 m
Steel type	STIIIa
Concrete	C25
Anchor horizontal spacing	2.0 m
Anchor vertical spacing	2.5 m
Anchor inclination	15°
Material model	Linear elastic

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