

Reliability Analysis of an Anchored Contiguous Pile Wall in Ankara Clay with the Random Set Finite Element Method

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Abstract. A deep excavation application characterized by imprecise data and lack of adequate information is used to demonstrate the efficiency, applicability, and validity of the random set theory in combination with finite element method (RS-FEM). A case history of an anchored contiguous pile wall in overconsolidated fissured Ankara Clay constructed for supporting the 15 m deep basement excavation of a nursing house in Seyranbaglari district of Ankara is considered. Existing buildings around the excavation area necessitated a careful examination of the wall deformations and the reliability of the system as a whole. However, the geotechnical parameters of the soil had to be estimated combining the results of very limited in-situ and laboratory tests with those obtained through previous experience of finite element analyses under similar conditions, i.e., expert knowledge. Plane strain finite element analyses were then performed to predict the contiguous pile retaining wall behavior. The parameters in the random set finite element model were chosen according to sensitivity analyses. Most likely bounds of the wall horizontal deformations were compared with those obtained from inclinometer readings. As suggested by previous case histories, wall deformations were observed to fall within the lower third of the range predicted by RS-FEM.

Keywords. random set theory, finite element method, deep excavation

1. Introduction

Dealing with uncertainties in the estimation of geotechnical parameters, which arise both from limited in-situ and laboratory test data and from the natural inherent variability of the geomaterial, as well as from the limitations within the empirical calculation models are one of the most important problems to be dealt with when designing a geotechnical structure. Because of the variability and uncertainties involved, there is an increasing tendency towards the use of risk-based or reliability-based approaches in the geotechnical design process. Numerous studies have been performed in recent years in the area of reliability-based and probabilistic design.

By using finite element codes in reliability analysis there are some advantages compared with limit equilibrium methods or other similar methods because more than one system parameter can be obtained without the need of changing the computational model. These can be used for the evaluation of the serviceability or the ultimate limit state of a geotechnical system

or of the respective elements of the system. The proposed method combines random sets to represent uncertainty and the finite element analysis (RS-FEM: random set finite element method), (H.F. Schweiger & G.M. Peschl, Plaxis Practice).

Within this context, the Random Set Finite Element Method (RS-FEM) that combines the Random Set Theory with the finite element method has proven to be a user-friendly framework in various applications to deal with uncertainties (e.g., Peschl, 2004; Schweiger and Peschl, 2007). Nasekhian and Schweiger (2010) demonstrated the application of random set theory in tunneling. Another application for a deep excavation problem was also illustrated by Schweiger (Plaxis Knowledge Base).

Following on these important contributions, in this study, an attempt was made to demonstrate the efficiency, applicability and validity of RS-FEM for a deep excavation application characterized by imprecise data and lack of adequate information. A case history of an anchored contiguous pile wall in

overconsolidated fissured Ankara Clay constructed for supporting the 15 m deep basement excavation of a nursing house in Seyranbaglari district of Ankara is considered. Existing old buildings around the excavation area necessitated a careful examination of the wall deformations and the reliability of the system as a whole. However, the site investigation included just four borings, from which only three undisturbed samples from relatively shallow depths were recovered. No in-situ tests other than SPT were conducted. Therefore, the geotechnical parameters of the soil had to be estimated combining the results of very limited in-situ and laboratory tests with those obtained through previous experience of finite element analyses under similar conditions, i.e., expert knowledge. These conditions deemed to produce a very suitable testing ground for the RS-FEM.

2. Soil Properties and Parameter Selection

2.1. Characterization of the Site

Due to the natural slope of the ground in the project area northwest–southeast direction, only the two sides of the 1500 m² excavation with a maximum height of 13.6 m and a total length of 62.5 m was required to be supported.

The geotechnical investigation originally involved drilling of three borings with depths ranging from 7.0 to 13.0 m. The top elevation of these borings is 948.00 m, which is 6.0 m lower than that of the retaining wall. Therefore, an additional boring at 953.00 m elevation with a depth of 16.0 m was also conducted. No in-situ test other than SPT were conducted, and only three undisturbed samples from elevations 946.00 m, 950.00 m, and 947.00 m, respectively, were recovered.

Information gathered from the borings indicate that, following a 0.5 m thick artificial fill, the site is underlain by a silty clay deposit that was classified either as CL or CH according to USCS. This deposit, which is typical in the metropolitan area of Ankara, is characterized also by thin layers of sand, which was not reported for this case. In boring no. 4, greywacke deposits was reached at about 16.0 m. Only one consolidation test was performed, and the

preconsolidation pressure was determined to be about 185 kPa at a depth of 2.5 m. The only strength data were undrained shear strength (s_u) values obtained from three standard UU compression tests.

As summarized above, the geotechnical exploration and the corresponding laboratory test results are limited both in terms of quality and suitability for the specific design case. It is well known that for retaining structures in stiff – hard, fissured clays, long-term stability usually constitutes the more critical design phase, and accordingly the effective strength parameters are required for stability calculations. However, the only available strength parameter for the clay is the undrained shear strength as obtained from UU tests, a value that is expected to be of low reliability. More importantly, except for a single consolidation test at a very shallow depth, no information about deformation parameters such as the modulus, which are needed for the estimation of wall deformations, is given. Therefore, the geotechnical parameters that are required for the numerical analyses had to be estimated combining the results of very limited in-situ and laboratory tests with those obtained through empirical relationships. Experience from previous local studies had to be utilized in selecting the proper correlations, as they are numerous in the literature. Note that, the clay profile is subdivided into two layers with the thickness of the first layer being 7 m, based on the variation of SPT N values with depth.

2.2. Estimation of Soil Parameters

In the literature, drained shear strength of clays have frequently been correlated with Atterberg Limits, especially the plasticity index (PI) (e.g., Kenney, 1967; Carter and Bentley, 1991; Bowles, 1988, Das 1975, EAB, 2008). A survey of these empirical relationships revealed the lower and upper limits for the drained friction angle to be 22° and 30°, respectively, for both clay layers in the project area with a PI that range between 22 and 28.

Considering both the shallow sampling depths, as well as the low reliability of the selected testing method, the undrained shear strength (s_u) values for the two layers given were determined using empirical correlations with the

SPT N value. According to Stroud (1974), for insensitive clays, the ratio of s_u to N , f_1 , is a function of the plasticity index (PI), and varies between 4.5 and 5.0 kPa for medium plasticity clays. Togrol and Sivrikaya (2007) estimated the same coefficient to be between 4.3 and 5.1 kPa, based on a statistical study using 185 clays from Turkey. Using the data obtained during the construction of the third section of the Ankara subway, Yaman (2007) recommended the use of Stroud's values for Ankara Clay. Based on these findings, f_1 was selected as 4.75, based on Stroud (1974) for the PI range of 22-28, in the current study. This corresponds to an undrained shear strength of about 190 kPa for the second clay layer.

According to Duncan and Buchignani (1976), considering the overconsolidation ratio and the average plasticity index of the clay, the ratio of undrained modulus (E_u) to s_u ranges between 200 and 500. However, the experience from similar excavation and retaining structure design studies in Ankara Clay indicates that the ratio between E_u and s_u can be estimated to be between 400 and 500. Assuming that the mean ratio of the drained (E_{50}^{ref}) to undrained modulus is 0.7 based on the recommendation given by Calisan (2009) specifically for Ankara Clay, E_{50}^{ref} for the second layer ranges between about 25000 and 65000 kPa. On the other hand, according to Stroud's (Clayton, 1995) correlation of modulus with N , the corresponding range is 40000 to 80000 kPa. A more recent study by Mayne (2001) indicates a much higher value of about 100000 kPa for the drained modulus of the second clay layer.

2.3. The Retaining System

The retaining system was designed to consist of 650 mm diameter drilled shafts at 1.0 m spacing, with an embedment length of 4.4 m. Five rows of anchors at 2.5 m and 2.0 m vertical and horizontal spacing, respectively, and each with 8.00 m fixed length were utilized. Note that an old masonry nursery building, with bad maintenance conditions exists at about 4.0 m distance from the excavation face, which is extra sensitive to ground displacements and settlements that could occur due to the proposed excavation. A typical cross section of the

anchored pile wall and a photograph taken during the last excavation stage are given in Figures 1 and 2, respectively.

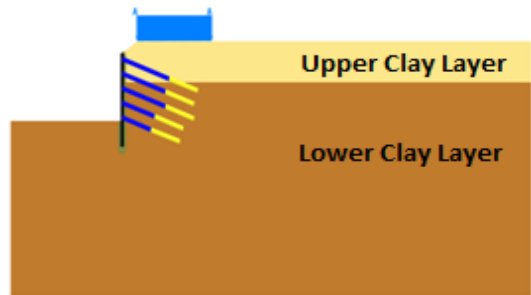


Figure 1. Cross-section of the anchored pile wall



Figure 2. A view of the retaining system

2.4. Sensitivity Study

As can be seen from the previous sections, due to limited characterization, the geotechnical parameters of the clay lies within a range of considerable extent, and thus, there is a significant amount of uncertainty. In the application of RS-FEM, it is important to identify the most influential parameters on the system response, in order to optimize the calculation effort. Thus, a sensitivity analysis can be used to reduce the number of uncertain parameter, whose impact on the result is negligible.

The sensitivity study conducted in this study involved the estimation of the effects of the variations of the effective stress friction angle, the effective cohesion, the drained modulus, and the power coefficient m , which helps simulate the stress dependency of stiffness in the hyperbolic soil model, on various performance measures. Note that the parameter m is reported by von Soos (1990) to take values between 0.5 and 1.0, in general. The measures of performance

include the maximum horizontal wall deformation, maximum amount of base heave, anchor force, and the ground displacement at the back of the wall.

The maximum and minimum values of the parameters considered in the sensitivity analyses are summarized in Table 1. Figures 3 through 7 present the effect of these parameters on the selected performance measures in a relative manner. A close look at these results indicates that the most influential parameters on the performance of the retaining system are the effective stress friction angle and the drained modulus of the lower clay layer.

Table 1. Range of Sensitivity Analysis Parameters

Sensitivity Analyses			
Parameter	$E_{{50}^{\text{ref}}}$ (Mpa)	$\phi'(o)$	c' (kPa)
Ranges	40 - 100	24 - 30	5 - 25

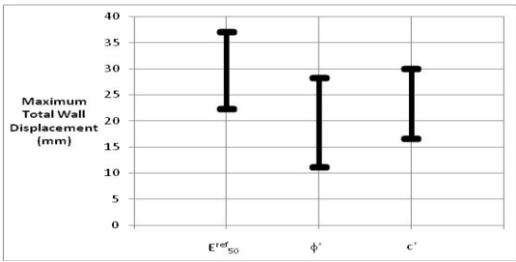


Figure 3. Change in total wall displacement

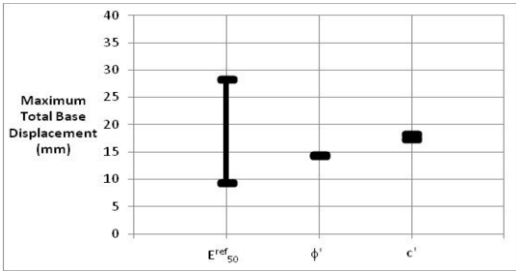


Figure 4. Change in maximum total base heave

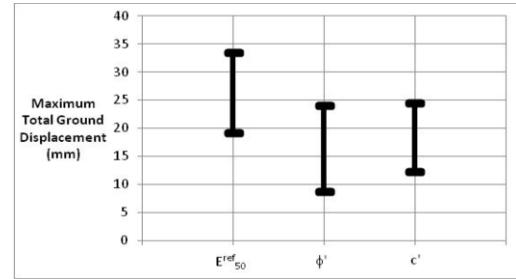


Figure 5. Change in maximum total ground displacement

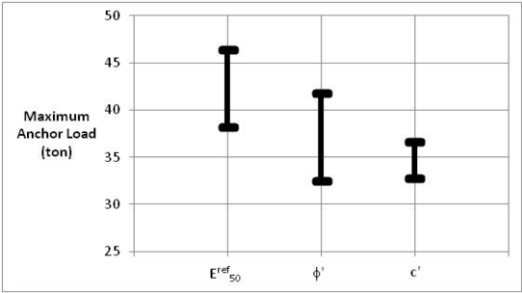


Figure 6. Change in maximum anchor load

3. RS-FEM Calculations

Plane strain finite element analyses were performed to predict the contiguous pile retaining wall behavior, focusing mainly on the deformations of the wall itself as well as the surrounding ground.

The plane-strain finite element model consists of 6293 fifteen noded triangular elements (Figure 7). The piles in the wall were modeled as beam elements, with interfaces around them to simulate soil-concrete interaction effects. The free and fixed anchor lengths were modeled by node to node anchors and using geotextile elements available in the finite element program, respectively. 370 kN pre-stressing force was applied to all of the anchors. The finite element mesh is finer around the fixed anchor length as well as around the piles, surcharge load and the pile tips for obtaining more accurate results with better convergence properties.

The seven excavation stages corresponding to each anchor elevation were simulated and the effect of the existing nearby building was modeled by an 80 kN surcharge load.

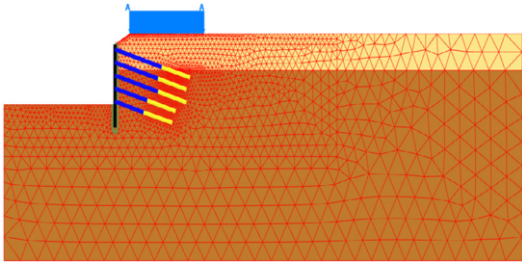


Figure 7. The finite element model of the wall

The parameters used in the analyses performed with the finite element code PLAXIS

2D V.2014 (Brinkgreve 2000) are summarized in Table 2, and a comparison of the inclinometer measurements and finite element results and deformed shape (scaled up to 200 times) of the wall are given in Figure 8 for various input combinations.

Table 2. FEM analysis parameter ranges

Parameters	E_{50}^{ref}	E_{oed}^{ref}	E_{UR}^{ref}	ϕ'	c'
Model	Hardening Soil				
Range	25 100	25 100	$3 \cdot E_{50}^{ref}$	22 30	5 25
Unit	MPa			°	kPa

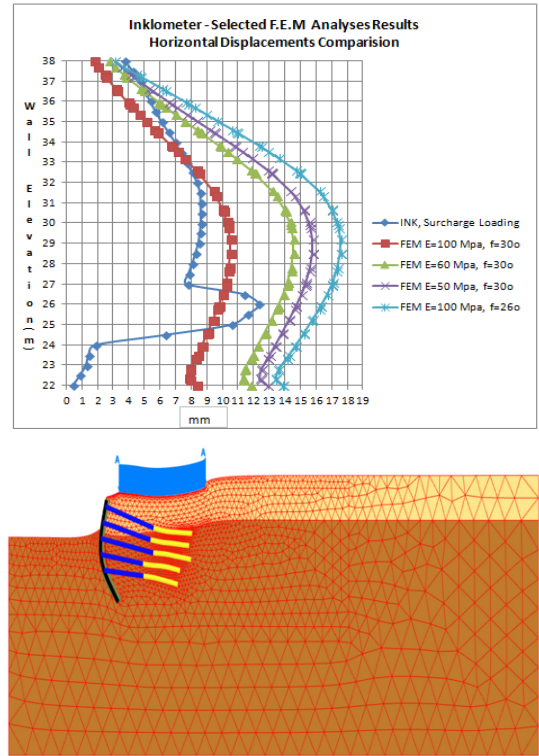


Figure 8. FEM results and comparison with measurements

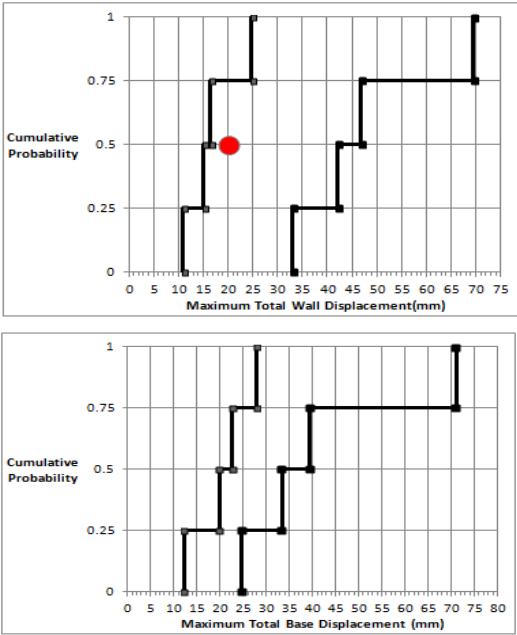
The final random set input variables utilized in the analyses are summarized in Table 3. After the identification of these input variables, the combinations of different sources and extremes of the parameters based on the random set model were calculated. Since there are only two independent random parameters, only 16 deterministic finite element calculations were performed in total. Since it was assumed that the random variables are stochastically independent,

the joint probability of response is equal to 0.25 for each lower and upper bound combination.

Table 3. Random set input variables

Parameters	E_{50}^{ref}	E_{oed}^{ref}	E_{UR}^{ref}	ϕ'	Prob. Of Assignment
Set No:1	25 - 50	25 - 50	75-150	22-25	0.5
Set No:2	60-100	60-100	180-300	26-30	0.5
Unit	MPa			°	-

Figure 9 plots the p-box of maximum total displacement of the anchored contiguous pile wall along with the respective measurement value. This plot indicates that the results are in good agreement with the measurement (20mm), and therefore it shows the general capability of the proposed approach to capture the uncertainties involved. The p-box plots of the maximum ground displacement, the maximum base heave, the maximum anchor force, and the maximum bending moment in the pile wall are also shown in Figure 9.



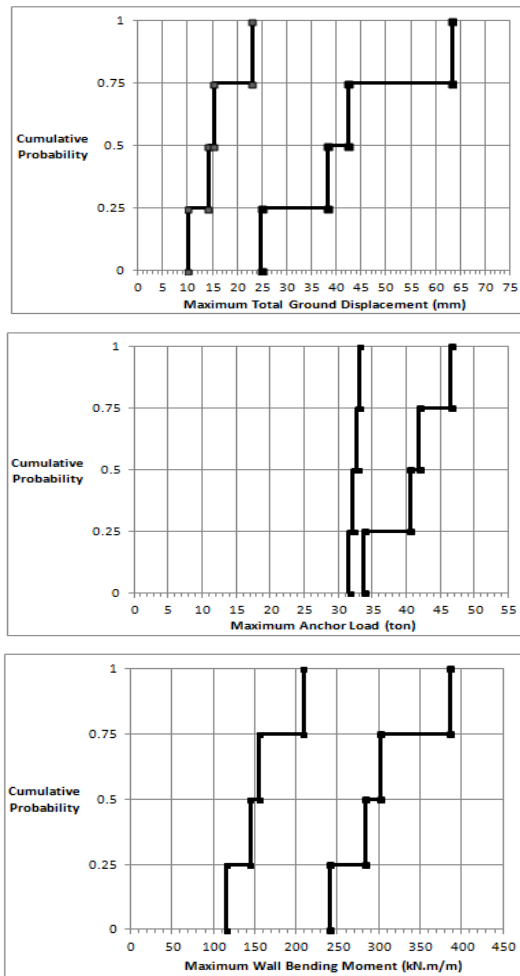


Figure 9. P – box drawings of analysis results

4. Conclusions

A deep excavation application characterized by limited amount of geotechnical information was used to demonstrate practical applicability of the RS-FEM. As suggested by previous case histories, wall deformations were observed to fall within the lower end of the range predicted by RS-FEM.

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