2D Numerical Modelling of Geosynthetic Reinforced Embankments over Deep Cement Mixing Columns

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ABSTRACT

This paper presents a two dimensional numerical implementation of a geosynthetic reinforced deep cement mixed (DCM) column supported highway embankment incorporating the strain softening behaviour of cement stabilised soil. This embankment was constructed over a very soft soil deposit in Finland. DCM columns were modelled using elasto-plastic Mohr-Coulomb model with isotropic strain softening behaviour. This material extension has been incorporated into ABAQUS/Standard through the user defined field subroutine (USDFLD). The geosynthetic reinforcement was modelled as a linear elastic perfectly plastic material using the Von-Mises failure criteria. Embankment fill, platform fill, soft soil, and firm soil were modelled as linearly elastic perfectly plastic materials using the Mohr-Coulomb failure criteria. The model simulations show good agreement with field data confirming the capability of the material extension and the 2D numerical modelling in predicting realistically the DCM column supported embankment behaviour.

Keywords: Geosynthetic reinforced embankments, Deep cement mixed columns, Finite element method, Strain softening, Soil arching

1 INTRODUCTION

In recent times, roads, railways and runways supported by embankments are constructed over soft clays or organic type weak soils with high compressibility and low shear strength characteristics, due to excessive pressure on land use. Although consolidation-based methods are available to improve the bearing capacity and settlement characteristics of soft ground, it is often not economical due to the uncertainties associated with soil conditions and time restrictions. Moreover these consolidation-based approaches may induce considerable settlement around the area and may damage adjacent facilities. Therefore DCM columns have become popular as a fast track ground stabilization method in Europe, North America and South-East Asia since the 1970s. Although reinforced concrete piles with pile caps can be used for the same purpose, DCM columns are more economical.

When dry cement is mixed with in situ soils to create DCM columns, the stiffness and strength of the in-situ soils increase significantly. However, when loads are applied, cement admixed soils show the strain softening behaviour due to degradation of the structure beyond yielding. Classical constitutive models (such as Modified Cam Clay and Mohr-Coulomb) are inadequate to predict the behaviour of this new material. Although, a few constitutive models have been developed for cement admixed soils, they are not available in commercial finite element programmes to analyse boundary value problems in geotechnical engineering. Hence, geosynthetic reinforced column supported (GRCS) embankments have not been analysed yet using a constitutive model incorporating key characteristics of cement admixed soil behaviour. In current practice the performance of GRCS embankments is assumed to be similar to pile supported embankments. However, there are significant differences between the constitutive behaviour of cement admixed soils and concrete commonly used for pile supported embankments. Though many researchers used numerical analysis based on the finite element method to investigate the behaviour of GRCS embankments, the constitutive behaviour of DCM columns are simulated assuming elastic-perfectly plastic behaviour using Mohr-Coulomb failure criteria. Therefore those analyses do not represent the real behaviour of DCM columns leading to underestimation of the total and differential settlements, unrealistic failure patterns and overestimation of bearing capacities. The aim of this paper is to investigate the GRCS embankment behaviour when the softening behaviour of cement admixed soils is incorporated into the finite element analysis. This paper describes the two-dimensional numerical modelling of a case study of a GRCS embankment constructed in Finland (Forsman et al. 1999). Elasto-plastic Mohr-Coulomb constitutive model is extended to address the strain softening behaviour of cement admixed soil under loading. Incorporation of material extension and numerical modelling are carried out with ABAQUS/Standard finite element program. This software facilitates the coupled mechanical and hydraulic modelling, gradual embankment placement and introduction of new material models. Numerical model is verified using the settlements measured during and five years after the construction of the embankment at the clay layer, the DCM columns and the strain of the geosynthetic layer.

2 MATERIAL EXTENSION FOR THE STRAIN SOFTENING CONSTITUTIVE MODEL

In this paper, an extended version of the elasto-plastic Mohr-Coulomb constitutive model is used to simulate the isotropic strain softening behaviour of cement admixed soil.

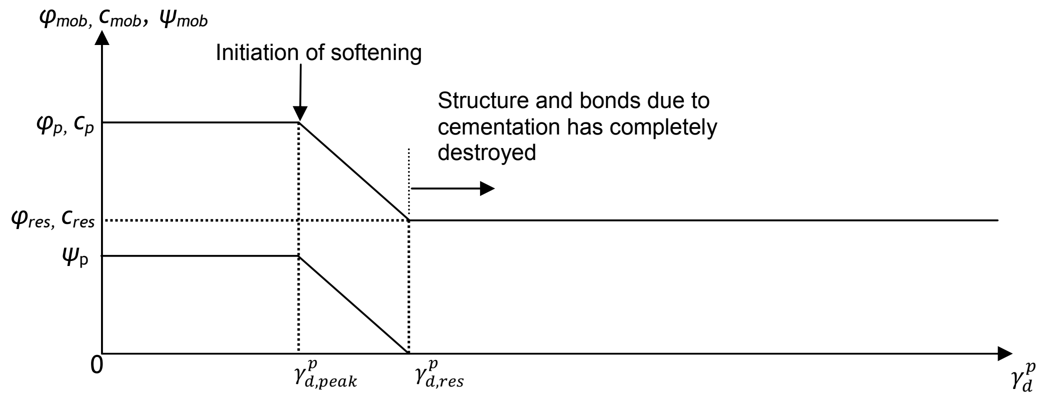

Figure 1. Variation of mobilized model parameters with octahedral plastic shear strain invariant, γ^p_d.

Strain softening is introduced by varying the mobilized friction and dilation angles, and cohesion intercept as a linearly decreasing function of the deviatoric plastic shear strain, γ^p_d. Once the stress state reaches the peak strength, cohesion (c_p), friction angle (φ_p) and dilation angle (ψ_p) start to reduce at deviatoric plastic shear strain, $\gamma^p_{d,peak}$, until the stress state reaches the residual strength of the soil (residual friction angle, φ_{res}, and cohesion, c_{res}) at plastic deviatoric shear strain, $\gamma^p_{d,res}$. Therefore strain softening behaviour of cement admixed clay is modelled as a reduction of undrained and drained shear strength parameters with increasing plastic deviatoric shear strain as illustrated in Figure 1. The values of these parameters are given in section 4.2. This material extension has been incorporated into ABAQUS/Standard finite element code through the user defined field subroutine, USDFLD (ABAQUS Inc.2010).

3 DESCRIPTION OF THE CASE HISTORY

This paper presents a two-dimensional numerical implementation of a well documented GRCS embankment constructed over a very soft clay deposit to approach a new bridge over Sipoo River at Hertsby, Finland. Detailed description of this case study has been given by Forsman et al. (1999) and numerical modelling of this case study has been already carried out by Huang and Han (2009) and Huang et al. (2009). However there are some differences between this study and previous studies and those differences are mentioned later in this paper.

To stabilize the soft soil deposit, DCM columns with diameter of 800 mm were used. Shear and unconfined compressive strengths of DCM columns were 150 kPa and 300 kPa respectively. Rows of DCM columns were installed parallel to the centre line of the embankment. Isolated columns and attached columns are used alternatively in rows. In order to increase the loads transferred to the columns and to reduce the loads transferred to the weak sub soil, the embankment was strengthened with a geosynthetic basal reinforcement layer which has 200 kN/m ultimate tensile strength in both longitudinal and transverse directions. This layer was placed 300 mm above the columns and sandwiched between two sand layers.

The construction sequence of the embankment consists of three main stages and they are simulated in six 0.3 m lifts (including the 30 mm thick geosynthetic reinforced layer). Figure 2 explains the layer thickness and the duration based on the actual construction procedure. The traffic load is assumed to be 12 kPa and it is applied over the crest as a uniformly distributed load. After the third stage, a consolidation period is continued up to a period of thirty years. The rate of fill placement within each stage is not available for the actual embankment construction. However, in the actual construction, embankment height is linearly increased and there should be a waiting period after each placement to dissipate the excess pore water pressures generated due to the fill loading. Therefore sequential construction procedure shown in Figure 2 is used for the analysis.

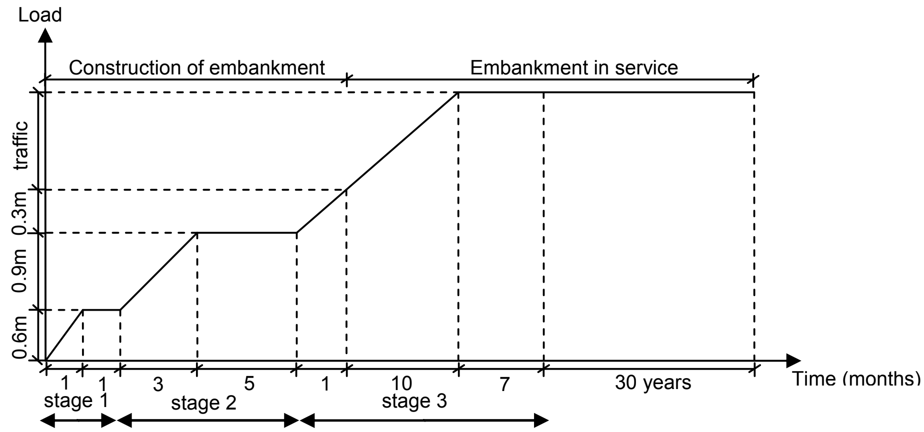

Figure 2: Sequential construction of the embankment.

4 DESCRIPTION OF THE NUMERICAL MODEL

4.1 Problem dimensions and model creation

Geometry and the boundary conditions of the two-dimensional Finite Element Model used for this case study are shown in Figure 3. ABAQUS/standard is used to model the problem and to simulate the staged construction procedure of the embankment. Due to symmetry of the problem, only half of the geometry is considered for the numerical model. This model of the embankment obey the rules of two-dimensional plane strain condition, because soil layers and applied traffic load are extended infinitely normal to the cross section shown in Figure 3.

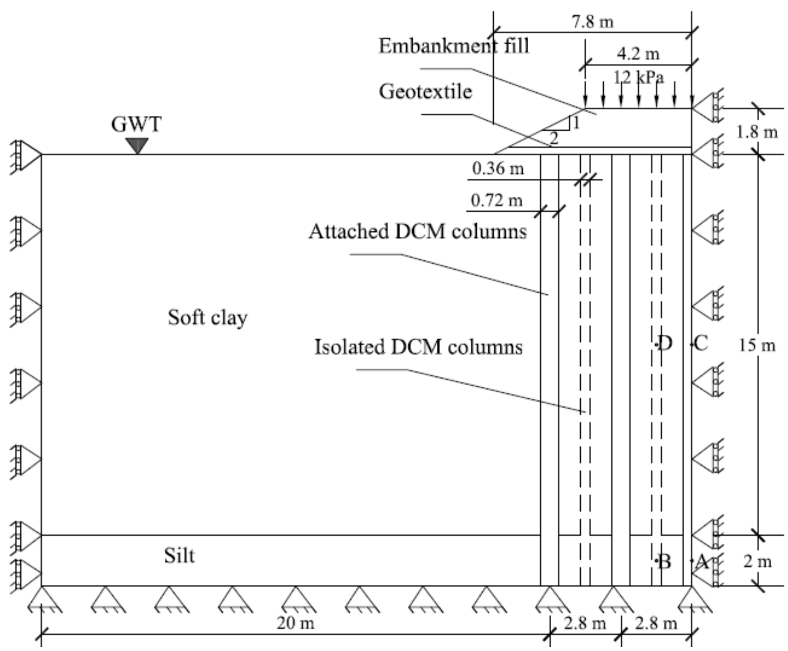

Figure 3: Numerical model of the embankment.

The geosynthetic reinforcement layer is modelled using truss elements as they have the capacity to transmit only axial forces without any bending moments. Since the three-dimensional problem is simulated using a two-dimensional plane strain model, isolated DCM columns are modelled as DCM walls. Therefore, the properties of isolated columns are kept constant while changing the width of the walls. The equivalent width of the DCM walls is calculated based on the following equation:

$$D' = \frac{\pi D^2}{4d} \tag{1}$$

where D' is the width of the equivalent DCM wall, D is the diameter of the isolated DCM column and d is the centre to centre distance in the direction perpendicular to the plane of the section considered. A number of finite element analyses were carried out with different element sizes. The mesh selected for this analysis gave the converged solution.

4.2 Material models and model parameters

Still there are uncertainties in relation to the strength envelope of cement admixed soil when used for the analysis of column-supported embankments. Huang & Han (2009) and Huang et al. (2009) assumed that the friction angle of cement admixed soil is zero. This assumption may be reasonable for their model because the load is applied instantaneously at the beginning of each loading stage similar to an undrained condition. However, in this numerical model, loads are applied as ramp loads during the staged construction and excess pore water pressures are allowed to dissipate similar to the actual field condition. Moreover cement admixed soils are not similar to fine grained soft clay, because the cementation will induce much higher permeability compared to the parent soft clay. Therefore the use of zero friction angle with the undrained shear strength is obviously not realistic for the analysis. In addition, when cement is mixed with soft clay, shear strength and friction angle of soft clay increase considerably (Broms 2003, Kivelo 1998). According to Broms (2003) and Kivelo (1998), cohesion of artificially cemented soil is considered to be equal to 0.289 times the unconfined compressive strength, q_u, which is 300 kPa for this case. The effective friction and the dilation angles used for the analysis are given in Table 1. Navin & Filz (2005) showed that the elastic modulus of DCM columns constructed by wet or dry mixing should be $300q_u$.. Bruce (2001) and Probaha et al. (2000) mentioned that this elastic modulus should be $100q_u$. Here, the elastic modulus of DCM columns is taken as $100q_u$, as it is more conservative.

Table 1: Material properties used in numerical model

Material	E (MPa)	ν	c' (kPa)	φ' (^{o})	γ (kN/m^3)	k (m/s)	ψ' (^{o})
Soft clay (NC)	0.3	0.2	8	13	18	6.342×10^{-11}	-
Silt	1.6	0.33	5	20	20	6.342×10^{-6}	-
Embankment fill	40	0.33	5	38	20	6.342×10^{-6}	-
Platform fill	20	0.33	5	32	20	6.342×10^{-6}	-
DCM columns	30	0.3	90	30	20	9.929×10^{-10}	5
Geosynthetic reinforcement	$J = Et$, J =1700 kN/m, c_i = 0.8, t = 30mm, tensile strength = 200 kN/m						

Note: E is tangential elastic modulus, ν is Poisson's ratio, γ is the unit weight (saturated and unsaturated unit weight for soils below and above the ground water table respectively), c' is the effective cohesion intercept, φ' is the effective friction angle, k is the coefficient of permeability, ψ' is the effective dilation angle, J is the tensile stiffness of the geosynthetic reinforcement, t is the thickness of the geosynthetic layer, c_i is the interaction coefficient between geosynthetic and platform fills

Huang & Han (2009) and Huang et al. (2009) simulated the constitutive behaviour of cement admixed soils assuming elastic perfectly plastic behaviour neglecting the strain softening behaviour. Furthermore it has been experimentally shown that the treated soil has a higher permeability than the same soil with the natural structure due to the higher void ratio sustained by artificially cemented soil (Chew et al. 2004, Lorenzo & Bergado 2004). This increase in permeability has not been considered by Huang & Han (2009) and Huang et al. (2009). However Shen & Miura (1999) showed that the coefficient of permeability of cement admixed soils in situ should generally be higher than the values measured in the laboratory due to the cracks and fissures in the DCM columns due to the shearing action of the rotating blades of augers during cement mixing. Therefore, DCM columns were assigned a higher permeability than the natural clay of the site as given in Table 1.

DCM columns are modelled using elasto-plastic Mohr-Coulomb criteria with isotropic strain softening behaviour, which has been incorporated into ABAQUS/Standard through the user defined field subroutine (USDFLD). The values used for the strain softening model in the analysis are $\varphi_p = 25^o$, $\varphi_{res} = 13^o$, $c'_p = 90$, $c'_{res} = 30$, $\psi = 5^o$, $\psi_{res} = 0^o$, $\gamma^p_{d,peak} = 2\%$ and $\gamma^p_{d,res} = 12\%$. These model parameters are selected after numerically simulating triaxial data for different cement admixed soils. The geosynthetic reinforcement is modelled as a linear elastic perfectly plastic material using the Von-Mises failure criteria and embankment fill, platform fill, soft soil, and firm soil were modelled as elastic perfectly plastic materials, using the Mohr-Coulomb failure criteria.

5 ANALYSIS OF RESULTS

5.1 Settlement on columns and clay layer and strain in geosynthetic layer

Forsman et al. (1999) has reported the time history of the settlement under the embankment at two different locations on the centre line and 1 m to the left of the centre line. These locations have two different cross sections. One has isolated columns and other one does not have isolated columns. Hence, two different finite element models were analysed with and without isolated DCM columns. But due to the page limit, results only for the case without isolated columns are discussed here. Figures 4 and 5 show the measured and finite element results for settlement profiles and strain in geosynthetic with and without strain softening behaviour.

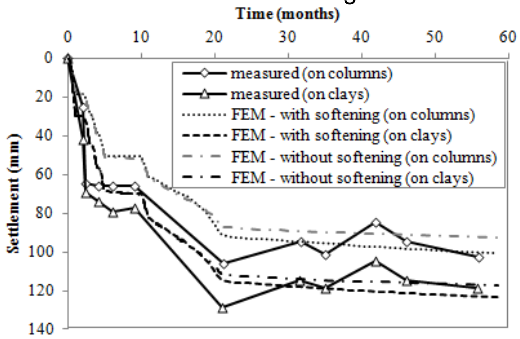

Figure 4. Consolidation Settlement with time

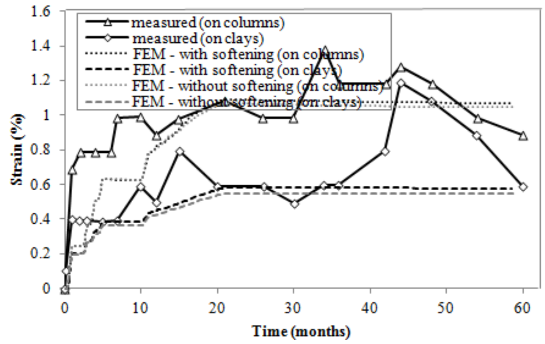

Figure 5. Strain in geosynthetic layer

For the construction stage 1, finite element results agree well with the measured settlements. The total settlement at the end of the second stage obtained from the finite element analysis is lower than the measured value. If the overall settlement profile is considered, it is clear that the computed settlements agree well with the field measurements. This suggests that the actual load transfer mechanism between geosynthetic reinforcement, DCM columns and foundation soil in the three-dimensional field situation is well captured by the two-dimensional plane strain modelling. Furthermore Figure 4 shows an increase in settlement after introduction of the strain softening behaviour of the artificially cemented soil. That is due to the breakage of cement-soil structure beyond yielding. However a significant difference in the settlement cannot be seen when the softening is incorporated as the load transferred to the columns is not sufficient to completely break down the artificial cement-soil structure.

Field measurements and numerical results of strain in geosynthetic are shown in Figure 5 and they are in good agreement with each other. Furthermore both measured and simulated results show that the strain of the geosynthetic layer over the columns is much higher than the strain in the geosynthetic layer over the soft foundation soil. With the introduction of the softening behaviour, strain in the geosynthetic increases due to transferring of part of the load, which is previously supported by columns, to geosynthetic and soft clay layers.

5.2 Tension in geosynthetic and excess pore water pressure in columns and clay layer

Figure 6 shows the variation of tension in the geosynthetic reinforcement layer along the width of the embankment at the end of the third stage and at the end of 30 years of service life. Tension gained at the end of the third stage remained constant within the service period as shown in Figure 6 and this is due to not including the creep effects for geosynthetic material in the finite element analysis. The maximum tension is concentrated around the columns confirming the field measurements (Forsman et al. 1999) and the previous findings from the numerical modelling (Huang et al. 2009). This also agrees with strains shown in Figure 5. The tension generated in the geosynthetic should be higher where the strain is higher. According to Forsman et al. (1999), the measured maximum tension of the reinforcement along the transverse direction of this embankment is 6 to 18 kN/m. However the finite element results yield a maximum tension of 18.04 kN/m, which is in a good agreement with field measurements. It is important to investigate the excess pore water pressure generation and dissipation during and after the construction of GRCS embankments. Figure 7 shows the variation of excess pore water pressure in the middle of the clay and silt layers (B & D) and at the same levels of DCM columns (A & C) closer to the centre of the embankment. This figure clearly explains how the excess pore pressure increases in each step as the embankment is constructed and the gradual dissipation during the waiting period due to the consolidation process. However, a substantial increase in pore pressure cannot be seen in this analysis due to the waiting periods involved with the actual construction sequence. The maximum excess pore water pressures are recorded at the end of the application of the traffic load and during the construction, which reduces the effective stresses within the clay layer and DCM columns. Soil strength depends on the effective stress state and hence the increased pore pressures will tend to reduce the strength of soils. Consequently clay layers and the DCM columns are highly susceptible to failure during or at the end of the construction or during the application of the traffic load. Therefore, it is not advisable to apply the traffic load just after construction of the embankment. The traffic load should be applied after the clay layers complete a significant proportion of the consolidation settlement.

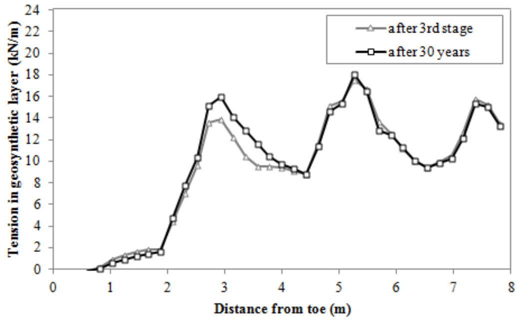

Figure 6. Tension in geosynthetic layer

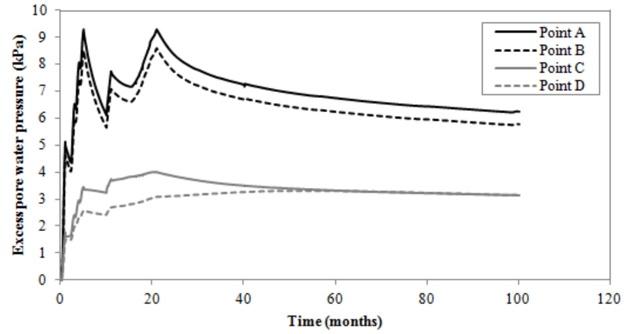

Figure 7. Calculated excess pore water pressure

The highest pore pressures are recorded for the DCM columns (points A & C) because the maximum stresses are concentrated on the DCM columns due to the higher stiffness of the cement admixed clay. Although excess pore pressure increases in each soil type just after application of load, there can be some deviations of the dissipation rates of the excess pore water pressure. This is mainly due to the difference in permeability of each soil type. In this study clay has a very low permeability and therefore its rate of excess pore water pressure dissipation is slow. This increase in excess pore water pressure due to loading and its dissipation have been well explained by Huang et al. (2009) using the Mandel-Cryer effect.

6 CONCLUSIONS

This study demonstrates that the two-dimensional coupled mechanical-hydraulic finite element modelling can describe the time-dependent behaviour of GRCS embankments during construction and post construction stages. Excess pore pressure predictions show the effectiveness of staged construction and the ability of DCM columns to strengthen the foundation soil by dissipating excess pore water pressure with the help of drainage and attracting stresses. Introduction of the strain softening behaviour increases the settlement in clay and DCM columns as well as the tension and strain developed in the geosynthetic. Therefore this material extension can be used to investigate the failure modes of GRCS embankments and thereby to evaluate the design procedures for these embankments against those for piled embankments.

7 ACKNOWLEDGEMENT

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