Numerical Modelling of a Deep Cement Mixed Column Supported Embankment

D.S. Liyanapathirana
School of Computing, Engineering and Mathematics, University of Western Sydney, NSW, Australia 2751
N.N.S. Yapage
Keller Ground Engineering, Baulkham Hills, NSW, Australia 2153
H.G. Poulos and R.B. Kelly
Coffey Geotechnics Pty Ltd., Chatswood, NSW, Australia 2067

ABSTRACT: This paper describes a case history of a deep cement mixed (DCM) column supported embankment. Field measurements of embankment settlements were significantly greater than predicted during the design stage, indicating that the DCM columns were overloaded and yielded. The computed settlement at the base is discussed and compared with field measurements. Results show that there is a good agreement between measured and computed embankment settlement when strain softening behaviour of DCM columns is included. These results clearly show that consideration of strain softening of DCM columns in the analysis is important if yielding occurs during or after construction of the embankment.

1 INTRODUCTION

Deep Cement Mixed (DCM) columns have been successfully used as an economical and fast-track soft ground improvement method for the construction of embankments. A large number of numerical studies have been carried out to understand the behaviour of column supported embankment systems but they assumed either linear elastic or linear elastic perfectly plastic constitutive behaviour for DCM columns. When cement is mixed, the stiffness and strength of in situ soil increases significantly showing a well-defined yield locus and beyond that strain softening occurs due to degradation of the soil-cement structure. Hence linear elastic or linear elastic perfectly plastic assumption for constitutive behaviour of columns may underestimate the settlements, produce unrealistic failure patterns, and overestimate the bearing capacity of the stabilised ground. However, there are no reported studies of embankments where DCM columns have yielded and numerical studies performed to calibrate strain softening constitutive models.

In this paper, a case history is presented, where the DCM columns are inferred to have yielded. The embankment was constructed over a soft clay deposit. Field measurements of embankment settlements were significantly larger than the settlements predicted during the design stage, indicating that the columns may have been weaker than expected. Two cases were analysed, with and without incorporating strain softening behaviour of DCM columns. The material parameters for the strain softening model have been selected considering a range of parameters extracted for cement stabilised Singapore Marine clay, Ariake clay and Hong Kong Marine clay based on triaxial tests (Yapage et al. 2013). The computed settlement at the base of the embankment is compared with field measurements.

2 DESCRIPTION OF THE EMBANKMENT

The column supported embankment was constructed over a flood plain with soft estuarine clay. Design constraint was such that the maximum settlement should not exceed 50 mm over 40 years. In order to achieve the above design criteria, an array of 0.8 m diameter columns with an area replacement ratio of 30% in a square arrangement was used beneath the crest for settlement control. Secant columns were set out in panels beneath the batters with an area replacement ratio of 25% to maintain stability.

Soil material parameters were estimated using a combination of in-situ piezocone and vane shear tests along with a laboratory testing programme designed to measure the moisture content, Atterberg limits and compressibility parameters. Based on these test results, the soil deposit can be divided into five layers: Top firm clay, soft clay, silty sand, firm clay and stiff to hard clay.

Design was performed based on the Swedish method described in SGF4:95E. Settlement was es-

timated assuming equal strain conditions within a unit cell. Beneath the crest of the embankment, the design ultimate shear strength of the columns, c_{col} was taken as 150 kPa, and beneath the batters it was assumed to be 100 kPa. The design stiffness of the columns was $200c_{col}$. Quality control (QC) testing comprised a combination of unconfined compressive strength (UCS) testing of cored samples in the laboratory and field pullout resistance tests (PORT) interpreted using an empirical N factor of 10. The acceptance criteria was that a minimum of 90% of the columns had to have a strength exceeding 150 kPa and the remaining 10% of the columns had to have a strength greater than 75% of the design ultimate strength. Production QC testing indicated that the columns had been successfully installed in accordance with the specification.
Predicted settlement during the design stage was 190 mm but the measured settlement in the field was 392 mm. For this to occur, it was inferred that the columns beneath the crest had exceeded their capacity and yielded.

3 FINITE ELEMENT ANALYSIS

3.1 *Problem dimensions and instrumentation*

The analysis was carried out using ABAQUS finite element program for the embankment section shown in Fig. 1. To improve the 8 m thick soft clay deposit, DCM columns with 800 mm diameter at 1.3 m spacing in a square configuration were used. Embankment height was 5.57 m and crest width is 18.95 m. The width of the embankment at the base was 30.09 m. The spacing between panels under the sloping sides was 3 m.

The embankment was instrumented with inclinometers, settlement plates (500 mm square), down-hole extensometers and vibrating wire piezometers prior to the construction as shown in Fig. 1. A settlement plate was located at 9.8 m away from the centre of the embankment and over the firm clay surface. A vibrating wire piezometer was positioned at 0.8 m away from the centre of the embankment and at a depth of 4.5 m from the ground surface to measure the excess pore water pressure in the mid depth of the soft clay layer.

3.2 *Boundary conditions and loading procedure*

The mechanical and hydraulic boundary conditions used for the analysis are illustrated in Fig. 1. A zero pore pressure boundary condition was applied at the top of the soft clay layer. Due to the symmetrical boundary condition imposed at the centre of the embankment, the hydraulic boundary condition at the centre was impervious. A fully drained condition was assumed for the embankment fill layers due to the high permeability of the fill material.

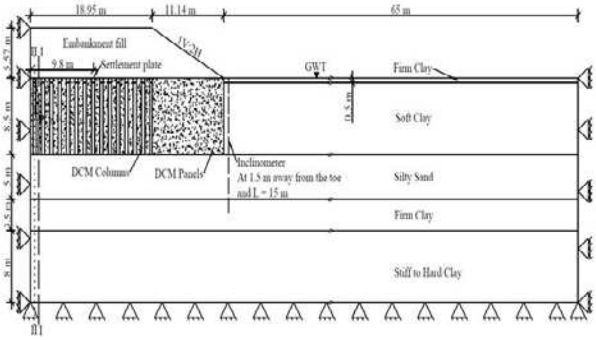

Fig 1 Cross section of the embankment

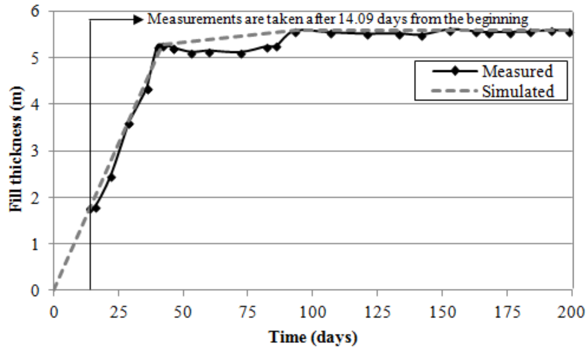

Fig 2 Measured and simulated construction procedure.

Staged construction of the embankment is shown in Fig. 2. Measurement of fill thickness was started when the fill height was 1.76 m. Fill height was linearly increased from 1.76 m to 5.27 m over 28 days followed by a waiting period of 30 days. Subsequently the fill thickness was increased by 0.3 m over 21 days to achieve the final embankment height of 5.57 m. The ground below the embankment was then allowed to consolidate.

3.3 *Conversion of the three-dimensional problem into a two-dimensional plane-strain model*

A three-dimensional model with a square column configuration can be simplified to a two-dimensional plane-strain model applying the equivalent area method as proposed by Ariyarathna et al. (2013). However, the equivalent area method cannot completely replicate the three-dimensional nature of the panel geometry underneath side slopes. Therefore, in this analysis, equivalent properties are assigned to model both panels and the soil between panels under the side slopes (Huang et al. 2009).

Table 1. Material properties for the embankment and subsoil.

Parameter	Unit	Top firm	Soft clay	Silty sand	Firm clay	Stiff	fill
Depth	M	0 - 0.5	0.5-8.5	8.5-13.5	13.5-17	17-25	-
Material model	-	MCC	MCC	MC	MC	MC	MC
Unit weight (γ)	kN/m^3	18	14.5	18	16.5	16.5	19
Coefficient of lateral earth	-	4.6	0.9 (0.5-4.5)	0.55	0.55	0.55	-
Void ratio (e_o)	-	2.0	3.0	2.23	2.0	2.0	-
λ	-	0.5	0.5	-	-	-	-
κ	-	0.053	0.053	-	-	-	-
Stress ratio (M)	-	0.98	0.98	-	-	-	-
Effective friction angle (ϕ')	Deg	25	25	30	25	25	30
Effective cohesion(c')	kPa	-	-	0	0	0	2
Poisson's ratio (ν)	-	0.3	0.3	0.3	0.3	0.3	0.3
Elastic modulus (E)	MPa	-	-	15	9	17	15
Permeability (k)	m/s	9.1x10^{-8}	6.0x10^{-8}	8.3x10^{-7}	6.0x10^{-10}	5.3x10^{-10}	-
OCR	-	135	1.3-4	-	-	-	-

Note: MCC – Modified Cam Clay, MC – Mohr Coulomb

The equivalent values of Young's modulus, cohesion and unit weight for panels are calculated based on the weighted average area of actual panels and the soil between them, while keeping the panel width same as in the three-dimensional geometry. Therefore, the area replacement ratio remained at 25% beneath the panels as in the field. The equivalent friction angle for panels in the plane-strain model is derived based on the force equilibrium method (Yapage et al., 2014).

3.4 *Material model and model parameters for the embankment and soil layers*

The soil properties and material models used for the analysis are summarised in Table 1. The top crust was overconsolidated and the soft clay layer below the top crust was normally to slightly overconsolidated with an OCR of 4-1.3. For the bottom firm clay layer and stiff to hard clay layers, field or laboratory test data are not available. Hence, they were estimated based on experience at similar site conditions. However, these two soil layers did not yield under the embankment load. Therefore, shear strength properties do not play an important role in this analysis.

3.5 *Material model and model parameters for DCM columns without strain softening*

The shear strength of the columns used for the finite element analysis was calculated from PORT test data using N = 13 (Liyanapathirana and Kelly, 2011). The average shear strength for the DCM columns was found to be 115 kPa, which gives an unconfined compressive strength of 230 kPa. This value agreed with results from the Unconfined Compressive Strength (UCS) tests carried out on core samples from DCM columns from 0 m to 5 m depth.

An eight-node reduced integration plane-strain element with pore pressure degrees of freedom at corner nodes (CPE8RP) has been used for the modelling of DCM columns, panels and soil layers. For embankment fill layers an eight-node reduced integration plane-strain element without pore pressure degrees of freedom (CPE8R) has been used.

Table 2. Material properties for DCM columns.

Parameter	Value/description
Depth	0 – 8.5 m
Material model	Extended MC
Unit weight	18 kN/m^3
Coeff. of lat.Earth	0.55
Void ratio	2.0
Effective friction angle	27^o
Effective cohesion	57.5 kPa
Dilation angle	0^0
Poisson's ratio	0.3
Elastic modulus	27.1 MPa
Permeability	6.0x10^{-8} m/s
Unconfined Comp. Str.	230 kN/m^2

3.6 *Material model for DCM columns incorporating strain softening*

An extended version of the Mohr-Coulomb constitutive model has been employed to simulate the strain softening behaviour of DCM columns. In this model, softening is introduced by varying the mobilized friction and dilation angles, and cohesion intercept as a linearly decreasing function of the deviatoric plastic strain as shown in Fig. 3. This material extension has been incorporated into ABAQUS/Standard finite element program through the user defined field subroutine, USDFLD (ABAQUS 2011).

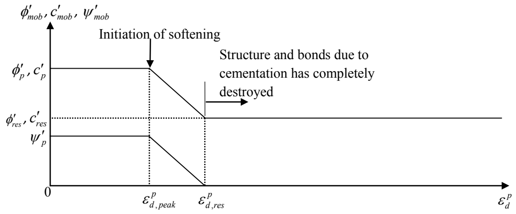

Fig 3 Variation of mobilized friction angle, ϕ'_{mob} ,cohesion, c'_{mob} , and dilation angle, ψ'_{mob} , with deviatoric plastic strain, ε_d^p .

Yapage et al. (2013) showed that there is no universal rule for peak and residual friction angles and cohesions, and corresponding plastic deviatoric strains. For the studied soil types, peak plastic shear strain varies between 1-4% and residual plastic shear strain varies between 4-15%. The ratio between residual strength properties to peak properties, which can be defined as the residual softening index (RSI), c'_{res}/c'_p and ϕ'_{res}/ϕ'_p vary between 0.4-0.7 and 0.4-0.8, respectively, when the cement content varies between 6% to 30%.

In the light of above discussion and by taking into consideration the 20% of cement mixed with soil at the site, RSI of 0.5 was assumed within the range of RSI observed for other cement stabilised soils. In addition, 1% and 12% are selected for the plastic deviatoric strains at peak and residual states, respectively.

4 COMPARISON OF FIELD PERFORMANCE WITH FINITE ELEMENT PREDICTIONS

4.1 *Settlement*

Settlements have been recorded in the field after placement of 1.76 m of the embankment and using a settlement plate located at 9.8 m away from the centre of the embankment and on the clay surface. Fig. 4 shows the finite element results for the settlement-time history curve over the soft soil surface and the column head close to the centre line of the embankment. In addition, Fig. 4 shows the comparison between measured and computed settlement using the finite element model at the base of the embankment, over the clay surface. The computed settlement including the strain softening behaviour of DCM columns agrees well with the field measurements. Due to strain softening, part of the load, which is supported by columns previously, transfers to the soft clay layer, increasing the settlement of soft clay. Therefore, if strain softening is excluded from the analysis, bearing capacity is over predicted over the improved ground if there is likelihood for column yielding. At the same time it would underestimate the load transferred to the unstabilised soft clay between columns. This entire phenomenon of strain softening is due to the breakage of cement-soil structure beyond yielding, which subsequently leads to the progressive failure of DCM columns.

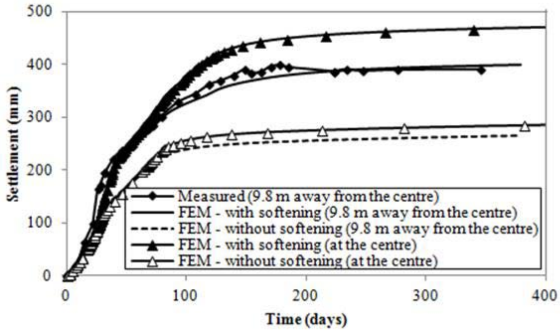

Fig 4. Measured and computed settlements.

Consequently, Fig. 4 clearly shows that the analyses without incorporating progressive failure of cement stabilised soils do not represent the actual behaviour of DCM columns when columns yield.

5 CONCLUSIONS

In the case history presented DCM column behaviour was modelled using an extended version of the MC model, which takes into account the strain softening behaviour of cement stabilised soils beyond yield. The settlement over the clay surface computed at the base of the embankment agreed well with the field measurements, confirming that the excessive settlements observed were due to column yielding and subsequent softening, a characteristic of cement stabilised soils.

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