Failure Modes for Geosynthetic Reinforced Column Supported (GRCS) Embankments

Les modes de rupture de massifs renforcés par colonnes sol-ciment et géosynthétique (GRCS) supportant des remblais

Yapage N.N.S., Liyanapathirana D.S., Leo C.J.
School of Computing, Engineering and Mathematics, University of Western Sydney, Locked Bag 1797, Penrith, NSW 2751, Australia

ABSTRACT: Deep cement mixed columns are widely used to support highway embankments constructed on soft compressible ground. Current design procedures for these embankments consider the sliding failure for external stability and the shear failure of deep cement mixed (DCM) columns for internal stability. Other failure modes such as collapse failure, slip circle failure, punching shear failure (overall or local) and bending failure of DCM columns are also significant for column supported embankments. However, still there are uncertainties in identifying the critical failure modes for these embankments. Hence, this paper investigates some failure modes for Geosynthetic reinforced column supported (GRCS) embankments using the finite element method. The embankment and traffic loads are gradually increased to bring the embankment to the verge of failure. Bending failure of DCM columns and subsequent shear failure for internal stability, local punching failure, overall punching failure and excessive total settlement failure are identified from the finite element analysis results and discussed in detail.

RÉSUMÉ : Les colonnes profondes réalisées par mélange sol-ciment sont très utilisées pour soutenir des remblais de route construits sur sol mou compressible . Les procédures actuelles de conception pour ces remblais considèrent la rupture par glissement pour la stabilité externe et la rupture par cisaillement des colonnes (DCM) sol- ciment pour la stabilité interne. D'autres modes de rupture tels que l'effondrement, le glissement circulaire, le poinçonnement de cisaillement (global ou local) et la flexion des colonnes de DCM sont également significatifs pour les remblais soutenus par colonnes. Cependant, il reste des incertitudes en identifiant les modes de rupture critiques pour ces remblais. Par conséquent, cette communication étudie quelques modes de rupture pour des remblais de GRCS par la méthode des éléments finis. Le remblai et les charges de la circulation sont graduellement augmentés pour s'approcher de la rupture.

KEYWORDS: deep cement mixed columns, embankment, finite element method, strain softening, progressive failure.

1 INTRODUCTION

Geosynthetic reinforced column supported (GRCS) embankments are widely used in infrastructure development projects in urban and metropolitan areas in most countries. The design process should critically consider the behavior of single columns as well as the global embankment system. A number of possible failure mechanisms for these embankments are discussed in the literature. Numerous research efforts have been expended to understand the failure modes for GRCS embankments using centrifuge modelling, numerical modelling and case histories of field performance (Broms 1999, Kitazume and Maruyama 2007) and thereby to develop analysis and design procedures incorporating possible failure modes. Current design procedures for these embankments only consider the sliding failure for external stability and the shear failure of DCM columns for internal stability (CDIT 2002, EuroSoilStab 2002). It is recently found that other failure modes such as collapse failure, slip circle failure, punching shear failure around column heads (overall or local), and bending failure of DCM columns are also significant for GRCS embankments (Kivelo 1998, Broms 2004, Kitazume and Mauyama 2007).

There are many case histories demonstrating that these foundation systems are likely to have slope stability problems, although they significantly improve the bearing capacity and reduce the excessive settlements inherent in soft ground. Progressive failure has been identified by Broms (2004) and bending failure has been observed by Terashi (2003).

The main focus of this paper is to identify the critical failure modes related to GRCS embankments. In this finite element analysis, the bending failure of individual columns and subsequent development of a slip surface shear failure are investigated. These failure modes are critical for internal stability of GRCS embankments. In addition, local and overall punching failure modes relevant for the stability of fill layers are investigated.

2 DESCRIPTION OF THE NUMERICAL MODEL AND MODEL PARAMETERS

The geometry of the problem used in this study is shown in Figure 1. The embankment is supported by DCM columns with 1 m diameter and 2.5 m center to center column spacing in each direction. The model parameters used for the analysis are given in Table 1. The embankment is constructed in stages expending 0.5 m fill layer followed by five 1 m thick layers. Each layer is applied over a period of one month and the waiting period after each fill layer is 0.5 months. Finally the traffic load is applied.

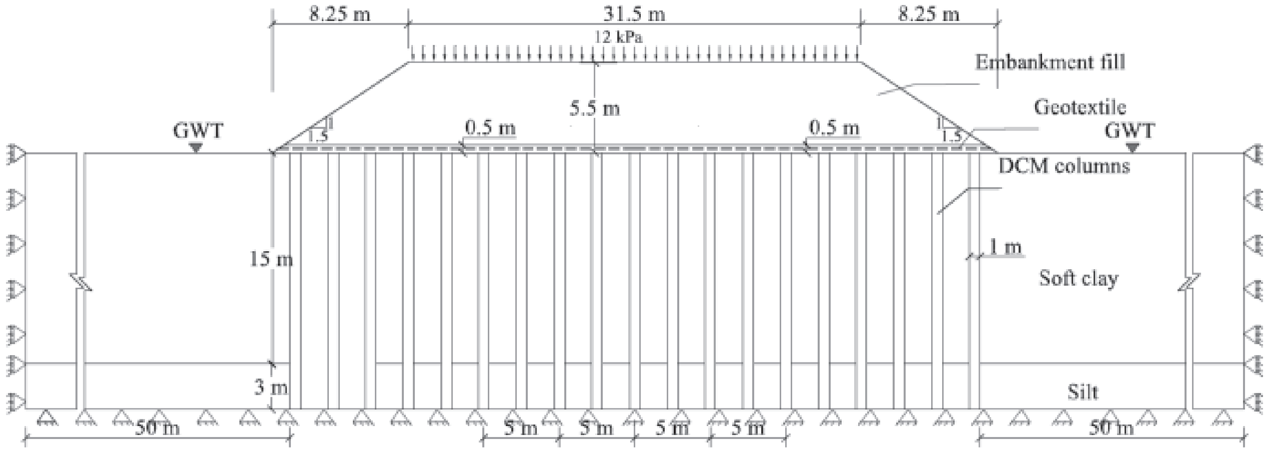

Figure 1.Geometry and boundary conditions for numerical model of the embankment.

The geosynthetic reinforcement is modelled as a linear elastic perfectly plastic material using the Von-Mises failure criteria and the embankment fill, platform fill, soft clay, and silt were modelled as elastic perfectly plastic materials, using the Mohr-Coulomb failure criteria.

An extended version of the Mohr-Coulomb model is used to simulate the strain softening behavior of the cement admixed soil (Yapage et al. 2012). This material extension has been incorporated into the finite element code, ABAQUS/Standard, through the user defined field subroutine, USDFLD.

The constitutive model is calibrated using triaxial test data found in the literature for cement admixed Singapore and Hong Kong marine clays. The parameters for the strain softening model in the analysis are peak friction angle, $\varphi_p' = 30^{\mathrm{o}}$, residual friction angle, $\varphi_{res}' = 13^{\mathrm{o}}$, peak cohesion, $c_p' = 90$, residual cohesion, $c_{res}' = 70$, peak dilation angle, $\psi_p' = 5^{\mathrm{o}}$, residual dilation angle, $\psi_{res}' = 0^{\mathrm{o}}$, Plastic deviatoric strain at peak, $\varepsilon_{d,peak}^{p} = 2\%$ and at residual, $\varepsilon_{d,res}^{p} = 12\%$.

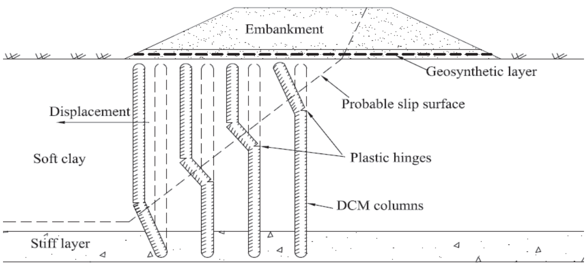

Figure 2. Failure mode of an embankment for internal stability (Broms 2004).

Table 1. Material properties used in numerical model.

Material	E (MPa)	ν	c' (kPa)	φ'(°)	γ (kN/m³)	k (m/s)	ψ'
Soft clay (NC)	0.3	0.2	8	13	18	6.34×10^{-11}	-
Silt	1.6	0.33	5	20	20	6.34×10^{-6}	-
Embankment fill	40	0.33	5	38	20	6.34×10^{-6}	-
Platform fill	20	0.33	5	32	20	6.34×10^{-6}	-
DCM columns	30	0.3	90	30	20	9.93×10^{-10}	5
Geosynthetic reinforcement	$J = Et$, J = 1700 kN/m, c_i = 0.8, t = 30 mm, *tensile strength* = 200 kN/m						

Note: E is tangential elastic modulus, ν is Poisson's ratio, γ is the unit weight, c' is the effective cohesion intercept, φ' is the effective friction angle, k is the permeability, ψ' is the effective dilation angle, J is the tensile stiffness of the geosynthetic, t is the thickness of the geosynthetic layer, c_i is the interaction coefficient between geosynthetic and platform fills

3 IDENTIFICATION OF FAILURE MODES USING FEM

Various instability criteria to identify the failure state during numerical analysis can be found in the literature: (i) Abrupt increase in nodal displacements or deformations at a certain location of the embankment, (ii) Initiation, development and distribution of plastic strain, shear strain or yielded material zone in a particular location and (iii) Non-convergence state within a user-defined maximum number of iterations for the solution. In this research first and second criterion are used to identify the failure mechanisms.

4 FAILURE MODES ASSOCIATED WITH EMBANKMENTS SUPPORTED OVER DEEP CEMENT MIXED COLUMNS

4.1 Combination of bending and shear failure modes

In this study, it is found that the bending failure and subsequent slip surface shear failure are critical for internal stability of GRCS embankments. Broms (2004) illustrated the probable slip surface for columns located in the active zone as shown in Figure 2. Therefore, the analysis is carried out considering the full cross section of the embankment giving allowance to develop an asymmetric slip surface.

The plastic hinge formation within the finite element model is shown in Figure 3. When the shear strain development with gradual loading is investigated, higher shear strains initially develop closer to the top of the columns at the center of the embankment and then they progressively develop towards the bottom of the columns closer to the embankment toe. During this process, DCM columns fail one by one due to bending failure. When the maximum bending moments within the columns exceed the moment carrying capacity of columns, plastic hinges will develop at these locations as illustrated in Figure 3. The soft soil in between these columns experience considerable shear distortions due to abrupt deformation of damaged columns. The resulting slip surface is not circular and it is a slip band with a certain thickness as shown in Figure 3.

Columns closer to the embankment toe have a single plastic hinge, while the middle columns have two plastic hinges with approximately same distance in between them. When there are two plastic hinges developed in the column, one should be at the location of the maximum positive bending moment and the other one should be at the location of the maximum negative bending moment. It can be observed that this failure mechanism agrees well with the critical slip surface given by Broms (2004) shown in Figure 2.

The bending failure mode mainly depends on the tensile strength of DCM columns. According to Figure 4, axial loads acting on columns induce compressive stresses within the column cross section, while the moment load induces both compressive and tensile stresses. Therefore, stress distribution within the column cross section may experience tensile stresses, depending on the magnitude of bending and axial stresses acting on columns. DCM columns fail when the resultant tensile stress exceeds the tensile strength of columns.

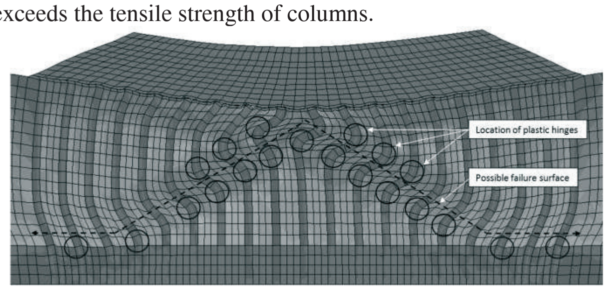

Figure 3. Deformed shape of the finite element model.

According to Broms (2004), the tensile strength of DCM columns are typically 10 to 20% of the unconfined compressive strength. However EuroSoilStab (2002) recommended that the columns created by dry method should not be subjected to tensile stresses due to the uncertainty in the tensile strength of DCM columns. Navin (2005) also recommended that the columns should be designed to satisfy the zero tensile stress condition at any point across the column cross section.

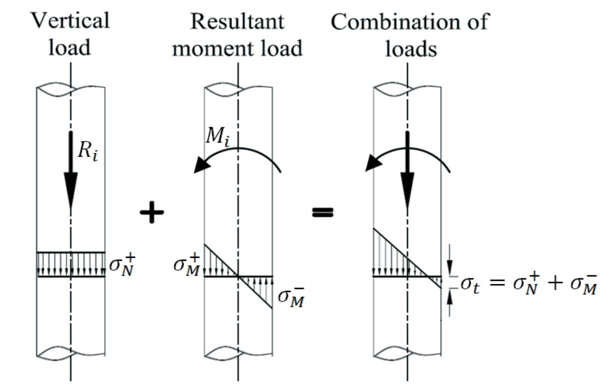

Figure 4. Induced stress distribution in DCM columns.

To avoid negative stress conditions;

$$\sigma_N^+ + \sigma_M^- > 0 \tag{1}$$

σ_N^+ and σ_M^- can be defined as follows.

$$\sigma_N^+ = \frac{R_i}{\pi D^2/4} \tag{2}$$

$$\sigma_M^+ = \sigma_M^- = \frac{M_i y}{I} = \frac{M_i}{\pi D^3/32} \tag{3}$$

where R_i is vertical load and M_i is resultant moment applied on columns.

The bending strength or the resistance against bending is mainly governed by the tensile strength of DCM columns. Numerical results show that the axial load is low for the columns closer to the toe of the embankment compared to middle columns. Therefore bending failure of columns is likely to initiate closer to the toe of the embankment. The geosynthetic reinforcement provides a resisting moment against the moment induced by the lateral earth pressure to reduce the tensile stress developed within the columns. Therefore, the geosynthetic reinforcement plays a significant role in resisting the bending failure of columns. Additionally, closer column spacing, larger diameter columns, or reinforcing the columns with steel bars or cages, can be used to withstand the tensile stresses developed within the DCM columns and thereby to protect columns against bending failure (Wong and Muttuvel 2011).

Kitazume (2008) proposed a simple stability calculation to assess embankments over improved grounds against ultimate bending failure. However, he has not considered the traffic load over the crest and the tension developed within the geosynthetic layer. He assumed that the envelope of failure plane of columns is horizontal. However, the failure plane is an inclined plane as shown in Figure 3. Therefore, a new stability equation should be developed against the bending failure considering the inclined slip surface. In that equation, the active earth pressure due to the embankment load, P_{ae}, soft clay, P_{ac}, and the traffic load, P_{at}, should be considered as shown in Figure 5 to calculate the driving moment. Resisting moment should consist of the contributions from passive earth pressure of the soft clay, P_{pc}, embankment and traffic load over the columns, P_{el}, self-weight of columns, P_{sw}, tension in the geosynthetic, T_{gs}, the skin friction mobilized along the surface of columns and the shear strength of clay between columns as shown in Figure 5. The resultant of driving and resisting moments due to loads applied on columns should not exceed the bending strength of DCM columns.

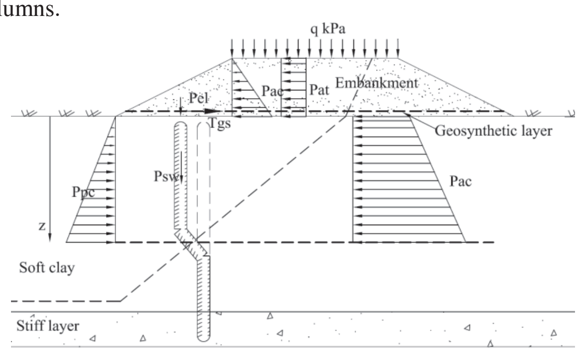

Figure 5. Load distribution over columns for embankment in consideration for the bending failure analysis.

It is important to determine how the gradient of this failure line varies with geometry and material properties of the embankment. To achieve this, a detailed parametric study needs to be carried out before developing analytical equations for the stability calculation against bending failure.

4.2 Punching failure of fill layers

It is important to investigate the failure modes related to embankment fill layers such as punching, slip circle or lateral spreading. However, only punching failure is discussed in this paper.

The punching failure can be categorized into two types: local punching failure and overall punching failure. When column heads are considered, the clay in between columns settles more than the columns. Therefore, it is possible for column heads to penetrate into the fill layers, which is known as the local punching shear failure. If overall punching shear failure occurs, it is clearly visible at the crest of the embankment, developing an irregular surface with "humps" at the column locations and "depressions" in between columns. Punching failure can be identified from the excessive shear strains above the columns, and excessive differential settlements at the base of the embankment in numerical modelling.

To identify these failure modes, two different numerical analyses were carried out with two different embankment height to clear spacing ratios. One embankment is 5.5 m high and other one is only 2.5 m high. The columns have 1 m diameter and the centre to centre spacing is 2.5 m in each case. The computed settlement profiles at the crest and the base of the embankment during 30 years of service life are shown in Figures 6 and 7 for low (2.5 m) and high embankments (5.5 m), respectively.

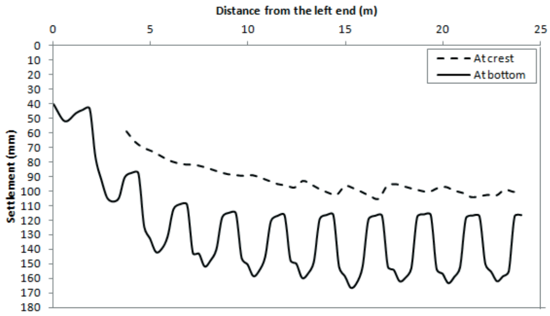

Figure 6. Settlement profile for 2.5 m high embankment.

Figure 6 clearly illustrates humps and depressions at the crest of the low embankment. Consequently overall punching failure is possible and this might be the crucial factor in determining the loss of serviceability of the embankment. Therefore overall punching is critical when the embankment height is low.

For the high embankment, even though there is a considerable differential settlement at the base of the embankment, it has not been transferred to the crest of the embankment and produced a fairly even embankment crest (Figure 7) showing the possibility of local punching failure. Therefore embankments with higher fill thickness relative to the column spacing are vulnerable to local punching failure.

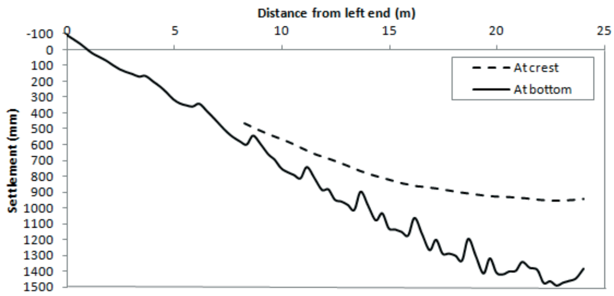

Figure 7. Settlement profile for 5.5 m high embankment.

Placing a stiffer geo-membrane immediately on top of the columns can mitigate local punching failure. Overall punching failure can be minimised by increasing the efficacy of columns, area ratio of columns, stiffness of the geosynthetic, thickness of the load transfer platform by placing more layers of geosynthetic, and embankment height relative to the column spacing to develop effective soil arching.

From the numerical results, it could be identified that the critical height to clear spacing ratio is important in controlling the overall punching shear failure. This ratio can be used to ascertain the development of full arching within the fill layers and thereby to ensure that there are no localized differential settlements at the crest of the embankment. However, the critical height defined in current design guidelines is not consistent.

4.3 Failure due to excessive total foundation settlement

According to Figure 7, it is clear that for high embankments, excessive total foundation settlement is more crucial than the differential settlement. Excessive foundation settlement can be more problematic for high embankments especially with floating DCM columns where columns penetrate partly into the clay layer without reaching a stiff base layer. Therefore, embankment design practice should also aim to prevent failure due to excessive total foundation settlement.

5 CONCLUSIONS

This paper investigated possible failure modes for GRCS embankments. The finite element results show that the bending failure is a critical failure mode for internal stability. Once the plastic hinges are formed, the embankment fails due to propagation of a slip surface, which is mainly governed by the tensile strength of the columns. Some weaknesses in existing analytical equations for calculation of stability against bending failure are identified and parameters to be considered for a new stability calculation are proposed. Overall punching failure is critical for low embankments and local punching failure is crucial for high embankments. It is important to establish a reliable equation for the critical height, considering different column layouts and geometries to avoid overall punching failure. High embankments are vulnerable to excessive total foundation settlement and therefore necessary precautions should be taken in the design process. Overall this paper identified some failure modes to be considered in the development of design procedures to evaluate the overall stability of GRCS embankments and proposed some future research directions to improve the current design practice.

6 ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support for this research provided by the Australian research council and Coffey Geotechnics Pty Ltd under LP099058.

7 REFERENCES

Broms, B.B. 2004. ‘Lime and lime/cement columns’, in *Ground Improvement* Ed. Moseley, M.P. and Kirsch, K. Spon Press, London, 252-330.

CDIT (Coastal Development Institute of Technology). 2002. ‘*The Deep Mixing Method: Principle, Design and Construction’*, A.A. Balkema: The Netherlands.

EuroSoilStab 2002. ‘*Development of Design and Construction Methods to Stabilise Soft Organic Soils’*. Design Guide Soft Soil Stabilization, CT97-0351, Project No: BE 96-3177.

Kitazume, M. 2008. ‘*Stability of group column type DM improved ground under embankment loading behavior of sheet pile quay wall’*, Report of the port and airport research institute, Nagase, Yokosuka,Japan, 47(1): 1-53.

Kitazume, M. and Maruyama, K. 2007. 'Internal stability of group column type deep mixing improved ground under embankment', *Soils and Foundations,* 47(3):437-455.

Navin, M. 2005. *'Stability of embankments founded on soft soil improved with deep mixing method columns'*, Doctor of Philosophy thesis, Virginia polytechnic institute and state university.

Terashi, M. 2003. 'The state of practice in deep mixing methods.', Proceedings of the 3rd International Conference on *Grouting and Ground Treatment*, New Orleans, 25-49.

Wong, P. and Muttuvel, T. 2011. 'Support of road embankments on soft ground using controlled modulus columns', Proceedings of *Int.Conf. on advances in geotech. eng.*, Perth, Australia, Nov.7-9.

Yapage, N.N.S., Liyanapathirana, D.S., Poulos, H.G., Kelly, R.B. and Leo, C.J. 2012. ‘2D numerical modelling of geosynthetic reinforced embankments over deep cement mixing columns’, *11th ANZ conference on Geomechanics* , Melbourne, Australia, 578-583.