INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

SIMSG ISSMGE

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Analysis of the Performance of Soft Ground Stabilized by Stone Columns

S. Basack, F. Siahaan, B. Indraratna and C. Rujikiatkamjorn
Centre for GRE, University of Wollongong, NSW, Australia

ABSTRACT: In this paper, an axisymmetric solution has been presented to study the performance of soft ground reinforced with stone columns. The arching effect initiated by column-to-soil stiffness ratio and the influence of fines intrusion were considered in the analysis. The radial consolidation aided with modified Cam-clay theory has been adopted. The comparison of computed results with available experimental and field test data indicates accuracy of the analyses. Using the model, important parametric studies were conducted and relevant conclusions drawn.

1 INTRODUCTION

Soft ground improvement by installing stone columns has numerous benefits including increased bearing capacity, accelerated consolidation, increased slope stability and liquefaction control (Bouassida and Carter 2014). The stone columns not only acts as vertical stiffening members increasing the overall strength and stiffness of the soft ground, but they also assist in effective radial consolidation (Fatahi *et al.* 2012).

The foundations supporting transport infrastructure necessitates special considerations including the flexibility of surcharge loading and the influence of cyclic loading. Due to the flexibility of transport embankments, adoption of free strain hypothesis is likely to better match the field conditions than an equal strain approach, although an intermediate state between the free strain and equal strain conditions is observed in the field (Basack 2010; Indraratna *et al*. 2013).

During column installation, the cavity expansion results in an initial fines intrusion. After preloading, migration of fines into the pores of the granular column inevitably occurs because of the high hydraulic gradient at the soil-column interface, resulting in further fines intrusion (Adalier *et al.* 2004). The fines intrusion initiates reduction in the effective radius of the column in terms of drainage as well as reducing the permeability in the zone with fines. Apart from fines intrusion, a zone of disturbed soil is produced around the stone column due to its installation, specifically denoted as smear zone.

2 NUMERICAL MODELLING

An advanced numerical model (finite difference technique) based on free strain hypothesis and unit cell analogy has been developed, based on a previously developed model published elsewhere (Indraratna *et al.* 2013).

2.1 *Mathematical Formulations*

Fig. 1(a) portrays the problem, where the soft clay layer of thickness H overlays an impervious rigid boundary and subjected to a surcharge loading, has been improved by a group of stone columns; the radii of the column and the unit cell are r_c and r_e respectively. Considering the phenomena of smear and fines intrusion, the cross section of the entire unit cell has four distinct zones (see Fig. 1b): column zone without fines, column zone with fines, smear zone and the outer undisturbed soil zone.

For finite difference analysis, the soil mass within the unit cell has been divided both radially as well as vertically into a large number of equal divisions n_r and n_z respectively (see Fig. 1d). The total computational time t_t as well was equally discretized into n_t divisions. The separators were thereby denoted as nodes.

The numerical analysis was based on the assumptions of purely vertical deformations of the column and the soil and only radial consolidation. The stone column in the zone without fines was assumed as a freely draining material.

For column to soil load transfer, the phenomenon of arching was considered, where the vertical stress distribution (Fig. 1c) in the column and the soil were derived as:

$$w_{col} = n_s\, w(r_c) \tag{1a}$$

$$w(r) = w(r_s) + (N - r/r_c)^2 F(N, n_s) \tag{1b}$$

where, w_{col} = stress on column; $w(r)$ = stress on soil at a radial distance of r; n_s = steady-state stress concentration ratio; $N = r_e/r_c$.

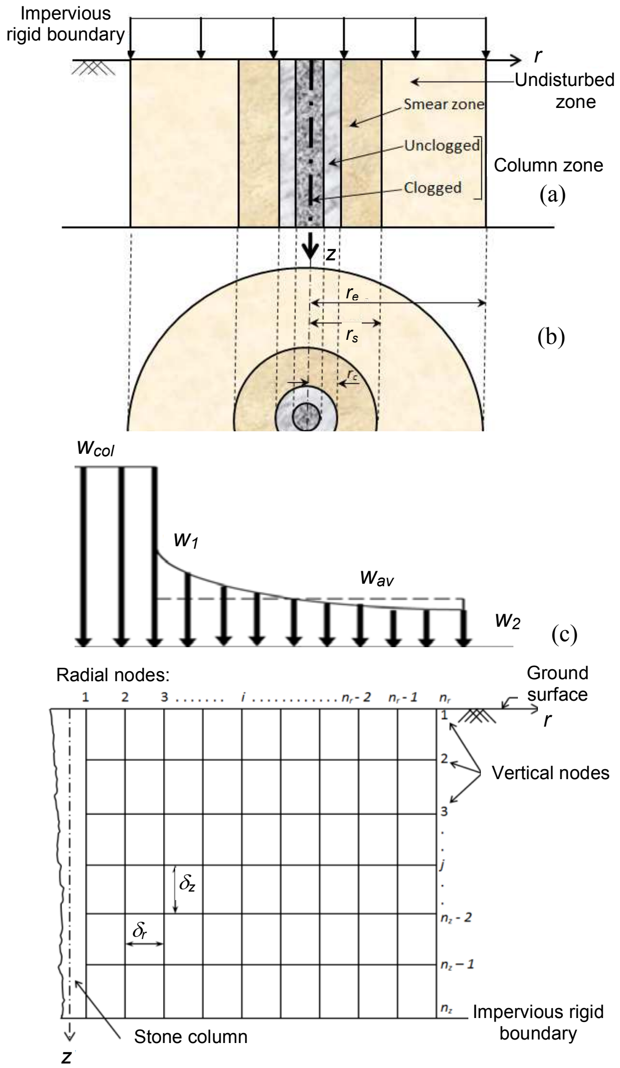

Fig. 1 The idealized problem: (a) Longitudinal section of unit cell. (b) Cross section of unit cell. (c) Vertical stress distribution. (d) Finite difference discretization.

For radial consolidation, the Barron's (1948) theory has been adopted and the relevant differential equation is shown below:

$$\frac{\partial u_{rt}}{\partial t} = c_{vr}\left(\frac{1}{r}\frac{\partial u_{rt}}{\partial r} + \frac{\partial^2 u_{rt}}{\partial r^2}\right) \tag{2}$$

where, u_{rt} = excess pore water pressure, t = time and c_{vr} = radial consolidation coefficient. To incorporate the influence of non-linear correlation between soil void ratio and effective stress, the modified Cam-clay theory (Roscoe and Burland 1968) has been applied. The expression for soil compressibility has been deduced as:

$$m_v = -\frac{\lambda p_0'}{\left[w(r) + \gamma' \frac{1}{H}\int_0^z p' dz - u_{rt}\right]\left[1 + e_0 - \lambda \ln\left(p'/p_0'\right)\right]} \tag{3}$$

where, m_v = volumetric soil compressibility; λ = slope of the *e-ln p'* curve; e_0 = initial void ratio corresponding to the unit effective pressure p_0'.

The soil settlement has been evaluated as:

$$\rho_{rzt} = -\int_0^t \int_z^H m_v \frac{\partial u_{rt}}{\partial t}\, dz\, dt \tag{4}$$

For simplification of analysis, the smear zone radius is 1.2 times the column radius (Weber *et al*. 2010) and the horizontal permeability is 0.1 times that in the undisturbed zone.

The fines intrusion initiates reduction in the effective radius of the column in terms of drainage as well as reducing the permeability in the zone with fines. Although fines intrusion is essentially time-dependent during the process of consolidation, the parameters accounting for fines' intrusion were assumed constant during consolidation, which can be expressed as (Indraratna *et al*. 2013):

$$r_c' = \alpha\ r_c \tag{5a}$$

$$k_{cl} = \alpha_k k_s \tag{5b}$$

where, r_c' = reduced radius of column in terms of drainage; α = a non-dimensional factor in the range $0 < \alpha \leq 1$; α_k = a non-dimensional parameter representing the ratio of horizontal permeability of the column zone with fines to that of the smear zone (k_{cl} and k_s respectively), the suggested range of which is $0 < \alpha_k < 1$.

2.2 *Numerical Results and Discussions*

The Malaysian Highway Authority constructed test embankments on Muar Plain, Malaysia (Redana 1999) with different ground improvement techniques including sand compaction piles (SCP). The SCP possess greater stiffness compared to the surrounding soft soil, so likely to behave in the same way as stone columns from the viewpoint of load transfer mechanism. Therefore, the field observations and the finite element results are compared with the present model. The soil and pile parameters were estimated as the weighted average and ignoring the drainage through the base layer.

The steady-state stress concentration ratio n_s has been taken as 4.7 (Basack 2010). Analysis was carried out with and without fines (for fines intrusion, $\alpha = 0.8$; $\alpha_k = 1.0$). The input parameters chosen for computations taken from Redana (1999) are presented in Table 1. The results from the model is observed to yield close proximity to the field observations for the time ranges of $0 < t < 200$ days and $300 < t < 450$ days. With the chosen parameters for fines intrusion, the predicted settlements are observed to be even closer to the field measurements. The average deviation between computed and field settlements is within 10%.

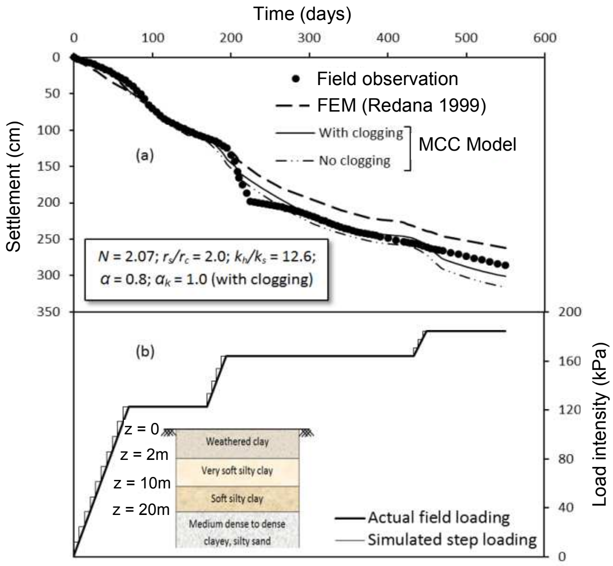

Fig. 2 (a) Comparison between computed and field observations. (b) Actual and simulated load intensities.

The variation of normalized excess pore water pressure in the space-time frame has been presented in Fig. 3. As observed from Fig. 3(a), the nodal excess pore water pressure decreases with time following an asymptotic stabilizing tendency, the rate of decrease being quite sharp in the range of $0 < T < 3$ days. With respect to radial distance, on the other hand, the nodal pore water pressure increases in a parabolic manner (see Fig. 3b), the slope being zero at the unit cell boundary. This observation is in accordance with the Eq. (2) above.

The influence of parameters accounting for fines intrusion on the excess pore water pressure dissipation has been studied, as shown in Fig. 4. As observed, the descending values of parameter α produces retarding effect on the excess pore water pressure dissipation. Moreover, the parameter α_k induced further retardation on the dissipation. This is justified by the chosen formulations (see Equations 5a and 5b) regarding reduction in effective diameter and permeability of the column in terms of radial drainage.

Table 1. Computational data

k_h	m_v	e_0	λ	K_0	K_p	r_c	r_e
2.7 x 10^{-9} m/s	10^{-6} m^2/N	2.2	0.38	0.8	3.0	0.6 m	1.24 m

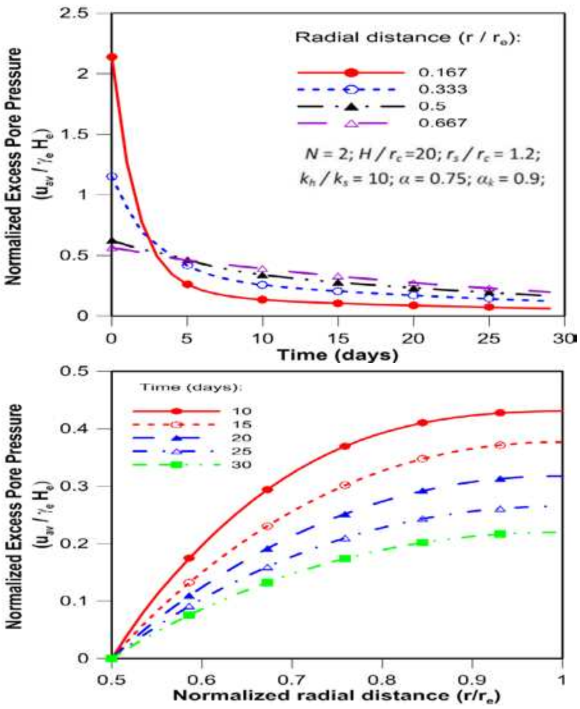

Fig. 3 Variation of normalized excess pore water pressure with: (a) time; (b) normalized radial distance

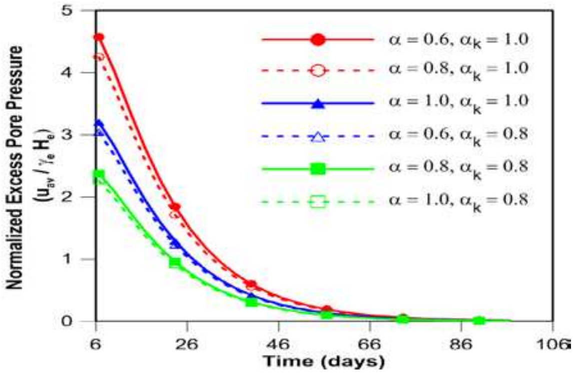

Fig. 4 Variation of normalized average excess pore pressure with time for different fines intrusion parameters

3 LABORATORY MODEL TESTS

As a part of the laboratory testing programme, one-dimensional consolidation tests were carried out on a unit cell which comprised of soft kaolin clay and stone column installed into the consolidated test bed by means of replacement method. The details of test set up are published elsewhere (Siahaan *et al.* 2014). The clay was pre-consolidated under a vertical pressure of 55 kPa. The pressures on the unit cell surface have been applied in several steps, with a final steady value of 135 kPa. The average settlement of the unit cell surface was recorded by means of a group of LVDT and the data was taken for up to 38 days. The measured settlements have been compared with the numerical results obtained by the present model. The computational input parameters taken from the test data are as follows: r_c = 0.05 m, r_e = 0.15 m, H = 0.6 m, $k_h = 10^{-9}$ m/s and $m_v = 2 \times 10^{-6}$ m^2/N. The values of modified Cam-clay soil parameters were varied as $1.5 < e_0 < 2$ and $0.5 < \lambda < 0.6$. The measured and computed surface settlements have been depicted in Fig. 5.

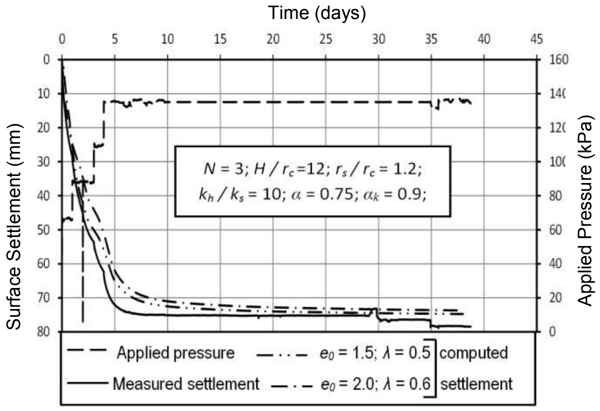

Fig. 5 Comparison between experimental and surface settlements of unit cell.

Acceptable agreement has been noted between the experimental and numerical values of the surface settlements with an average deviation below 10%, the computed settlements being on the lower side. The majority of the deviation between measured and computed data have been observed in the time range of 3 – 10 days. The experimental settlement stabilized after about 6 days, against about 12 days relevant to the computed settlement. The values of numerical settlements were found to alter with the variation of modified Cam-clay (MCC) model parameters e_0 and λ. The simplified assumption of purely radial pore water flow during consolidation, in addition to the assumed values of soil parameters relevant to fines intrusion and MCC model, have attributed to the discrepancy between measured and computed settlements.

4 CONCLUSIONS

A numerical model based on advanced finite difference technique adopting free strain hypothesis and aided with modified Cam-clay theory was developed considering arching, fines intrusion and smear effects. Comparison of the model with available field test data and experimental results justifies the validity of analysis carried out. The study indicates that the nodal excess pore water pressure decreases with time following an asymptotic stabilizing tendency and increases in a parabolic manner with respect to radial distance. The parameters accounting for fines intrusion α and α_k produce retarding effect on the excess pore water pressure dissipation. Comparison between experimental and computed settlements indicated a good agreement with an average deviation below 10%. The alteration of MCC soil parameters affects the numerical results significantly.

ACKNOWLEDGMENTS

The Authors thankfully acknowledge the financial supports received from the Australian Research Council (ARC) and industry partners, namely Coffey Geotechnics and Keller Ground Engineering, in the form of an industry linkage project.

REFERENCES

Adalier, K. and Elgamal, A., (2004)., Mitigation of liquefaction and associated ground deformations by stone columns, Engineering Geology, 72(3-4): 275-291.

Basack, S. (2010). A study on effectiveness of vertical drains for improving long term performance of soft clay deposits with particular reference to stone columns, Final Technical Report, Endeavour Postdoctoral Research Fellowship, 1524_2010, CGRE, University of Wollongong, Australia.

Basack, S., Indraratna, B. and Rujikiatkamjorn, C. 2011 Design recommendation for stone column reinforced soft clay deposit, PanAm Conf. on Geotech. Eng., Toronto, Canada.

Barron, B. A., (1948)., Consolidation of fine grained soil by drain wells." Transactions of American Society of Civil Engineers, 113(2346): 712–748.

Bouassida, M., and Carter, J., (2014)., Optimization of design of column-reinforced foundations, International Journal of Geomechanics, 14(6), 04014031.

Fatahi, B., Basack, S., Premananda, S., and Khabbaz, H., (2012)., Settlement prediction and back analysis of young's modulus and dilation angle of stone columns, Australian Journal of Civil Engineering, 10 (1): 67-80.

Indraratna, B., Basack, S. and Rujikiatkamjorn, C., (2013)., Numerical solution to stone column reinforced soft ground considering arching, clogging and smear effects, J. of Geotech. and Geoenvironmental Eng., 139 (3): 377-394.

Roscoe, K. H. and Burland, J. B. (1968). "On the Generalised Stress-Strain Behaviour of 'Wet' Clay", In Heyman, J., and Leckie, F. A. eds., Engineering Plasticity, Cambridge University Press: 535-609.

Redana, I. W. (1999), Effectiveness of vertical drains in soft clay with special reference to smear effect, PhD thesis, GRE, University of Wollongong, Australia.

Siahaan, F., Indraratna, B., Rujikiatkamjorn, C. and Basack, S. 2014 Vertical stresses in stone column and soft clay during one-dimensional consolidation test. Proceedings of Soft Soils 2014, Indonesia.

Weber, T. M., Plotze, M., Laue, J., Peschke, G., and Springman, S. M. 2010 Smear zone identification and soil properties around stone columns constructed in-flight in centrifuge model test, Geotechnique 60 (3): 197 – 206.