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Analysis of the Performance of Soft Ground Stabilized by Stone Columns

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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 act 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 pre-loading, 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 \( t \) 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:

\[
\begin{align*}
\text{w}_{\text{col}} &= n_z \text{w}(r_e) \\
\text{w}(r) &= \text{w}(r_e) + (N - r/r_e)^2F(N, n_z)
\end{align*}
\]

where, \( \text{w}_{\text{col}} \) = stress on column; \( \text{w}(r) \) = stress on soil at a radial distance of \( r \); \( N = r_c/r_e \).


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:

\[
\frac{\partial u_r}{\partial t} = c_{vr} \left( \frac{1}{r} \frac{\partial u_r}{\partial r} + \frac{\partial^2 u_r}{\partial r^2} \right)
\]  

(2)

where, \( u_r \) = 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:

\[
m_s = \frac{\lambda p'_0}{\left[ w(r) + \gamma' \frac{1}{H} \int_0^r \rho' dz - u_r \right] \left[ 1 + e_0 - \lambda \ln \left( \frac{p'}{p'_0} \right) \right]}
\]  

(3)

where, \( m_s \) = volumetric soil compressibility; \( \lambda \) = 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:

\[
\rho_{set} = \int_0^H m_r \frac{\partial u_r}{\partial t} \, dz \, dt
\]  

(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):

\[
r_c' = \alpha r_c
\]  

(5a)

\[
k_{cl} = \alpha_k k_s
\]  

(5b)

where, \( r_c' \) = reduced radius of column in terms of drainage; \( \alpha \) = a non-dimensional factor in the range \( 0 < \alpha \leq 1 \); \( \alpha_k \) = 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%.
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 $\alpha$ produces retarding effect on the excess pore water pressure dissipation. Moreover, the parameter $\alpha_k$ 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

<table>
<thead>
<tr>
<th>$k_h$</th>
<th>$m_v$</th>
<th>$e_0$</th>
<th>$\lambda$</th>
<th>$K_0$</th>
<th>$K_p$</th>
<th>$r_c$</th>
<th>$r_e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$2.7 \times 10^{-9}$</td>
<td>$10^{-6}$</td>
<td>2.2</td>
<td>0.38</td>
<td>0.8</td>
<td>3.0</td>
<td>0.6</td>
<td>1.24</td>
</tr>
</tbody>
</table>

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

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 preconsolidated 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: \( r_e = 0.05 \text{ m}, r_c = 0.15 \text{ m}, H = 0.6 \text{ m}, k_0 = 10^{-9} \text{ m/s} \) and \( m_w = 2 \times 10^{-6} \text{ m}^2/\text{N} \). 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.

![Comparison between experimental and surface settlements of unit cell.](image)

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 \( \lambda \). 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 \( \alpha \) and \( e_0 \) produce retarding effect on the excess pore water pressure dissipation.

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