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Numerical modelling capturing the behaviour of reinforced soft ground for public transport infrastructure

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ABSTRACT

Foundations on soft soil without proper ground improvement can initiate excessive settlement causing undrained failure of infrastructure. Amongst various ground improvement techniques, reinforcement by stone columns is one of the convenient and effective methods, with numerous advantages including: increased bearing capacity, reduced settlement, improved slope stability and liquefaction control. The stone columns not only act as vertical stiffening members increasing the overall bearing capacity of soft ground while reducing the overall settlement, they also assist in effective radial consolidation. The existing analytical and numerical solutions to predict the behaviour of reinforced soft ground, the models are based on equal strain hypothesis aided with linear void ratio-effective stress correlation, and therefore incapable of capturing the special considerations relevant to transport infrastructure. The authors have developed a novel numerical model based on Finite Difference Technique to analyse the response of stone column reinforced soft soil supporting public transport infrastructure, adopting a free strain hypothesis and also considering arching, clogging and smear effects aided with effective stress dependant soft soil compressibility. The model has incorporated the stiffening effect of the columns as well as accelerating the consolidation by radial drainage. Apart from predicting excess pore water pressure dissipation and resulting settlement, the load transfer mechanism and degree of improvement were captured by the model. The predictions of the model were compared to the results obtained from a full-scale trial embankment construction at the Australian National Field Testing Site at Ballina, NSW, Australia.

Keywords: clogging radial consolidation, arching, soft clay, pore water pressure, stone columns

1 INTRODUCTION

In many countries, reducing long-term settlement of infrastructure and providing cost-effective foundations with sufficient load-bearing capacities are national priorities (Basack et al. 2011). Foundations constructed on soft, compressible soil can cause excessive settlement initiating undrained failure of infrastructure if proper ground improvement is not carried out (Indraratna 2009). Over several decades, different ground improvement techniques have been developed, which include stone columns, preloading with vertical drains with or without vacuum preloading, piling, geogrids and chemical stabilization. Reinforcing the ground by installing stone columns is one of the well-established and effective techniques practised worldwide (Wang 2009). The concept involves partial replacement of the soft soil with compacted vertical columns of stone aggregates which act as in-situ reinforcement to the soft ground. The presence of stone columns transform the soft ground into a composite mass of granular cylinders, having reduced compressibility with increased shear strength in comparison to the natural soft soil.

Numerous analytical and numerical studies have been carried out to study the behaviour of stone column reinforced soft ground (Han and Ye 2001; 2002, Wang 2009, Castro and Sagaseta 2009; 2011). Most of these models are based on the ‘equal strain’ hypothesis true for rigid surcharge load. In the case of embankment loading, the flexible nature of the applied surcharge load induces uneven surface settlement or ‘free strain’ (Barron 1948). Clogging is initiated by migration of clay particles into the pores of the stone column, significantly reducing the hydraulic conductivity. Unclogged columns will continue to operate under steady state flow with a relatively constant hydraulic conductivity. In this paper, the free strain hypothesis has been chosen with adequate consideration for arching, clogging, smear and time-dependant radial consolidation. The solutions were developed by means of unit cell analogy and finite difference coding with central, forward and backward difference techniques.

2 MATHEMATICAL MODELLING

2.1 Problem Identification

The current model is an extension of the simple free strain model by Indraratna et al. (2013). In the current model, the application of the Modified Cam-clay model has not only captured the nonlinear variation of void ratio-effective stress relationship, but also facilitates cyclic loading relevant to transport infrastructure**.** As observed from the Figures 1(a) and (b), the soft clay (thickness = H) overlays a stiff clay layer (equivalent to an impervious rigid boundary), and is improved by a group of stone columns. When the soft soil overlays a stiff clay, dense sand or rock stratum, this may be idealized as a rigid and impervious medium. The model deviates from accuracy, if the soft clay layer is underlain by a relatively loose sandy deposit. The radii of the column and the unit cell are r_c and r_e respectively. The cross section of the entire zone of the unit cell is divided into four distinct zones (see Figure1c): unclogged column zone, clogged column zone, smear zone and the outer undisturbed soil zone. A steady and uniform load intensity q_s is imposed on the ground surface. Considering the self-weight of the embankment, the average load intensity on the ground surface may have been $\overline{q} = q_s + \gamma_e H_e$, where, γ_e and H_e are the unit weight and height of the embankment, respectively.

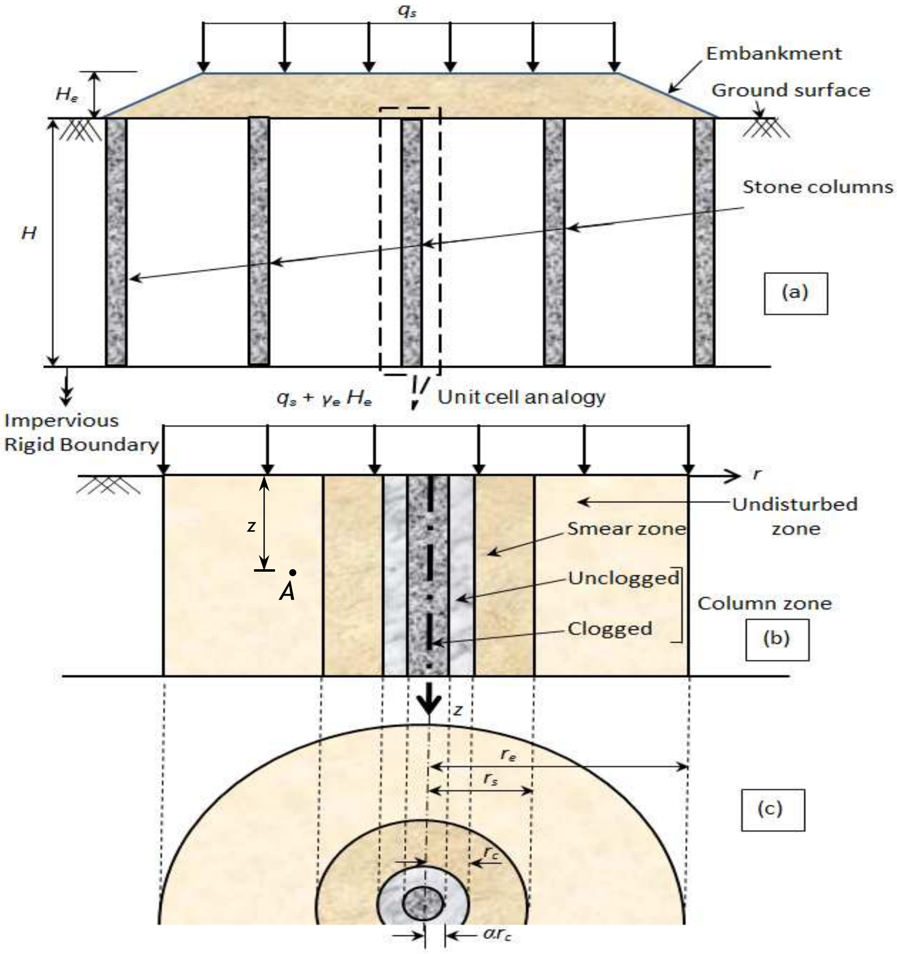

Figure 1. Soft clay reinforcement: (a) Embankment resting on stone column installed ground. (b) Unit cell analogy. (c) Cross section of unit cell.

For finite difference coding, the soil mass within the unit cell was divided both radially as well as vertically into a large number of equal divisions, n_r and n_z respectively (see Figure 2). The total computational time t_t as well was discretized divided into $(n_t - 1)$ equal divisions with each division being $\delta_t = t_t / (n_t - 1)$. The separators are hereby indicated as *nodes* and the soil elements are understandably ring-shaped. The specific time 't_t' is the desired computational time to be chosen arbitrarily. The numerical analyses are based on the assumptions that the deformations of the column and the soil are vertically downward and pore water flow is purely horizontal and radially inward towards the column. Also, the stone column was assumed to be a freely draining material.

2.1 Column-to-Soil Load Transfer

Whenever an embankment is supported on a soft soil deposit reinforced with a vertical stiffening material like stone columns, arching is obvious because of significant column to soil stiffness ratio. This initiates greater stress transfer from the soil. Following the limit state analysis under passive condition of embankment material (Indraratna et al. 2013), the load distribution function on the soft ground was derived as:

$$q(r) = q_2 + \left(N - r / r_c\right)^2 f(N, n_s) \qquad \text{......(1)}$$

where, q_2 is the stress on soil at the unit cell boundary, the function f depends on embankment characteristics and N and n_s, the term n_s being the stress concentration ratio between the soil and column (i.e., $n_s = q_c / q_1$), N is the geometrical constant r_e/r_c, q_c and q_1 are the stresses on column and soil respectively. The typical range of the term n_s is between 2 – 10 (Han and Ye 2000; 2002; Indraratna et al. 2013). The proposed model is based on a 'free strain' hypothesis which initiates uniform load intensity on the unit cell surface with uneven vertical strains. The arching effect produces a parabolic stress distribution on the unit cell surface, as shown in the Figure 3 based on Eq.(1).

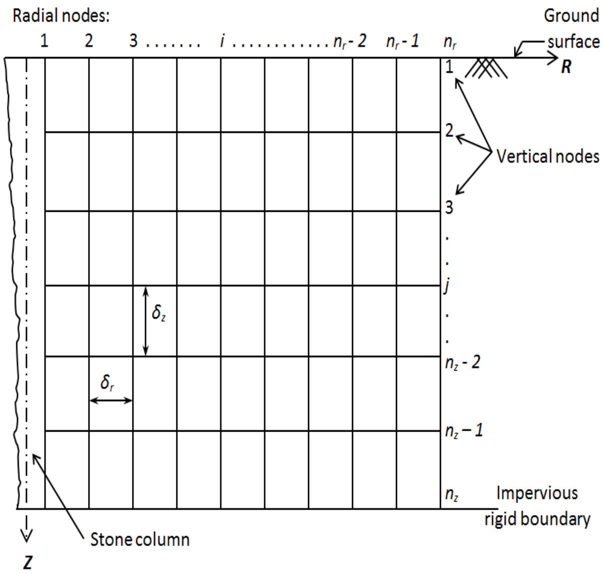

Figure 2. Finite Difference discretization

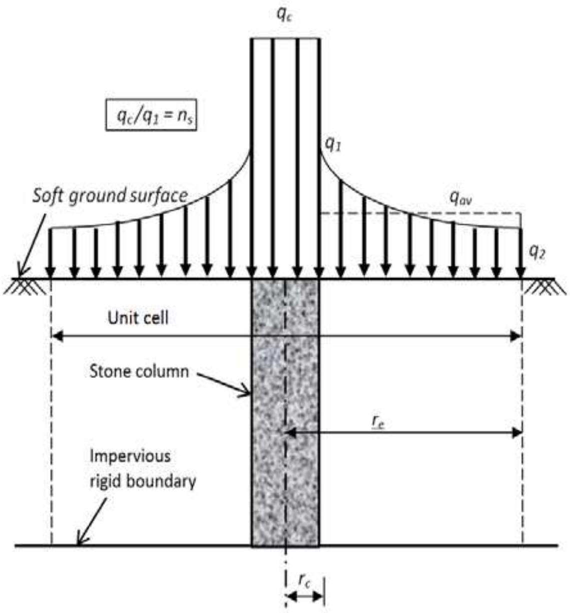

Figure 3. Vertical stress distribution on unit cell surface

2.2 Radial Consolidation

The following differential equation pertaining to radial consolidation theory (Barron 1948) was adopted:

$$\frac{\partial u}{\partial t} = c_h \left(\frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) \qquad \text{...(2)}$$

where, u is the nodal excess pore water pressure, c_h is the coefficient of radial consolidation and t is the time. Applying the finite difference technique with central, forward and backward difference methods and with appropriate boundary conditions, the following matrix equation was obtained:

$$[A]\{u\} = \{b\} \qquad \text{...(3)}$$

where, *[A]* is the relevant coefficient matrix, *{u}* is the column vector relevant to the unknown nodal excess pore pressures and *{b}* is the augment vector. Solution to the above equation yielded the unknown pore pressures at the nodal points, and hence their average value and degree of consolidation. For stone column reinforced soft ground, since the imposed vertical stress is mainly carried by the column due to its higher stiffness compared to the surrounding soft soil, the influence of the void ratio dependant soil permeability is not significant (Han and Ye 2001; 2002; Wang 2009;

Indraratna et al. 2013). Hence, the change in permeability during the consolidation of soft clay is not considered here. Also, due to much smaller horizontal seepage path, hence the vertical degree of consolidation is negligible compared to radial degree of consolidation. The displacement of a point *(r, z)* in the soil mass of the unit cell at time *t* was evaluated by:

$$\rho_{rzt} = -\int_0^t \int_z^H m_v \frac{\partial u(r,z)}{\partial t} dz dt \qquad \ldots(4)$$

where, m_v is the coefficient of volume compressibility. With the analysis of Khan et al. (2010), the effective stress developed in the soil mass at any point *(r, z, t)* in the space-time coordinate have been expressed as:

$$\sigma'(r,z,t) = \gamma' z + q(r) - u(r,t) \qquad \ldots(5)$$

where, γ' is the effective unit weight of soil. The clogging decreases the hydraulic conductivity of column. The 'clogged' and 'unclogged' parameters referred to in the model are α and α_k which are the ratio of the diameters of unclogged column zone to the overall column diameter, and the ratio of horizontal soil permeability of the clogged column zone to the smear zone of the soft soil, respectively.

3 VALIDATION

Oh et al. (2007) carried out field tests in soft estuarine clays in Queensland, Australia with a trial embankment incorporating stone columns and a reference section without any stone columns. The area replacement ratio for 2m and 3m spacing are respectively 0.196 and 0.125 respectively, and the average embankment height has been 4m. A comparison of computed ground settlements using the present solution with the field results is presented in Figure 4. The computed values of ground settlements are in reasonable agreement with the field data. With clogging effect being incorporated (α = 0.5, α_k = 1.0), the predicted settlements are closer to field measurements. It is true that the magnitude of final settlement is not affected significantly by column inclusion, but it is the rate of consolidation settlement which is important as observed in Figure 4.

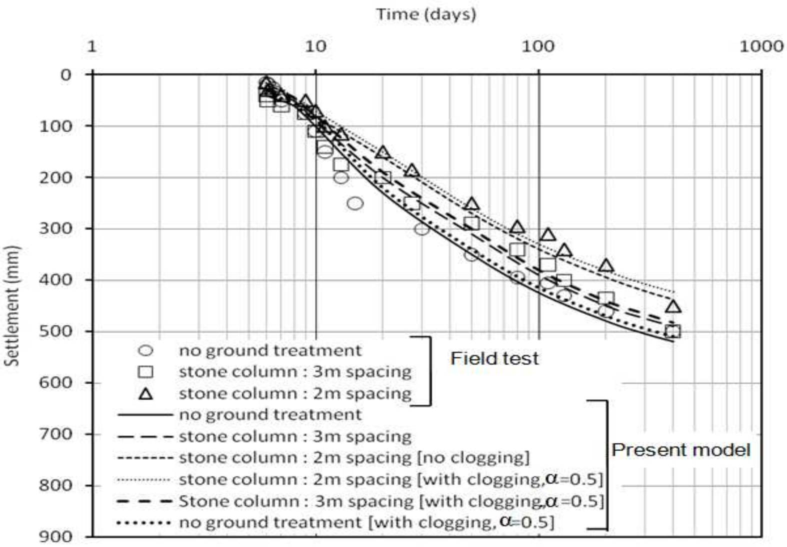

Figure 4. Comparison with field data (modified after Indraratna et al. 2013)

4 PARAMETRIC STUDIES

With the initiative of the Australian Research Council (ARC) funded Centre for Excellence in Geotechnical Science and Engineering (CGSE), Australia's first national geotechnical field testing facility has been established at Ballina, New South Wales. Under the supervision of the Geotechnical Research Group at University of Wollongong, a group of test stone columns were installed at Ballina site (see Figure 5). Parametric studies were carried out using the field data presented in Table 1.

Table 1: Input parameters for numerical analysis

Soil				Embankment				Stone Column
k_h (m/s)	m_v (m^2/N)	K_0	H (m)	H_e (m)	γ_e (kN/m^3)	q_s	K_p	r_c (m)
1 x10^{-9}	3 x10^{-6}	0.8*	10	4	20	0	3	0. 5
* After Hayashi et al. (2012).								

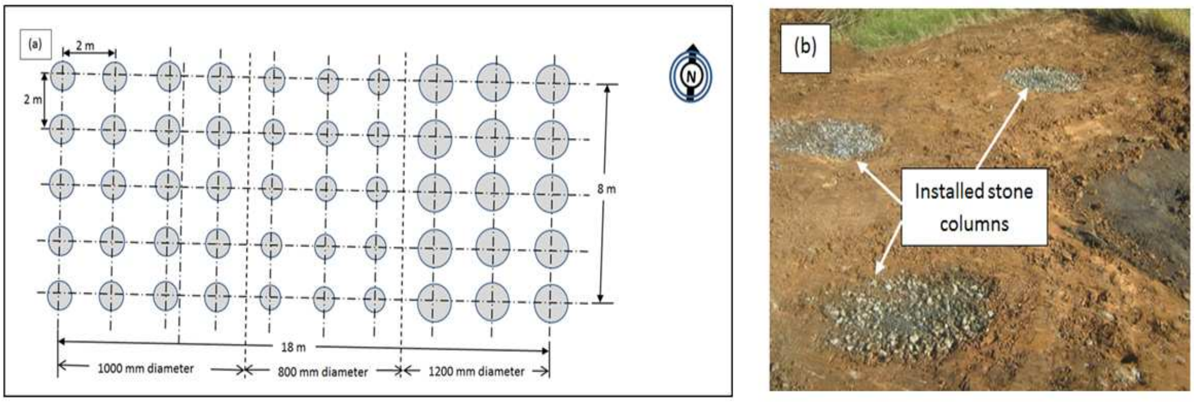

Figure 5. The installed test columns: (a) Plan. (b) Photographic view.

The stress distribution patterns in the column and the soil were studied with respect to the stress concentration ratio n_s. As observed from Figures 6, the parameter n_s increases in a parabolic manner with an ascending slope with the vertical stress on column, while it decreases following a hyperbolic pattern with a descending slope with an average vertical stress on the soft ground surface. In the range of $2 < n_s < 14$, the normalized stresses on column and soil varied between 2–8 and 0.05 – 0.55, respectively.

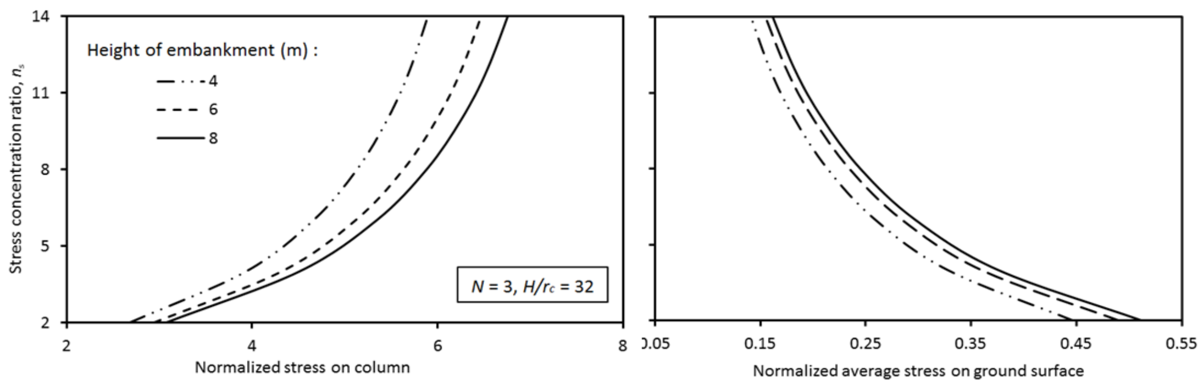

Figure 6. Vertical stresses on column and soil (modified after Indraratna et al. 2013)

The time variation of normalized average excess pore water pressure in the soil and the average degree of consolidation are shown in Figures 7(a) and 7(b) respectively. Understandably, from Eq. (2), the dissipation pattern is exponential. Also, considerable resistance to the dissipation due to clogging has been observed. The degree of consolidation decreases with increasing values of α and α_k, reasonably justifying the effect of clogging. As observed from Figure 8, the normalized average ground settlement exponentially increases with time. The clogging influences the settlement pattern at initial consolidation stage ($0 < T_r < 0.6$). Due to the steady vertical stress distribution on unit cell surface, the ultimate settlement unaltered although the effect of clogging only retards the rate of consolidation. The depth-wise variation of normalized effective vertical stress in soil for different radial distances has been shown in Figure 9. With increase in depth below ground surface, the effective vertical stress was observed to increase following a hyperbolic pattern with a descending slope. At a particular depth, the effective stress was found to decrease with the ascending radial distance.

5 APPLICATION OF THE PROPOSED ANALYSIS WITH MODIFIED CAM-CLAY MODEL

The Eq. (2) above is based on the linear pattern of variation of the void ratio with effective overburden stress in the soil, which is a simplified assumption. Amongst several recent constitutive models (e.g.: Fahey and Carter 1993; Basack and Sen 2014) to capture the nonlinear behaviour of soft clay, the Cam-clay models have been most effective and convenient (Carter et al. 1982; Ni et al., 2014). In this section, the Authors have attempted to apply the modified Cam-clay soil model (Roscoe and Burland 1968) to predict the behaviour of stone column reinforced soft ground.

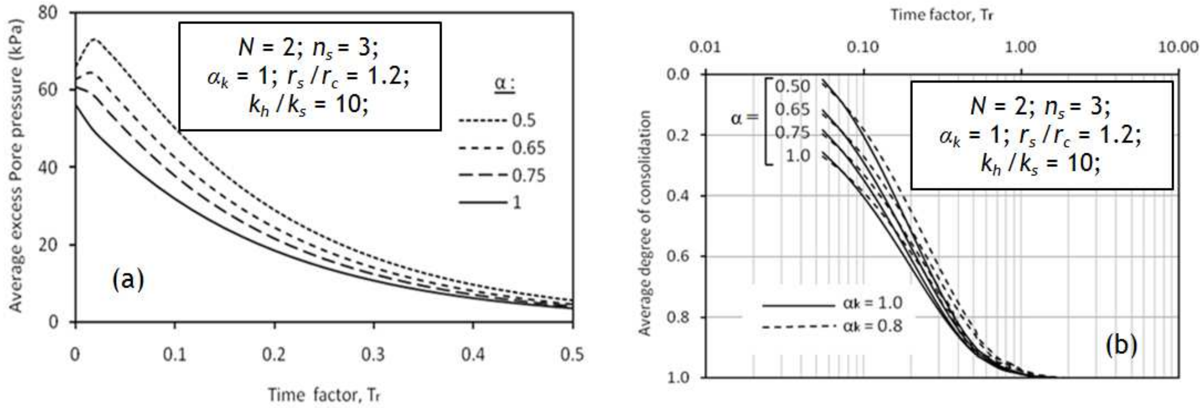

Figure 7. Time variation of: (a) excess pore water pressure. (b) degree of consolidation.

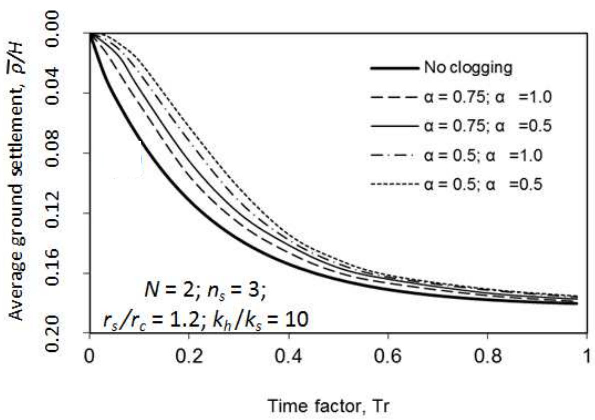

Figure 8. Variation of average ground settlement with time

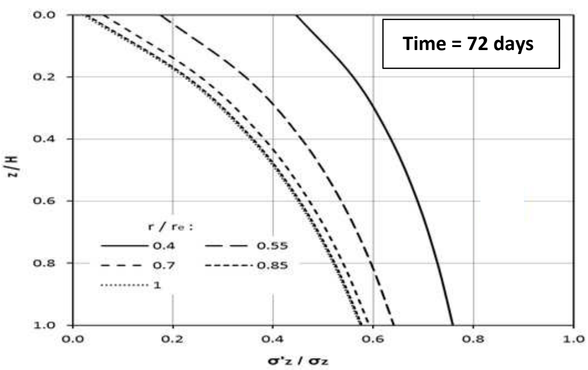

Figure 9. Variation of vertical effective stress in soil with depth

Assuming the soil within the unit cell under K_0–condition, the mean effective stress at any point *A* (see Figure 1b) in the soil is given by:

$$p' = \left[q(r) + \gamma' z - u_{rt} \right] \frac{1 + 2K_0}{3} \qquad \text{…(6)}$$

In accordance with the modified Cam-clay model (MCC), the void ratio *e* of normally consolidated clay decreases in a logarithmic manner with an increasing mean effective stress *p'* in the soil (see Figure 10) which essentially implies the following:

$$m_v = -\frac{(1 + 2K_0)\lambda p'_0}{3p' \left[1 + e_0 - \lambda \ln\left(p'/p'_0 \right) \right]} \qquad \text{…(7)}$$

where, e_0 is the void ratio of soil corresponding to the unit pressure p'_0. However, the dependency of *p'* on *z* implies a depth-wise variation of the parameter m_v initiating a vertical component of pore water flow, which deviates from the initial assumption for radial consolidation only. To remove this redundancy and to simplify the already complex equations, the average value of the mean effective stress in the soil has been taken in the model. From Eq. (6), the expression for m_v has thus been deduced as:

$$m_v = -\frac{\lambda p'_0}{\left[q(r) + \gamma' \frac{1}{H} \int_0^z p' dz - u_{rt} \right] \left[1 + e_0 - \lambda \ln\left(p'/p'_0 \right) \right]} \qquad \text{…(8)}$$

The incorporation of MCC model is carried out by iterative trial-and-error technique. Starting with an initial value of $m_{v,}$ computations have been conducted following the methodology described in section 2 above. The values of nodal excess pore water pressures obtained were then utilized to calculate the modified value of m_v given by Eq. (8). The procedure is then recycled till the desired convergence is achieved. The entire computation has been performed by means of a user friendly program COLMCC written in Fortran 90 language, the flowchart of which is given in Figure 11.

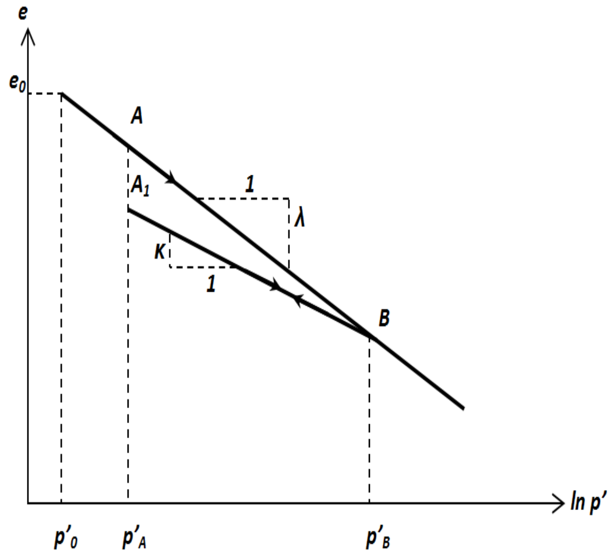

Figure 10. e-ln p' correlation as per MCC model

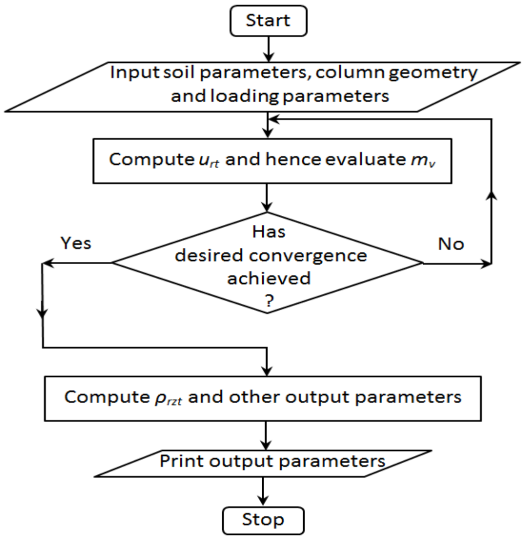

Figure 11. Flowchart of the program COLMCC

Using the modified model, further analyses have been performed using different values of e_0 and λ. The variation of the average values of excess pore water pressure and ground settlement with time have been depicted in Figures 12 (a) and (b) respectively. As observed, both the excess pore water pressure and the ground settlement vary exponentially with time for both the linear and the MCC models. The radial consolidation is promoted leading to accelerated settlement, under essentially drained condition. To simplify the already complex mathematical equations, it was assumed that the applied load would remain steady, which was considered equivalent to step loading (Wang, 2009), thus the strain rate effect is eliminated. Compared to the excess pore water pressure, the values of ground settlements were found to be more influenced by the variation of e_0 and λ. As observed from Figure 12(b), the values of average ground settlement relevant to the MCC model are reduced up to about 30% compared to that for the linear model. The ground settlement, having a direct correlation with the parameter m_v, is highly sensitive to the MCC model parameters (see Equations 4 and 8). It appears from these observations that reasonable accuracy of analysis based on the MCC model depends upon the appropriate choice of these parameters. Another reason for such deviation is the simplified assumption of a pure radial consolidation, whereas a more rigorous analysis (e.g.: Han and Ye 2001) demands adequate consideration of the vertical component of pore water flow, which is increasingly important for shorter (partially penetrated) columns.

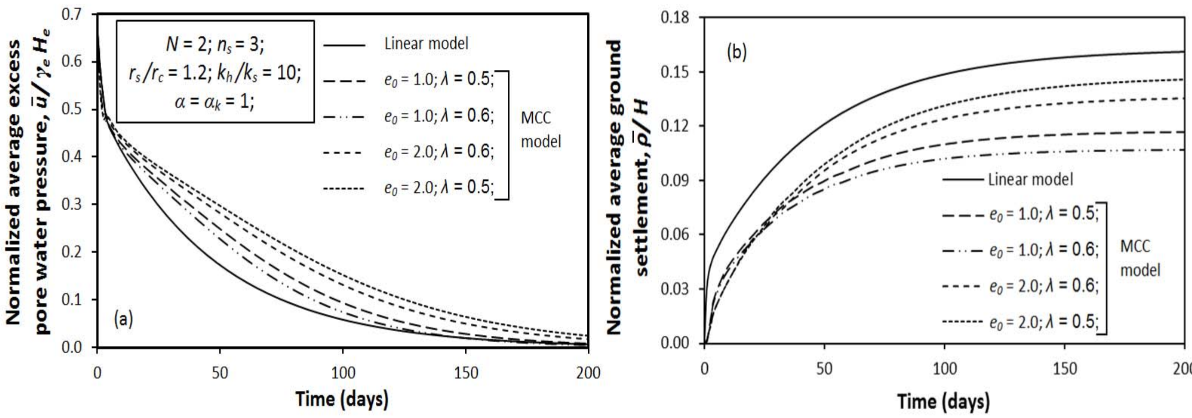

Figure 12. Comparison of linear and MCC models for time-variation of: (a) average excess pore water pressure. (b) average ground settlement.

6 CONCLUSION

The Authors have developed a numerical model based on a unit cell analysis to predict the time-dependant behaviour of stone column reinforced soft ground, with particular reference to transport infrastructure. Initially, a linear model has been developed assuming a linear relationship between void ratio and effective overburden pressure in soft clay. The effects of arching, clogging and smear were considered in the model. Validation of the model with available field test data indicates reasonable accuracy. Parametric studies have been carried out as relevant to the in-situ soil properties of the Ballina site. The study indicates that the stress concentration factor n_s increases in a parabolic manner with an ascending slope with the vertical stress on column, while it decreases following a hyperbolic pattern with a descending slope with an average vertical stress on the soft ground surface. Both the excess pore water pressure (hence the degree of consolidation) and ground settlement vary exponentially with time. Introduction of clogging parameters have retarded the consolidation settlement. The effective vertical stress in soil increases with depth in a hyperbolic pattern with a descending slope, and it decrease with the ascending radial distance.

The linear model has been upgraded by the application of the modified Cam-clay model. The MCC parameters e_0 and λ have pronounced influence on the consolidation characteristics, with an average deviation up to as high as 30%.

7 ACKNOWLEDGEMENT

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