Design Recommendation for Stone Column Reinforced Soft Clay Deposit

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ABSTRACT
Reducing long-term settlement of infrastructure and providing cost-effective foundations with sufficient load-bearing capacities are national priorities for infrastructure development in most countries. Among various methods of soft soil improvement, installing stone columns is one of the well established and effective techniques practised worldwide. The stone columns not only act as reinforcement to the surrounding soil, but also speed up the time-dependant dissipation of excess pore water pressure due to surcharge loading by shortening the drainage path. A novel numerical model has been developed and validated by the authors to analyse the response of stone column reinforced soft soil under embankment loading, adopting the free strain approach and considering arching, clogging and smear effects. Using the model, a design methodology associated with a series of charts and curves for various clogging and smear zone parameters has been suggested by the authors. Utilizing them, a typical design example for stone column reinforcement in a soft clay deposit has been presented.

RÉSUMÉ
Dans la majorité des pays, le développement des infrastructures constitue une priorité nationale pour construire sur des foundations à coût raisonnable, offrant une portance suffisante et une réduction de tassement à long terme. L'exécution des colonnes ballastées représente l'une des techniques d'amélioration efficace des sols mous largement pratiquée à l'échelle internationale. En plus du renforcement du sol environnant, les colonnes ballastées accélèrent la dissipation des surpressions interstitielles, générées par le chargement, par la réduction de la distance de drainage. Une nouvelle methode numérique, mise au point et validée par les auteurs, a été déveoppée pour l'analyse du comportement d'un sol mou renforcé par colonnes ballastées sous l'action d'un remblai. Cette méthode se base sur une approche à deformation libre et tient compte des effets de voûte, de l'obturation et du remaniement. Avec ce modèle, les auteurs proposent une méthode de dimensionnement illustrée par des abaques et de courbes avec variation des paramètres de l'obturation et de remaniement. Un exemple type de renforcement par colonnes ballastées d'une argile molle est étudiée à l'aide de ces outils.

1 INTRODUCTION

It is imperative to apply adequate ground improvement technique to the existing soft soils before any construction in order to prevent unacceptable excessive and differential settlement and increase the bearing capacity of the foundations (Indraratna et al. 1992).

Amongst various methods of soft soil improvement, reinforcing the ground by installation of stone column is one of the well established and effective techniques followed worldwide (Wang, 2009). As reported by Guetif et al. (2007), there are basically three components which contribute to the degree of improvement achieved by installation of stone columns in soft clay deposits, which are: (i) The stone column, possessing greater strength and stiffness in comparison to the surrounding soft soil, acts as a reinforcement increasing the overall load bearing capacity of the ground and decreasing the settlement, (ii) During installation of stone column, densification of soil in the vicinity of the interface takes place, and (iii) The stone column speeds up the time dependant dissipation of excess pore water pressure due to surcharge loading by shortening the drainage, thereby accelerating the improvement the strength and stiffness of the surrounding soil within the zone of influence.

Various analytical and numerical solutions have already been developed by many researchers for understanding the load transfer mechanism of soft soil reinforced with stone column, e.g., Han et al.(2000 & 2002), Ambily et al. (2007), Malarvizhi et al. (2008), Wang (2009), Lo et al. (2010) and Murgasen et al. (2010). All these solutions are based upon unit cell analysis assuming 'equal strain' hypothesis. However, such an assumption is applicable whenever the surcharge load applied on the ground surface possesses adequate flexural rigidity initiating uniform ground settlement, thereby resulting in an unequal distribution of stress induced on the soil surface. In case of embankment, the flexible nature of applied surcharge loading is most likely to induce an equal distribution of surface load resulting in an uneven ground settlement, i.e., the 'free strain' (Barron, 1948).

In case of a fill embankment, the behaviour of the soil-stone column system is time-dependent. Initially, most of the imposed total stress is borne by the pore water. Due to dissipation of the resulting excess pore pressure, progressive settlement of the soft clay occurs and arching takes place resulting in an uneven distribution of vertical stress on the ground surface. This phenomenon is duly supported by other researchers (Low et al., 1994, Abusharar et al., 2009 and Deb, 2010).

As reported by Han et al. (2002), a smear zone is developed in the soil adjacent to the soil-column interface due to installation. Also, because of the migration of clay particles from soil into the pores of the column, a clogged zone may be formed within the column in the vicinity of the soil-column interface (Adalier et al., 2004).

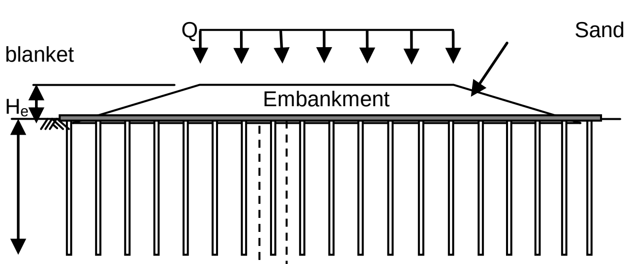
Based on 'free strain' hypothesis and considering the arching, smear and the clogging effects, a numerical model (finite difference method) have already been developed and validated by the authors (Indraratna et al., 2010). Using the model, a design methodology associated with appropriate design curves has been proposed and a typical design example has been illustrated in this paper.

2 NUMERICAL ANALYSIS

The details of the numerical model developed have been described elsewhere (Indraratna et al., 2010). A brief illustration of the analysis carried out is given herein.

The analysis was done based on the assumption that the compressive strains of the soil column occur only in the vertical direction, and ignoring the elastic settlements which are insignificant compared to the consolidation settlement. The soil was assumed to be fully saturated with incompressible water, the flow of water through the soil to be purely horizontal (radial towards the column) following Darcy's law and no flow of water to take place through the cylindrical boundary and the impervious base of the unit cell. The coefficients of permeability (k_h) and compressibility (m_v) of the soil was assumed to remain constant during the process of consolidation.

The idealised problem is depicted in Figures 1(a) and (b). The soft clay layer of thickness H has been assumed to overlay on an impervious rigid boundary, and is improved by a group of stone columns having a radius r_c each, extended to the bottom of the clay layer. The average load intensity on the ground surface ($\overline{q}$), was expressed as a sum of the uniform surcharge load intensity Q on the embankment fill and the self weight of the embankment ($\gamma_e H_e$). The radius of influence of the unit cell r_e could be calculated following the method of Wang (2009). The cross section of the entire zone of the unit cell was divided into four distinct zones (Fig.1c), viz., the unclogged column zone, clogged column zone, smear zone adjacent to the column and the outer undisturbed soil zone. As shown in Fig.2, the soil mass within the unit cell had been divided both radially as well as vertically into *(n-1)* number of equal divisions; *n* being a positive integer greater than unity, such that each of such divisions may be expressed respectively as: $\delta_r = (r_e - r_c)/(n-1)$ and $\delta_z = H/(n-1)$. The total time interval of computation t_t is divided into *(n-1)* number of equal divisions, i.e., $\delta_t = t_t/(n-1)$. In this paper, these separators are denoted as 'nodes'. The soil elements are understandably ring-shaped. The primary objective of the analysis was to compute the excess pore water pressures and the effective stresses developed at each separator at the corresponding time, and thereby compute the other time-dependant variables such as the degree of consolidation and settlement.



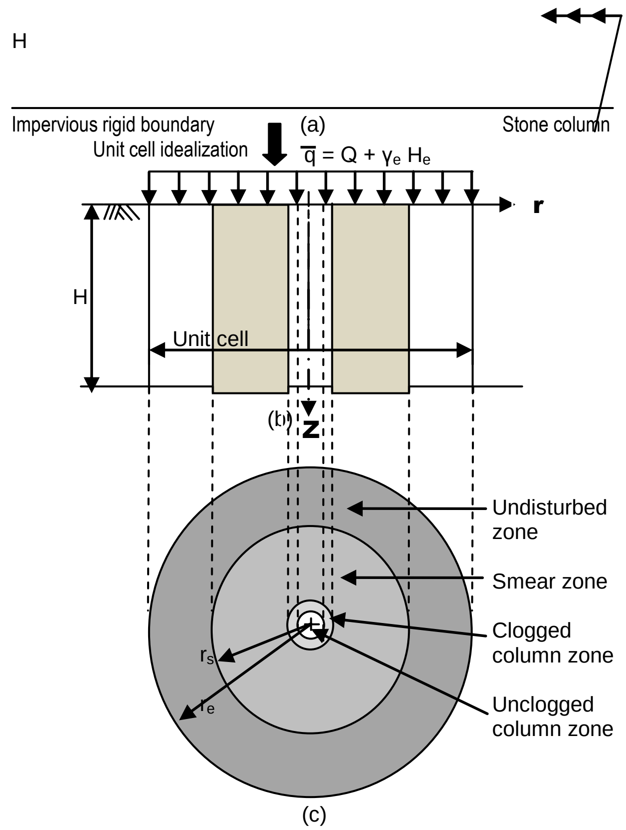


Fig.1. (a) A typical stone column reinforced soft clay deposit supporting an embankment. (b) Unit cell idealization. (c) Cross section of the unit cell.

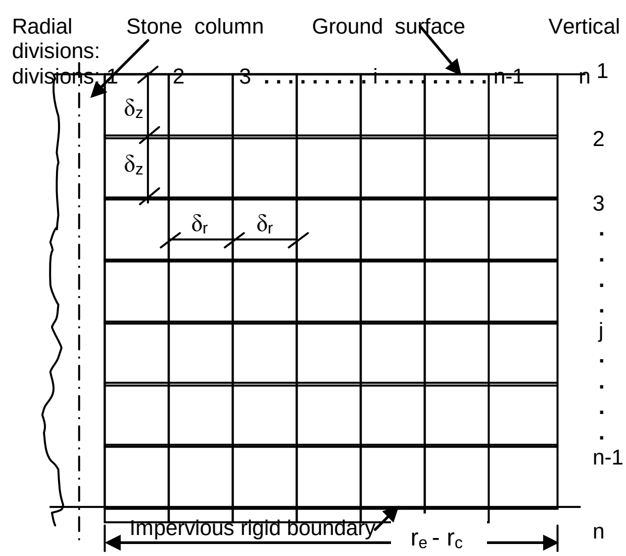


Figure 2. Soil element discritization within the unit cell.

The smear zone parameters, viz. r_s and k_s may be reasonably estimated following the recommendations of Han et al. (2002), Walker et al. (2006) and Wang (2009), where, r_s and k_s are the radius and the horizontal permeability of soil in the smear zone respectively. The clogging effect was quantified by the non-dimensional parameters α and α_k. Because of the different column to soil stiffness ratio, the soil arching beneath the embankment occurs. Following the analysis of Abusharar et al. (2009), the arching effect was analysed to compute the vertical stress distribution *q(r)* on the ground surface which was found to be parabolic and a unique function of N ($=r_e/r_c$), n_s, r_c, r, q and K_p (passive earth pressure coefficient of the embankment). The term n_s, referred as the stress concentration ratio, was defined as the ratio of stiffness between soft soil and stone columns and a function of height of embankment, properties of the soils in the embankment and that of the soft clay deposit. The values of n_s can be reasonably estimated following the recommendations Castro et al. (2009).

The nodal excess pore pressures were computed based on the radial consolidation theory of Barron (1948). Using appropriate boundary conditions, the following matrix equation was evaluated:

$$[A]\{u\} = \{b\} \qquad \text{......(1)}$$

where, $[A]$ = coefficient matrix; $\{u\}$ = unknown vector for excess pore water pressure at nodes; $\{b\}$ = augment vector.

Solving the Equation (1), the nodal excess pore pressures, and hence the degree of consolidation, were computed. The nodal displacements within the soil mass of the unit cell at time t was given as:

$$\rho = -m_v \int_0^t \int_0^z \frac{\partial u(r,z)}{\partial t} dz\, dt \qquad \text{......(2)}$$

The nodal and average ground settlements were computed by carrying out numerical integration.

The effective stress developed in the soil mass at any point *(r, z, t)* in the space-time coordinate may be expressed by (Khan et al., 2010):

$$\sigma'(r,z,t) = \gamma' z + q(r) - u(r,t) \qquad \text{......(3)}$$

where, γ' is the effective unit weight of the soil mass.

During consolidation, the undrained strength and stiffness of the soil increase progressively. Guetif et al. (2007) carried out extensive finite element analysis to investigate the improved soft clay characteristics due to stone column installation. The undrained cohesion at any point in the space-time coordinate system was computed following the analysis of Umezaki et al. (1993). The increased soil strength was quantified by a non-dimensional improvement factor β which was defined as the minimum value of the ratio of the post-consolidation to initial undrained cohesion at ground surface. Similarly, the increase in stiffness of the soft soil was expressed as a settlement factor ξ defined as the ratio of the average ground settlements of the reinforced to unreinforced soils at 90% consolidation.

For design purpose, a modified time factor T'_{90} has been introduced herein, which is expressed as:

$$T'_{90} = c_{vr}\, t_{90} / H^2 \qquad \text{......(4)}$$

where, t_{90} is the time required for the average degree of consolidation to attain 90% and c_{vr} = coefficient of radial consolidation of the soil = $k_h / (m_v \gamma_w)$, γ_w being the unit weight of water. Since both r_c *and* r_e are variables, the constant parameter H is used for normalizing the time.

The computation was carried out by means of a user-friendly computer software written in FORTRAN 90 language. The relevant flowchart is shown in Fig.3.

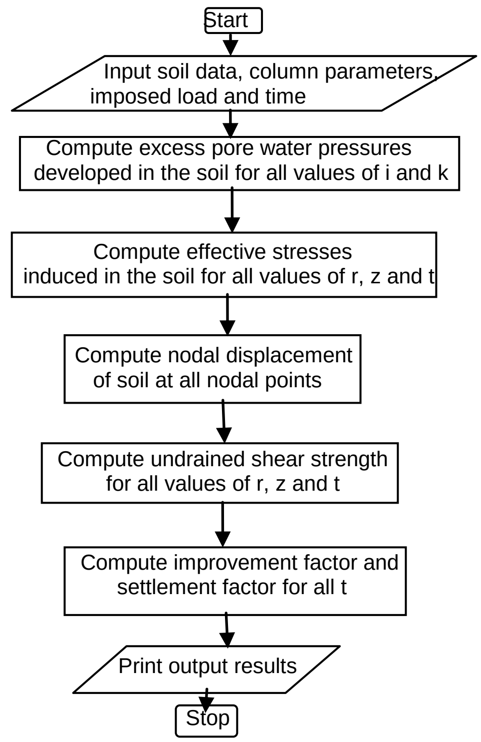


Figure 3. Flowchart for the computer program.

3 VALIDATION

Comparison of the average degree of consolidation by radial drainage only has been made with the existing solutions of Barron (1948), Hansbo (1981), Han et al. (2000 & 2002) and Wang (2009). The variations of average degree of consolidation with time factor are presented in Fig.4. It was observed that the results obtained using the present model are in acceptable agreement with the other solutions and close to those of Han et al. (2002).

Redana (1999) analysed the performance of two test embankments T1 and T2 constructed at a naval dockyard of Pom Prachul, Thailand. For improvement of soft ground, sandwitch drains were installed in a square

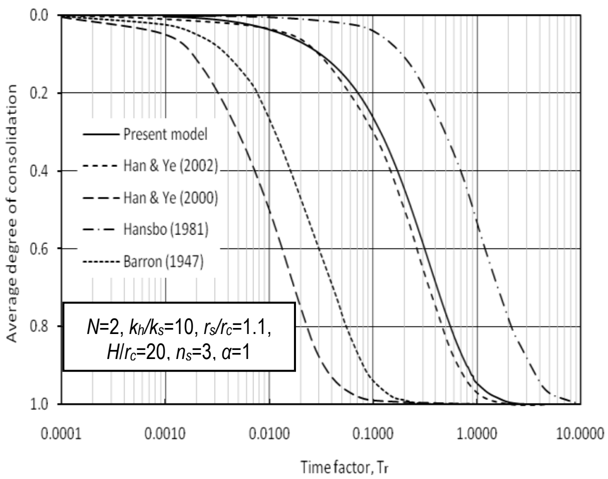
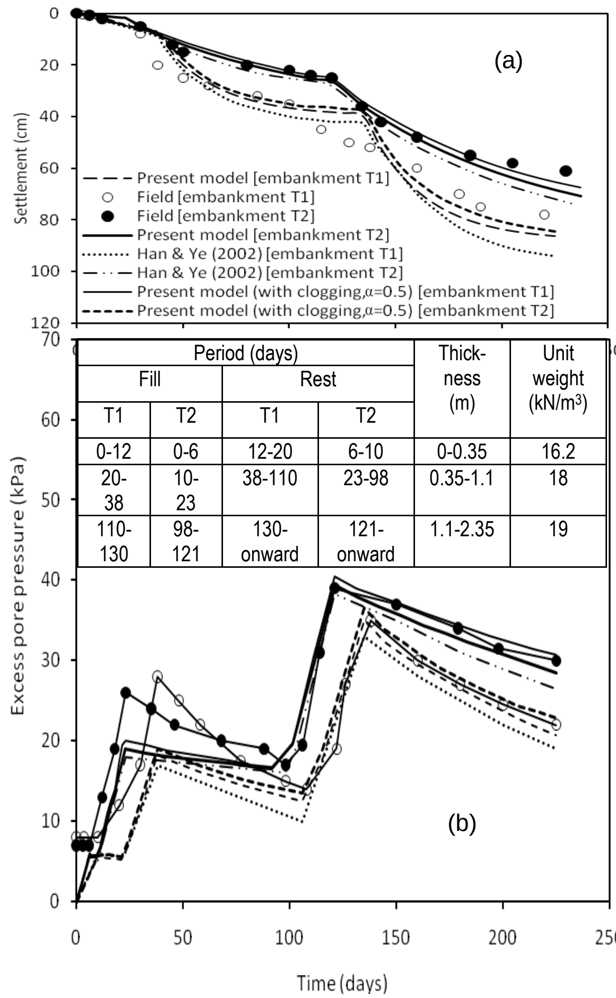


Figure 4. Comparison of computed rates of consolidation using different methods.

grid pattern and filled with compacted dry sand. The field test results were compared with those obtained by using the model and that of Han et al. (2002). The computational parameters adopted are same as thosefound by Redana (1999) and presented in Table1. The variations of average ground settlement and excess pore water pressure with time are shown in Figures 5(a) & (b) respectively. As observed, although the time-settlement variation obtained by the present model is in reasonably good agreement with the field data, the solutions of Han et al. (2002) slightly over-predicts the values. The assumption of free strain and the parabolic distribution of vertical stress on ground surface in the present analysis have significantly affected the results. As regards to the time variation of excess pore pressure, the present solution yields promising results as compared to the field data for t > 100 days, where as the solution of Han et al. (2002) slightly under-predicts the values. For t < 100 days, both the solutions were observed to give relatively lower values compared to the field test results. Solutions are also obtained considering clogging effect (α = 0.5, α_k = 1) which predicts the settlement and the excess pore water pressure even closer to the field values.

4 DESIGN RECOMMENDATION

Indraratna (2009) reported ground improvement at the Ballina Bypass for construction Pacific Highway linking between Sydney and Brisbane. This site had a floodplain consisting of highly compressible and saturated marine clay deposits. A soft silty layer of clay approximately 10 m thick was underlain by moderately stiff, silty layer clay located 10-30m deep, which was in turn underlain by firm clay. The groundwater level was almost at the ground surface. The relevant field data are utilized herein to carry out a hypothetical case study followed by a design illustration. The parameters used for computation in this case study are given in Table 1. Using these values, a set of typical design curves are developed. The variation



Period (days)				Thick-ness (m)	Unit weight (kN/m³)
Fill		Rest			
T1	T2	T1	T2		
0-12	0-6	12-20	6-10	0-0.35	16.2
20-38	10-23	38-110	23-98	0.35-1.1	18
110-130	98-121	130-onward	121-onward	1.1-2.35	19

Figure 5. Comparison of numerical results with field test data of Redana (1999) for: (a) settlement versus time. (b) excess pore pressure versus time.

of T'_{90} with N for different values of H/r_c and n_s are presented in Fig.6, assuming reasonable smear and clogging parameters. Similarly, the variation of the stiffness factor ξ with N for different values of H/r_c and n_s are presented in Fig.7. Lastly, the variation of the improvement factor β with normalized imposed stress q/c_{u0} were plotted (Fig.8) for different values of N and n_s , c_{u0} being the initial undrained cohesion at ground surface. It has been observed that the variation of β with H/r_c is insignificant, and therefore not considered.

For other values of smear and clogging parameters, similar design curves can be prepared using the model and the relevant computer software developed.

4.1 Design Example

In this section, how the solutions developed and the above-mentioned design curves are used in actual design are described. The following are the design requirements:

(i) The consolidation is expected to be completed 90 % by 1 year after the completion of the embankment construction.

(ii) The height of embankment above the virgin ground level should be 4.3 m.
(iii) The desired improvement factor should not be below 2.75.
(iv) From the serviceability criteria, the average settlement should not exceed 500 mm at 90% consolidation.

Although the value of n_s actually depends upon the relative stiffness of the column and the soil which should be estimated reasonably from the laboratory tests and the method of installation, it is hereby assumed as 6 (for illustration).

From the given data, T'_{90} = 0.00263. Using Fig.6, the value of N and H/r_c can be estimated as 4 and 40 respectively, from which the column parameters may be chosen as: r_c = 0.5 m and r_e = 2 m, although the column radius might be chosen depending upon the installation technique. The improvement factor obtained from the Fig.8 for t = 1 year is 2.926 which is well above the allowable limiting value of 2.75. Using Terzaghi's one dimensional consolidation theory, assuming the vertical permeability to be same as the horizontal permeability, the average surface settlement of the untreated soft ground at the end of 90% consolidation at the design embankment loading has been estimated as 4320 mm. The allowable settlement factor is therefore calculated as 500/4320 = 0.116, as against the actual value of 0.11 estimated from the Fig.7.

Table 1: Input parameters for field study.

Material	Parameter	Values: Redana (1999)	Values: Indraratna (2009)
Soil	k_h	2 x 10^{-10} m/s	1 x 10^{-9} m/s
	m_v	6.13 x 10^{-7} m^2/N	3 x 10^{-6} m^2/N
	k_s / k_h	0.1*	0.333$
	r_s / r_c	1.15*	2.5$
	H	17 m	20 m
Embankment	H_e	As reported by Redana (1999) [Given in Fig.5(b)]	4.3 m
	γ_e		20 kN/m^3
	Q		0
	K_p	3*	3*
Stone column /Vertical drain	r_c	0.025m	To be designed
	r_e	0.8475m (T1); 1.413m (T2)	
	n_s	4.72	As given in Figures 6-8
	α	As given in Fig.5	
	α_k		
* $ Assumed values ($ as per Indraratna, 2009)			

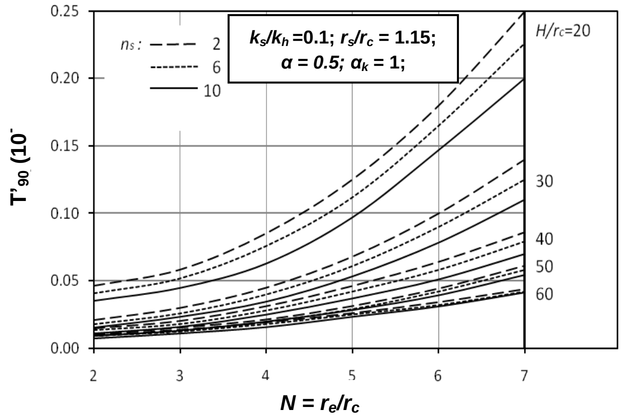


Figure 6. Variation of T'_{90} with N.

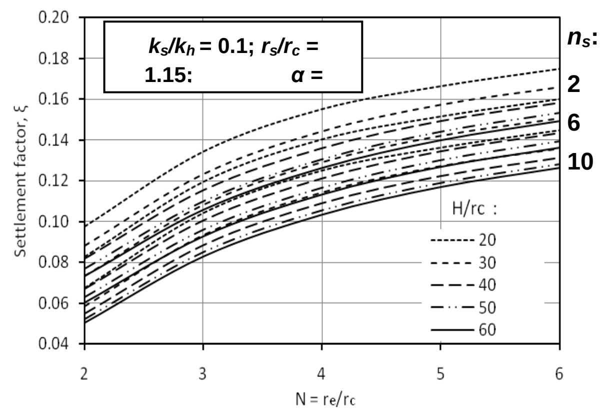


Figure 7. Variation of ξ with N.

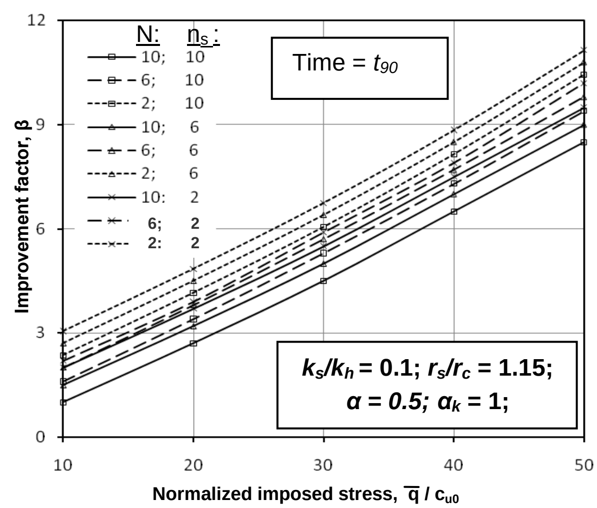


Figure 8. Variation of β with normalized imposed load intensity.

5 CONCLUSIONS

Considering the characteristics of stone column reinforced soft clay, a numerical solution based on unit cell theory was developed by the authors for computing the rate of consolidation, stress distribution, settlement and degree of post-consolidation ground improvement achieved. The free strain hypothesis is adopted for analysis which appears to be more realistic for embankment loading when the arching effect and clogging are taken into account. The comparison of the numerical results with the available solutions and field data indicates acceptable agreement which justifies the validity of the model. Based on the methodology, design recommendations followed by a set of design curves are developed. Considering a hypothetical case study on a recent ground improvement project at Ballina bypass, Australia, a design example is illustrated. It is expected that the work reported herein will go a long way in design of stone column reinforcement in soft clay deposit.

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