ANALYSIS OF SOFT GROUND REINFORCED BY COLUMNAR INCLUSIONS

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ANALYSIS OF SOFT GROUND REINFORCED BY COLUMNAR INCLUSIONS

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by

Muhammed Alamgir

Approved as to Style and Content by the Examination Committee:

Professor Norihiko Miura (Chairman)
Department of Civil Engineering, Saga University, Saga, Japan.

Professor Shigenori Hayashi
Institute of Lowland Technology, Saga University.

Dr. Koji Ishibashi
Associate Professor, Department of Civil Engineering, Saga University.

Dr. Akira Sakai
Associate Professor, Department of Civil Engineering, Saga University.

Professor Hormoz B. Poorooshasb
Department of Civil Engineering, Concordia University, Montreal, Canada.

To

my parents who taught me simplicity and responsibility

and

my elder sister who nurtured the same

ABSTRACT

A general method of analysis to solve an important class of problems encountered in the field of geotechnical engineering is developed in this dissertation. The analysis is formulated from the fundamental equation of equilibrium. It leads to develop a simple integro-differential equation to characterize the overall behaviour of the system. The behaviour of rigid columns (e.g. concrete/timber piles, lime/cement columns), deformable columns (e.g. stone columns/granular piles, sand compaction piles), pile groups, pile-raft foundations are the type of situations that can be analyzed by the proposed method. A numerical scheme using finite difference method is proposed to solve the governing equation with the aid of the relevant boundary conditions. A systematic process of trial is proposed to identify the possible slip and its magnitude developed at the column-soil interface. The numerical scheme can be used for end bearing and floating columns and for the situation involving stratified layers.

The behaviour of soft ground reinforced by a group of columnar inclusions such as stone columns/granular piles, sand compaction piles and lime or cement columns, are predicted using the proposed method of analysis. The reinforced ground is covered by a layer of granular fill. The response of reinforced ground ranges from flexible to rigid loading conditions depending on the magnitude of thickness and deformation modulus of overlaying granular fill. The compacted granular fill over the column reinforced ground is very effective in reducing both the overall and the differential settlements of the composite ground. The compressibility of the granular fill has an appreciable influence on the settlement response of the composite ground as long as the modulus of granular fill is less than approximately fifty times that of the soft ground.

The depth of slip zone at column-soil interface increases with the decreasing value of limiting shear stress. It also increases with the increase of degree of penetration of column. Slip situation predicts higher depth of neutral plane than its no slip counterpart and it increases with the decreasing value of limiting shear stress. In case of end bearing column, a good portion of column sustains little or no interface shear stress at all but for floating column, the whole length of column is subjected to shear stress either positive or negative. The stress in column increases with depth and attains a high value at the bottom for end bearing columns. But in case of floating column, it increases up to the depth of neutral plane beyond which decreases up to the bottom of column. In both the cases no slip situation predicts higher depth of neutral plane than that of possible slip. The differential settlement of the composite ground is noticeable for the case of uniform flexible loading acting over the entire area. The overall and the differential settlements are more for slip analysis than that of no slip case. As the differential settlement does not reduce in case of floating column, end bearing column is more effective due to giving less overall settlement. The influence of soil stratification is evident in the predictions. This is, of course, as expected; the present analysis quantifies it. The spacing, length to diameter ratio, the degree of penetration of columns, the relative stiffness of column and soil, and the angle of friction between column and soil have a significant influence on the mobilization of shear stress, variation of normal stresses in column and soil and the settlement of the treated ground. But the Poisson's ratio of soil has little influence on them.

A simple uncoupled consolidation model is proposed to determine the time-dependent response of soft ground reinforced by columnar inclusions. This method is simple compared to Biot's coupled consolidation theory. The radial inhomogeneity of the soil properties such as deformation modulus and shear modulus and also soil stratification can be handled easily by the proposed method. From predicted results, it is realized that to evaluate the subsequent response of reinforced ground, the value of degree of consolidation predicted at every nodal points should be used rather than using the average value. The mobilization of shear stress at column-soil interface, the stresses in column and soil and the settlement profile with time can be evaluated easily and reasonably accurately by the proposed method.

The proposed foundation model has been compared with the existing approaches and verified by the finite element analysis as well as experimental results both in the laboratory and in the field. The existing approaches can be used for rigid loading condition but their application is restricted for flexible loading. In both cases flexible and rigid loading, the proposed model offers better solution as it can be used by taking account the role of overlaying granular fill, soil stratification, slip and no slip situations, can be used for both end bearing and floating columns and also for time-dependent analysis. The comparison of results obtained from finite element method and those of by the proposed model, indicates that the proposed model can be used with a reasonable degree of accuracy to depict the settlement behaviour of end bearing or floating columns reinforced ground subjected to either flexible or rigid loading. The predictions obtained from the proposed model show also good agreement with test results both in laboratory and field.

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NOTATION

All notation and symbols are defined where they first appear in the text or figures. For convenience, the more frequently used symbols and their meanings are listed below.

A_c	plan area of column
a	radius of column
b	radius of the zone of influence of column
C_c	compression index
C_h	coefficient of consolidation in the vertical direction
C_v	coefficient of consolidation in the radial direction
c_g	geometry dependent constant
d_c	diameter of column
d_e	diameter of the zone of influence of column
e_o	initial void ratio
E	deformation modulus
E_c	deformation modulus of column
E_f	deformation modulus of granular fill
E_s	deformation modulus of soil
E_{so}	deformation modulus at the top of soil layer
E_{s1}	deformation modulus of upper soil layer
E_{s2}	deformation modulus of lower soil layer
F_d	downdrag force
G	shear modulus
H_f	thickness of granular fill

H_s	total depth of soil media
H_{s1}	thickness of upper soil layer
H_{s2}	thickness of lower soil layer
i	vertical nodal points in finite difference mesh
i'	vertical nodes up to which slip occurs
$imax$	maximum number of nodes in vertical direction
j	radial nodal points in finite difference mesh
$jmax$	maximum number of nodes in radial direction
K_0	coefficient of earth pressure at rest.
k_h	coefficient of permeability in the radial direction
k_v	coefficient of permeability in the vertical direction
L_c	length of column
m_g	slope of plastic portion of stress-strain curve
m_s	rate of increase of modulus with depth
n	spacing ratio of columns
p_o	applied stress at the ground surface
p_c	stress in column
p_s	stress in soil
r	radial distance measured from the center of column
s_c	spacing of columns
S_o	settlement of untreated ground
S_t	settlement of column reinforced ground
t	time
T_v	time factor for one-dimensional consolidation
u	excess pore water pressure
u_o	initial excess pore water pressure
U	degree of consolidation

w, w_z	vertical displacement component of reinforced ground
w_r	radial displacement component of reinforced ground
w_θ	displacement component in the direction of θ-axis.
z	vertical depth measured from the top of surface
z_c	vertical depth measured from the top of column
z'	depth of slip zone measured from the top of column
Δr	radial interval of nodes in finite difference mesh
Δz	radial interval of nodes in finite difference mesh
σ_{zz}	normal stress component
σ'_{zz}	effective stress
σ_{rz}	shearing stress component
γ'	effective unit weight of soil
γ_{rz}	shear strain
ϕ	angle of friction of soil
ϕ_c	angle of friction of column material
δ	angle of friction between column and soil
τ	shearing stress
τ_f	ultimate shearing stress
ν_c	Poisson's ratio of column material
ν_f	Poisson's ratio of granular fill
ν_s	Poisson's ratio of soil

CHAPTER ONE

INTRODUCTION

1.1 General

The demand for the improvement of marginal sites are increasing continuously for construction of infrastructural facilities of cities due to the ever increasing and growing trend of urbanization. Since most large cities have been developed along large rivers and estuaries, the subsoils usually consist of soft materials of alluvial or diluvial age, which were deposited in relatively recent times and have generally remained in incomplete state of consolidation. Generally, most of the alluvial deposits are loose or soft in nature having low strength and high compressibility. Many sites have sensitive soils, in the sense that their strength is reduced significantly when subjected to disturbance. Foundation failure in soft ground, both due to lack of sufficient bearing capacity or excessive settlement, is very common. Surface loading beyond yield stress levels due to embankments or shallow foundation, etc., inevitably results in large total and differential settlements and to instability. Some typical regions of soft clay deposits around the world is shown in Fig.1.1.

As extensive urbanization and industrialization continues on soft ground, geotechnical engineers have been trying to solve this problem technically and economically for a very long time. Some of the traditional options are: change of site, designing the proposed

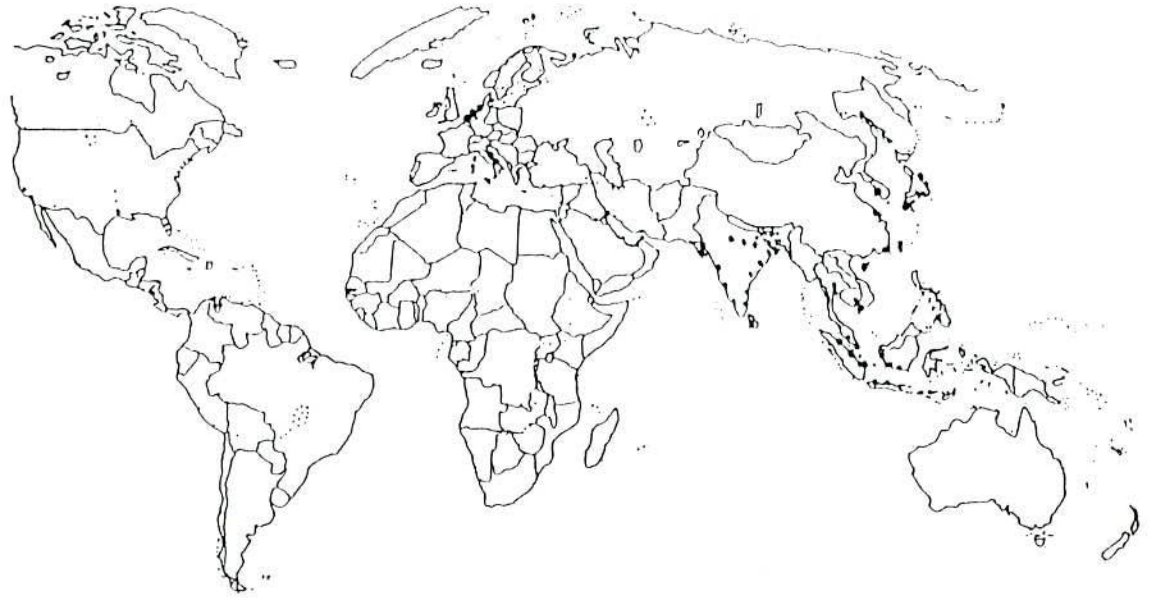


Figure 1.1 Some typical soft soil regions of the world.

structures accordingly, excavation and replacement with suitable soil, deep foundations placed through the unsuitable soils, wait until natural consolidation occurs, or stabilization with injected additives. In many situations, the conventional foundation systems could not be chosen to solve soft ground problems due to the several environmental constraints and because of their expensive and time consuming nature. These inherent limitations of conventional foundation systems led to the development of modern foundation practice, namely, ground/soil improvement, which has proved as a viable alternative both technically and economically.

The basic concept of soil improvement, namely, drainage, densification, cementation, reinforcement, drying and heating, were developed hundreds of years ago and remain valid today. Availibility of machines in the 19th century resulted in vast increases in both the quantity and quality of work that could be done. Among the most significant developments of the past 65 years are the introduction of vibratory methods for the densification of cohesionless soils which, in turn, transform the in-situ soil into stiffer and stronger columns, injections and grouting of materials, mixing of reactive materials such as lime or cement with soft clay deposits and the new concept of soil reinforcement. Due to these innovations of ground improvement and the everincreasing value of land, the development of marginal sites, once cost prohibitive and time consuming, is now economically feasible. The increasing costs of conventional foundations and the numerous environmental constraints greatly encourage the in-situ improvement of weak soil deposits.

Among the various techniques for improving *in situ* soft ground conditions, columnar inclusions is considered as one of the most versatile and cost effective ground improvement technique compared to the other methods such as preloading, dredging and replacement, dynamic compaction, thermal stabilization and ground freezing. Soil displacement techniques can no longer be used due to environmental restrictions and post construction maintenance expenses. The columnar inclusions can be of the form such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., which are stiffer and stronger than the surrounding soil. They are ideally suitable for soft clays and silts and also for loose granular deposits. This technique has been and is being used in many difficult foundation sites throughout the world to increase the bearing capacity, reduce settlement, increase the rate of consolidation, improve stability and resistance to liquefaction. The applicability of this ground improvement technique has already been proven throughout the world by its implementation in various geotechnical engineering projects (Greenwood 1970, Baumann & Bauer 1974, Aboshi et al. 1979, Barksdale & Bachus 1983, Broms 1984, Mitchell & Huber 1984, Kimura et al. 1985, Miura et al. 1986 & 1987, Bergado et al. 1991, Takemura et al. 1991, Ranjan & Rao 1994 and Asaoka et al. 1994).

1.2 Historical Background

The oldest historical evidence of the use of columnar inclusions is found in 1830's when the French military engineers used stone columns/granular piles to support heavy foundations of iron works at the artillery arsenal in Bayoune (Hughes & Withers 1974). During the same period i.e. between the 1830 to 1850, sand compaction piles were constructed in Japan (Ichimoto 1981). However, the modern origins of the stone columns/granular piles foundations truly began in 1930's in Germany by Russian Emigres, Sergei Stenermann and Wilhem Degen as a by product of the technique of vibroflotaion for the compaction of cohesionless soils both above and below the water table (Glover 1982). On the other hand, large diameter compacted sand columns were constructed in Japan in 1955 using the compozer technique (Aboshi et al. 1979). The vibro-compozer method of sand compaction pile construction was developed by Murayama in Japan in 1958 (Murayama 1962). Although, lime/cement mixing method has been used to improve the properties of soils near the ground surface since olden times, deep stabilization of soft soils with lime and/or cement stabilized columns has been the subject of research in Sweden, Japan and other countries in recent times (Bergado et al. 1994). The modern application of this method for deep mixing of in-situ soils (in the form of lime or cement columns or walls) started in the late 1970's in Japan (Terashi et al. 1979, Kawasaki et al. 1981 and Suzuki 1982). After the beginning of the modern phase of the use of columnar inclusions, the theoretical background, analysis and design aspects, and installation techniques have been developed by various researchers and the practicing engineers. As a result, this method of ground improvement has been used extensively throughout the world for site improvement. It has now reached a stage where design methods have changed from empiricism to a rational approach.

At first, an empirical design was suggested by Thornburn and MacVicar (1960) for composite ground constructed by vibroflotation method. This was followed by development of a rational method by Gibson & Anderson (1961) to evaluate the limiting stress of a cylindrical cavity which, in turn, is used to obtain the load carrying capacity of composite ground. Since then, several analytical methods for the determination of the supporting capacity and load-settlement behaviour of column reinforced ground have been developed (Greenwood 1970, Vesic 1972, Tanimoto 1973, Hughes & Withers 1974, Priebe 1976, Madhav & Vitkar 1978, Aboshi et al. 1979, Balaam & Booker 1981, Hansbo 1987, Enoki et al. 1991, Van Impe & Madhav 1992, Alamgir et al. 1994, Ranjan & Rao 1994, Madhav et al. 1994, Bouassida & Hadhri 1995 and Alamgir et al. 1995). A considerable number of numerical methods based on either finite difference or finite element methods have also been developed for the predictions of behaviour of such improved ground (Balaam et al. 1977, Schweiger & Pande 1986, Canetta & Nova 1989, Poorooshasb et al. 1991, Asaoka et al. 1994 and Madhav & Van Impe 1994). A large number of laboratory and field tests have been conducted in order to quantify the applicability of this ground improvement technique to improve the behaviour of soft ground (Hughes et al. 1975, McKenna et al. 1975, Rao & Bhandari 1977, Terashi & Tanaka 1981, Madhav 1982, Charles & Watts 1983, Kimura et al. 1985, Mitchell & Huber 1985, Bergado & Lam 1987, Juran & Guermazi 1988, Madhav & Thiruselvan 1988, Bergado et al. 1988, Honjo et al. 1991, Al-Refeai 1992, Leung & Tan 1993, Ekstrom et al. 1994, Pan et al. 1994 and Miura & Madhav 1994).

1.3 Objective of this Research

The cardinal aim of this research work is to develop a general approach to determine the behaviour of soft ground reinforced by columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., which the practicing engineers can use with a high degree of confidence. The soft ground reinforced by different types of columnar inclusions are categorized here as a single foundation type i.e. a composite ground consisting of stiffer and stronger columns and the surrounding soft soil. With some idealization a general approach is developed from the standpoint of foundation analysis. A theoretical model, simple in concept and computations but versatile in applications, is proposed to predict the settlement response of the column-reinforced ground that is covered by layer of granular fill subjected to uniform loading over the entire area. The backbone of this analysis was developed by Poorooshasb and Bozozuk (1967) who, in turn, used a concept proposed by Hill (1963). It is a straight forward approach as it advocates the use of a simple kinematically admissible displacement field and attempts to obtain the overall equilibrium of the system. The paper of Poorooshasb and Bozozuk (1967) contains a closed form solution which utilized a simple linear constitutive law. The present study can handle versatile aspects which often are encountered in geotechnical engineering problems. The proposed model incorporates the nonlinearity of the material behaviour, the interaction as well as the stress transfer between the columns and the surrounding soil along the depth. The compressibility of the granular fill placed over the reinforced ground are considered to account the actual contribution of granular fill on the overall settlement response of the reinforced ground. The analysis can be done for the situation of possible slip along the column-soil interface and the time-dependent response of the reinforced ground due to the consolidation of surrounding soft soil. The proposed model can also handle certain types of material inhomogeneity (i.e. radial inhomogeneity), different column geometry, soil stratification and end bearing or floating columns. The results from proposed model are compared with those from existing approaches and verified by the finite element analysis. The experimental results both in laboratory and field are also compared with the predictions obtained by using the proposed model. Parametric study is also carried out to illustrate the influence of various parameters on the predicted behaviour of soft ground reinforced by columnar inclusions.

1.4 Thesis Arrangement and Outline

The dissertation is written in the following sequence. A critical review of available literature pertaining to the ground improvement techniques with the use of columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., is presented in Chapter Two. The mechanisms of columnar inclusions in improving the properties of soft ground are briefly stated. The historical background and the chronological development of this method both in theoretical and practical aspects are also reported. The existing analytical and numerical solutions and the experimental investigations are discussed followed by their limitations in obtaining the rational solution of the encountered problem. From the review of literature, it has been revealed that the following aspects such as material nonlinearity, column-soil interaction as well as stress transfer between the column and the soil along the depth, the exact role of the granular fill placed over the reinforced ground, the possible slip between the interface of column and soil, and the time dependent behaviour resulting from consolidation of the soft ground must be incorporated in the model for the rational design of the column-reinforced ground.

Chapter Three deals with the development of the governing equations in conjunction with the appropriate boundary conditions that are needed for a unique solution. A numerical scheme of finite differences is also developed for the solution of the governing equations. A single vertical column of circular cross section, fully penetrating into the soft ground and cylindrical coordinates are considered to develop the governing equation. The central hypothesis for the development of the governing equation is that the radial displacements are negligibly small and the only remaining component is the vertical displacement. The boundary conditions are introduced at the column-soil interface and at the outside boundary of the influence zone, to account for compatibility and possible slip at the column-soil interface, and the group effects of columns, respectively.

Chapter Four describes in detail the development of a foundation model, its solution based on the governing equation and the numerical scheme developed in the Chapter Three. A layer of granular fill overlaying the soft ground improved by a group of columnar inclusions subjected to uniform loading over the entire area is taken into consideration. The behaviour of the improved ground is predicted for the following cases: (i) end bearing columns in stratified or non stratified soil systems and (ii) floating columns in stratified or non stratified soil systems. Consideration is also given to account possible slip at the interface of column and the surrounding soil. Numerical evaluations are made to illustrate the influence of various parameters such as (i) thickness and deformation modulus of granular fill; (ii) spacing and length to diameter ratio of columns; (iii) degree of penetration of column into the soft ground;

(iv) the modulus of deformation and the Poisson's ratio of surrounding soft soil, (v) relative stiffness of column and surrounding soil, and (vi) angle of friction between column and soil. The predictions reveal that the settlement response of the column-reinforced ground is greatly influenced by the presence of granular fill at the top. The effects of spacing and length to diameter ratio of columns, the relative stiffness of column and soil and the degree of penetration of column, are found to be significant on the distribution of shear stresses with depth, the load sharing between column and soil and the settlements of the improved ground. It is observed that the Poisson's ratio of soil has little influence on the above quantities.

The time-dependent behaviour of the column-reinforced ground is described in Chapter Five. The columnar inclusions such as stone columns/granular piles and sand compaction piles driven into soft clay deposits act as reinforcing elements and as drains. They accelerate the rate of consolidation as the radial drainage is predominant. For the prediction of the rate of settlement, a solution is required to evaluate the vertical consolidation of clay which is due to the expulsion of pore water by vertical and radial flow towards the column. In the present time-dependent analysis, the "Diffusion Theory" which is an extension of Terzaghi's one dimensional consolidation theory (Terzaghi 1925) is used. The governing equations are solved numerically by finite difference method in conjunction with the appropriate boundary conditions to obtain the excess pore water pressure for any time $t>0$ at any point in the surrounding soil media. The stress concentration in column due to its higher stiffness than the surrounding soil, is not considered to evaluate the excess pore water pressure. These uncoupled excess pore water pressures are then used to obtain the time-dependent response of soft ground reinforced by columnar inclusions. The solutions can be used for the non stratified and stratified soil systems. Predictions are made to depict the dissipation of excess pore water pressure, distribution of load between the components of the system with time and the time-settlement relationship of the improved ground. Parametric study is also performed to illustrate the influence of horizontal to vertical permeability ratio of soil and the spacing of columns.

The validation of the proposed model with theoretical and experimental results is presented in Chapter Six. The proposed model is compared with the existing approaches, verified by the finite element analysis using CRISP program, and also compared with some laboratory and field experimental results. The existing approaches can be used for rigid loading condition but their applicability is restricted for flexible loading. It is observed that good agreement exists between the predictions by the proposed model and those by the finite element analysis. From the comparison of experimental results, it is revealed that the proposed model can be used with a reasonable degree of accuracy to predict the behaviour of the soft ground reinforced by columnar inclusions both in the field and laboratory.

Finally, a summary and the conclusions are presented in Chapter Seven.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., have been used as a ground improvement technique to increase the bearing capacity, reduce settlement, increase the time rate of consolidation, improve stability and resistance to liquefaction of soft ground since 19th century. However, the modern origins of this type of ground improvement technique truly began in 1930's in Germany and in 1950's in Japan. A brief account of the historical development has been presented in section 1.2. In the modern phase of the use of columnar inclusions, the theoretical background, analysis and design aspects and installation techniques have been developed by various researchers and practicing engineers and this method of ground improvement is being used extensively throughout the world for site improvement. It has now reached at a stage where design methods have changed from empiricism to a rational approach.

Amongst the various techniques for improving *in situ* soft ground conditions, columnar inclusions are considered as one of the most versatile and cost effective ground improvement techniques. They are ideally suitable for the improvement of soft clays and silts and also for loose granular deposits. The concepts have been developed to explain the reinforcing mechanism of columnar inclusions into soft ground from various point of views. Several design methods have been developed for the determination of the supporting capacity and load-settlement behaviour of column reinforced ground based on empirical estimates and purely theoretical aspects. A large number of theoretical models have been developed to assess the behaviour of soft ground reinforced by columnar inclusions. In the theoretical models both the analytical solutions and numerical techniques are employed. The numerical techniques are developed using finite difference and finite element methods. A large number of experimental investigations have been carried both in the laboratory and field to bring out the effects of various parameters on the load carrying capacity, load distribution and the settlement characteristics of the reinforced soft soil system using columnar inclusions. A comprehensive review of the state of the art of the theoretical and experimental studies associated with ground improvement by using columnar inclusions is presented in the following sections.

2.2 Ground Improvement Techniques

The need of ground improvement in civil engineering projects has become inevitable because of the presence of extensive deposits of very soft soils in the plains and shortage of lands in Japan, Southeast Asia and other countries. Various ground improvements techniques are discussed here that have been tested to provide improvement in soil strength, mitigation of total and differential settlements, shorten construction time, reduce construction costs, and various other characteristics which may have impact on their utilization to specific projects in soft ground. In general, the term soft ground includes soft clay soils, soils with large fraction of fines such as silts, clayey soils which have high moisture content, peat foundations, and loose sand deposits just above or under water table (Kamon & Bergado 1991). Table 2.1 represents an outline for identification of soft ground according to the types of structures. It may be noted that the criteria are different and depend on the structures constructed. The general ranges of N-values (SPT), unconfined compressive strength, q_u, cone penetration resistance, q_c, and the water content of these soft ground are also stated in the table.

Table 2.1 Outline for Identification Soft Ground (after Kamon & Bergado 1991).

Structures	Soil conditions	N-values (SPT)	q_u (kPa)	q_c (kPa)	Water content (%)
Road	A: Very soft B: Soft C: Moderate	Less than 2 2 to 4 4 to 8	Less than 25 25 to 50 50 to 100	Less than 125 125 to 250 250 to 500	
Express Highway	A: Peat soil B: Clayey soil C:Sandy soil	Less than 4 Less than 4 Less than 10	Less than 50 Less than 50 -		More than 100 More than 50 More than 30
Railway	(Thickness of layers) More than 2m More than 5m More than 10m	0 Less than 2 Less than 4			
Bullet train	A B	Less than 2 2 to 5		Less than 200 200 to 500	
River dike	A: Clayey soil B: Sandy soil	Less than 3 Less than 10	Less than 60		More than 40
Fill dam		Less than 20			

To improve the physical and mechanical properties of the above mentioned soft ground, several ground improvement techniques have been and are being used since the 19th century. The different soil improvement methods can be classified into geometrical, mechanical, physical and chemical, and structural methods as follows depending on how the methods affect the stability or reduce the settlement (Broms 1987):

(i) Geometrical methods: where the moment or force causing failure or excessive settlement is reduced; (a) Floating foundation and (b) Light weight fills.

(ii) Mechanical methods: where the shear strength is increased or the compressibility reduced primarily by reducing the water content of the soil; (a) Preloading (often combined with vertical drains to increase the consolidation rate), (b) Lime piles and (c) Heating.

(iii) Physical and chemical methods: where the shear strength is increased and the compressibility of soft clay reduced by altering the clay-water system e.g. by freezing or by mixing the soil with lime, cement or other chemicals; (a) Lime or cement columns, (b) Electro-osmosis and (c) Freezing.

(iv) Structural methods: where structural elements such as geofabric, piles, sand, gravel or stone columns are used to reinforce the soil or to transfer the load to an underlying less compressible stratum or layer; (a) Geofabrics and geomembranes, (b) Excavation and replacement, (c) Soil displacement, (d) Heavy tamping/Dynamic consolidation, dynamic replacement, and mixing, (e) Jet grouting, (f) Stone, gravel or sand columns, (g) Embankment piles and (h) Soil nailing.

There are several different ground improvement techniques as mentioned above. Each has its own advantages, limitations, and special applications. Therefore, none can be considered suitable for solution of all problems in all soils. For soft and cohesive soils in subsiding environments, ground improvement by reinforcement (i.e. stone columns or sand compaction piles), by admixtures (i.e. by deep mixing method) and by dewatering (i.e. vertical drains) are applicable. For loose sand deposits, various in-situ compaction methods are applicable such as heavy tamping/dynamic compaction, resonance compaction, vibroflotation, sand compaction piles, etc. Considering the factors such as significance of the structures, applied loading, site conditions and period of construction, etc., decision should be taken about the possible application of soil improvement method suitable for soil types and their problems. Applicability of soil treatment methods for different soil types is outlined in Table 2.2. The improving period and the improvement state of soil for different ground improvement techniques are also depicted in this table. For detailed account about the various types of ground improvement techniques, their mechanisms of improvement, installation techniques, advantages and disadvantages including cost effectiveness, reference can be made to Mitchell (1981), Broms (1987), Kamon (1991) and Bergado & Miura (1994).

Table 2.2 Applicability of Ground Improvement for Different Soil Types (after Kamon 1991).

Improvement mechanism		Reinforcement	Admixtures	Compaction	Dewatering
Improving period		Depending on the life of inclusion	Relatively short-term	Long-term	Long-term
Soil types	Organic soil				
	Volcanic clay soil				
	Highly plastic soil				
	Lowly plastic soil				
	Silty soil				
	Sandy soil				
	Gravel soil				
Improve state of soil		Interaction between soil and inclusion	Cementation	High density by decreasing void ratio	
		(No change in soil state)	(Change in soil state)		

Since the domain of ground improvement is indeed very vast, it is often a difficult task to select a particular type of ground improvement technique. The selection of the most suitable one in any case can only be made after evaluation of several factors specific to the problem at hand. Most important considerations among these are (Mitchell 1981): (i) The purpose to which the treated ground will be put. This will establish the level of improvement required in terms of properties such as strength, stiffness, compressibility and permeability. (ii) The area, depth and total volume of soil to be treated. (iii) Soil type and its initial properties. (iv) Material availability; e.g. sand, gravel, water and admixtures. (v) Availability of equipment and skills. (vi) Environmental factors-waste disposal, erosion, water pollution, effect on adjacent structures and facilities. (vii) Local experience and performances. (viii) Time available and (ix) Cost. About the selection of a particular technique, Schlosser and Juran (1979) made an excellent comment, when dealing with techniques of soil improvement, experience has almost always preceded theory. However, keeping in mind the above factors and the comments, the following flow chart can be used to select a suitable method for ground improvement. In the flow chart, only deep ground improvement techniques are stated consistent with the present study.

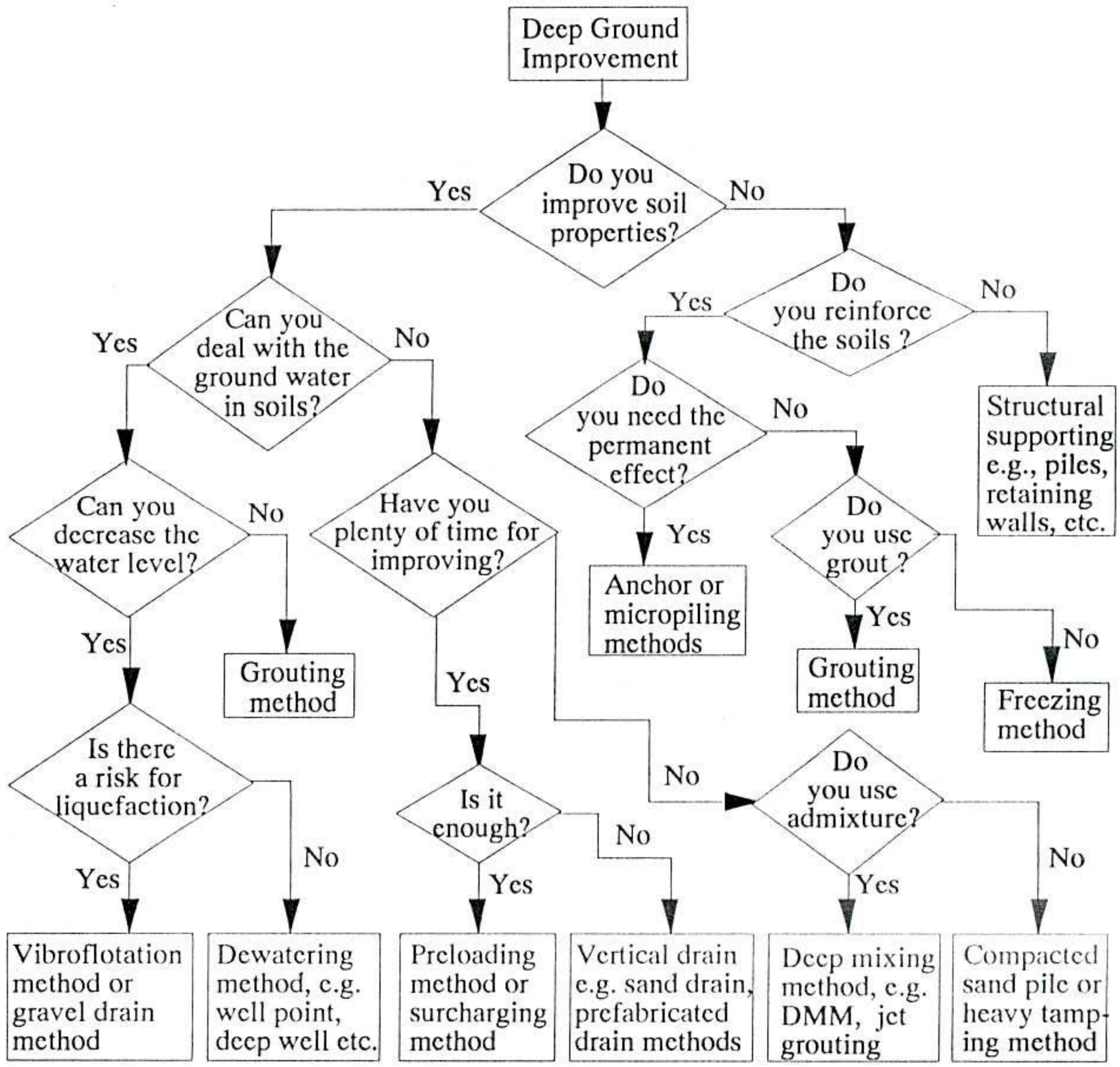


Figure 2.1 Selection flow of deep ground improvement techniques (after Bergado & Miura 1994).

2.3 Ground Improvement by Columnar Inclusions

To utilize the marginal sites and to make the many problematic soils into useful construction sites, soil improvement has become a part of many present day civil engineering projects. The various techniques by which this improvement can be accomplished are discussed briefly in the previous sections. The flow chart to choose a ground improvement technique suitable for a particular project considering the associated situations are also discussed. Amongst the various ground improvement techniques for improving *in situ* ground conditions, columnar inclusions is considered as one of the most versatile and cost effective ground improvement technique (Alamgir et al. 1995). The columnar inclusions can be of the form such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc.,which are stiffer and stronger than the surrounding soil. The theoretical and constructional aspects of ground improvement by columnar inclusions have been developed intensively after the beginning of modern phase and have been used extensively throughout the world in the past several years. The horizon of applicability and the advantageous aspects of this method are wider than any other ground improvement technique. They are applicable for all types of soft soils ranging from soft clays to loose granular deposits. Installation techniques also ranging from simple mechanical equipment to sophisticated computerized one. As the other conventional ground improvement techniques such as preloading, dredging and soil displacement techniques can often no longer be used due to environmental restrictions and post construction maintenance expenses (Barksdale and Bachus 1983), the columnar inclusions can be treated as an ideal choice for today's soft ground improvement projects. Advantageous aspects of columnar inclusions over other conventional methods may be described as the followings: (i) Moderate increase in load carrying capacity; (ii) Significant reduction of ground settlement; (iii) Granular columns being free draining, post-consolidation settlement will be small; (iv) Installation is relatively simple and involves low energy input or moderate labour; (v) Increase in resistance to liquefaction and (vi) Cost effectiveness.

The ground improvement technique such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., are categorized here in to a single group of ground improvement technique, namely, columnar inclusions, although the constituent materials and their techniques of formation are different. Various techniques of installation have been conceived for various types of columnar inclusions in a wide variety of soils such as loose sandy to soft compressible soils depending on technical ability, efficiency and local conditions. In Europe and U.S.A., the vibroflotation technique i.e. vibro-compaction and vibro-replacement, is widely used for stone columns/granular piles installation (Baumann & Bauer 1974 and Engelhardt & Golding 1975) while in Japan, the vibro-compozer method is widely used for the installation of sand compaction piles (Aboshi et al. 1979 and Aboshi & Suematsu 1985). In India, granular piles are constructed by simple bored piling equipment (Datye & Nagaraju 1975 and Datye 1978, 1981). For the construction of lime or cement columns deep mixing method (DMM) and dry jet mixing method (DJM) are generally used (Broms 1987 and Miura et al. 1987). The physical and mechanical properties of column materials and their interactions with the surrounding soil are also different. These factors lead the columns to behave in a different manner in post installation phase and during loading.

2.4 Mechanism of Improvement

Stone columns/granular piles and sand compaction piles are generally cylindrical in shape and composed of compacted gravel, crushed stone or sand. They are inserted into the soft ground by partial or full displacement methods. A considerable volume of weak soils is replaced by the granular materials. The presence of the columns creates a composite ground of lower overall compressibility and higher shear strength than the native soil. Confinement and thus stiffness of the granular material, is provided by the lateral stress within the weak soil. Upon application of vertical stress at the ground surface, the granular column and the weak soil move downward resulting in a concentration and transfer of stress to the column. Depending on the loading condition, the shear stress is mobilized along the column-soil interface which results in the sharing of load between the components of the system. The resulting stress concentration in the column is primarily due to the column being stiffer and stronger than the ambient soil. An axial load applied at the top of a single granular column produces a large bulge to a depth of 2 to 3 diameters of column beneath the surface. This bulge, in turn, increases the lateral stress within the clay which provides additional confinement of the granular material. An equilibrium state is eventually reached resulting in reduced vertical movement when compared to the unimproved soil. Granular columns installed in a group and loaded over the entire area undergo considerably less bulging than for a single column. Moreover, since the component material is granular with high permeability, granular columns can also accelerate the consolidation settlements, and consequently the strength gain of surrounding clay subsoil due to the vertical and radial flow of pore water.

Lime or cement columns are constructed in-situ by mechanically mixing lime or cement with soft clay. The increase in strength and decrease in compressibility of the soft clay result from the reaction of the clay with lime and/or cement through the process of ion exchange and flocculation as well as pozzolanic reaction. The divalent calcium ions replace the monovalent sodium ions in the double layer surrounding or the individual clay particles. Thus fewer number of divalent calcium ions are needed to neutralize the net negative surface charge of each clay mineral, reducing the size of double layer, and thereby, increasing the attraction between the clay particles leading to a flocculated structure. Furthermore, the silica and alumina in the clay mineral react with the calcium silicates and calcium aluminate hydrates to form a cementing gel in a process called pozzolanic reaction. As a result, the undrained shear strength of the clay stabilized with lime or cement increases with time. After the formation of stiffer and stronger lime or cement columns in the comparatively softer soil media, they form a composite ground. Subsequent response of the loaded composite ground can be obtained from the consideration of stress concentration on column and stress distribution between the components resulting from the mobilization of shear stresses along the column-soil interface.

2.5 Analytical Solutions

The beginning of the modern phase of soft ground improvement by columnar inclusions was accompanied by the simultaneous development of design methods. At first, an empirical design method was suggested by Thornburn & MacVicar (1960) to evaluate the load carrying capacity of the composite ground constructed by vibroflotation method. This was followed by a rational formula proposed by Gibson & Anderson (1961) to evaluate the limiting radial stress into the cylindrical cavity. They assumed that the surrounding clay media of the cylindrical probe behaves like an elasto-plastic material. The soil properties such as undrained shear strength, elastic modulus and the Poisson's ratio are required for the evaluation. This formula can be used to evaluate to ultimate vertical load that a compacted column placed into the cylindrical probe, can carry.

Greenwood (1970) proposed that the clay surrounding the stone column can be expected to mobilize passive pressure conditions during failure. The stone column material gets compressed axially and expands laterally. He developed an equation to determine the ultimate lateral resistance of soil based on in-situ stresses, cohesion of soil and the Rankine passive earth pressure coefficients. The ultimate vertical load that can be carried by stone column is evaluated using the classical relationship between the lateral and the vertical stresses. The plane strain condition is considered. An empirical curve giving the settlement reduction due to ground improvement with stone columns as a function of undrained soil strength and stone column spacing are also presented.

Vesic (1972) developed the cavity expansion theory which constitutes the main theoretical basis for the estimation of the yield stress or maximum vertical stress on a stone column, beyond which excessive deformations would occur. The cavity expansion theory can be applied to evaluate the vertical yield stress according to the derived equation. The parameters needed for the predictions, are undrained shear strength of the soil, effective mean normal stress, angle of shearing resistance, modulus of elasticity and the Poisson's ratio of soil. Numerical evaluation are made in the form of tables and graphs suitable for application in engineering practice. The solution takes into account the effects of volume change in the plastic region. They can also be used to evaluate pore water stresses if the cavity expansion takes place in undrained conditions. For the case of purely cohesive and incompressible soil, Vesic's

(1972) theory become identical to that of Gibson and Anderson (1961).

Tanimoto (1973) proposed a design procedure of sand compaction piles depending on the type of soil to be stabilized. In stabilizing loose sandy ground, the design is directed to calculate a void ratio of the soil required to assure stability against failure and provide tolerable settlement. The design is based on a concept of a composite ground made of clay soil and sand piles, in which the load of the structure above ground is concentrated of the sand piles. As the load on the sand piles increases, their shear strength also increases and the load on the clay decreases. This results in an increase in bearing capacity and decrease in settlement. A simple equation is proposed to calculate the stresses on columns and the surrounding soil, which makes it possible to pursue computations of bearing capacity and settlement, if the stress ratio (ratio of stress on column to that of soil) is known. For simplicity, a constant value of stress ratio in the range of 3 to 4 is considered. It is also suggested that the value of stress ratio is required to check by future studies. In stability analysis of a slope, the combined shear resistance by sand piles and clay soil is assumed to act on a potential slip surface.

Hughes and Withers (1974) proposed a method for the analysis of ultimate capacity of the single stone column based on model experiments. The bulging type failure of a single stone column is observed in the laboratory tests. In their approach the elastic-plastic theory given by Gibson & Anderson (1961) for a frictionless material and an infinitely long expanding cylindrical cavity is used for predicting the undrained, ultimate lateral stress of the soil surrounding the stone column. The ultimate vertical stress that the stone column can carry is related to the ultimate lateral stress and the ultimate vertical stress considering the stone column to be confined in a triaxial stress system.

The reduced stress method, proposed by Priebe (1976), for estimating reduction in settlement due to ground improvement with stone columns uses the unit cell concept. The stone column is assumed to be in a state of plastic equilibrium under a triaxial stress state. The soil within the unit cell is idealized as an elastic material. Since the stone column is assumed to be incompressible, the change in volume within the soil is directly related to vertical shortening of the cylindrical column. The radial deformation of the elastic soil is determined using an infinitely long, elastic hollow cylinder solution. The elastic cylinder of soil, which has a rigid external boundary coinciding with the boundary of the unit cell, is subjected to a uniform external pressure. The methods neglects the deadweight of the soil and columns. Other assumptions made in the analysis include (i) equal vertical settlements of the stone column and the surrounding soil, (ii) uniform stresses in the two materials, and (iii) end bearing column on to a rigid layer. The solution considers some form of interaction between the soil cylinder and the column. The settlement improvement is expressed by Priebe in terms of the ratio of the settlement of the untreated ground to that of treated ground. Priebe proposed some design charts in which this settlement improvement is related with the area replacement ratio i.e. the proportion of the plan area that is covered by stone columns and the angle of internal friction of the stone.

Seed and Booker (1977) presented a method to determine the stability of potentially liquefiable sand deposits using gravel drains. Due to the installation of gravel drains, the generated pore water pressure due to repeated loading may be dissipated almost as fast as they are generated. In this method, the one-dimensional theory of pore-water pressure generation and dissipation developed by Seed et al. (1975) is generalized to three dimensions and applied to the analysis of columnar gravel drains under a variety of earthquake conditions. The results of these analyses are summarized as a series of charts that provide a convenient basis for design considerations.

The ultimate bearing capacity of strip footings constructed on soft soil stabilized with granular trench has been studied by Madhav and Vitkar (1978) using plane strain analysis. They postulated a failure mechanism for such foundations and derived analytical expressions for the ultimate bearing capacity using the generalized Prandtl's mechanism i.e. upper bound theorem of limit analysis. The equation proposed by them, to calculate the ultimate load carrying capacity of a granular trench, is similar to the bearing capacity equation of a shallow footing suggested by Terzaghi for ideal soil condition. From this study they reaffirmed that a granular trench significantly reinforces weak soil deposits.

The equilibrium method, proposed by Aboshi et al. (1979), is based on the concept that vertical stress concentration on the sand column gives a reduced average stress on the soft soil. Important parameters required to estimate in this method are the stress concentration ratio i.e. ratio of stress on column to the stress on soil, and the area replacement ratio. The rest of the approach is very simple and straight forward. The replacement ratio is determined by the knowledge of stone column diameter and spacing. The stress concentration ratio must be estimated using past experience and the results of previous field measurements of stress. If a conservatively low stress concentration ratio is used, a safe estimation of the reduction in settlement due to ground improvement will be obtained. The following considerations are taken into account in developing the equilibrium method: (i) the extended unit cell idealization is valid, (ii) the total vertical load applied to the unit cell equals the sum of the force carried by the column and the soil (i.e. equilibrium is maintained within the unit cell), (iii) the vertical displacements of stone column and surrounding soil are equal, and (iv) a uniform vertical stress due to external loading exists throughout the length of stone column. Because of its simplicity, versatility and reasonably good assumptions made in its derivation, the equilibrium method offers a practical approach for estimating settlement reduction due to ground improvement with stone columns (Barksdale & Bachus 1983).

Balaam and Booker (1981) proposed an analytical solution to predict the settlement of rigid foundation on soft clay stabilized by large number of fully penetrating stone columns. The expressions for evaluating the moment and shear distributions across the foundation are also given. It is assumed that the stone columns and clay remain elastic i.e. there is no slippage between the column and the surrounding soil throughout the range of applied load. The solution is obtained from the analysis of a "unit cell" with the equal vertical strain assumption. It is assumed that the vertical stresses are almost uniform on horizontal planes in the stone column and also uniform in the cohesive soil. The stress state in the unit cell is essentially triaxial despite the consideration of underlying firm strata which may rough or smooth. The two regular arrangements of stone columns used in practice, square and triangular, have been considered. Parametric study is carried out to illustrate the influence of different design parameters. The results obtained by this method are also compared with finite element solutions and a good agreement is obtained. This analytical method is relatively simple and can be calculated swiftly to show the effect of many parameters governing the solution. Later, the same authors (Balaam & Booker 1985) extended this method to develop an interaction analysis which takes account of yielding within the stone column. This interaction analysis is based on a number of plausible assumptions regarding the behaviour of the clay and columns. The column is assumed in a triaxial state and there is no shear stress at the column-soil interface. It is also assumed that there is no yield in the surrounding soil so that its behaviour is entirely elastic. In order to check the validity of these assumptions elasto-plastic finite element analysis has been performed and the agreement between the two methods is found to be reasonably good.

Hansbo (1987) proposed a method to determine the bearing capacity and the settlement of lime or cement columns. The bearing capacity of lime or cement columns depends on the shear strength in planes of weakness that are most likely to exist in the columns. The settlement of the column is calculated based on "equal strain" theory provided the stress on column should not exceed creep limit. It is suggested that the vertical pressure leading to creep failure of the columns can be calculated according to Holm and Ahnberg (1986). When the columns are placed in groups, local shear failure or block failure may take place in a similar way as for floating pile groups. As the investigation of lime columns in normally consolidated Scandinavian clays has shown that the permeability of the lime columns is 100 to 1000 times higher than that of the clay itself, consolidation of the column reinforced clay can be calculated based on the theory of sand drains.

Enoki et al. (1991a) made a comparative study of the predicted bearing capacities of clayey ground improved by the installation of sand columns by adopting different limit equilibrium methods of analyses. Fellenius method is compared with Terzaghi (1943), Caquot-Kerisel (Vesic 1975), slip line and generalized limit equilibrium methods, GLEM (Enoki et al. 1991b). In the homogeneous cohesive ground, the solutions agree with each other. The bearing capacity obtained by the Fellenius method is much smaller than those obtained by other methods in the sand deposits, but is larger than those obtained by other methods in homogeneous cohesive soil in which the undrained shear strength increases with depth. It is revealed from this study that the conventional concept of the shear strength of the improved ground has a fundamental defect. The result suggest that Fellenius method may generally overestimate the bearing capacity of improved ground of low area replacement ratio, but underestimate the bearing capacity of high area replacement ratio.

Van Impe and Madhav (1992) proposed a method of analysis to show the effect of dilatancy on the settlement response of the stone column reinforced ground. The method is based on the Van Impe - De Beer (1983) approach. The "unit cell" concept is used to analyse the end bearing stone columns installed in group to reinforce the soft ground and subjected to uniform load applied through a rigid platform. The densified stone column material is considered to be at the limit yield condition and hence dilating. Results obtained bring out the importance of incorporating the dilatancy effect on the prediction of settlement (a significant reduction in settlement) and the stresses on the stone column and the soil. Induced lateral stresses in the soil adjacent to the column are shown to be of the same order as the vertical stresses. The predicted results are also compared with the test results and the existing foundation models. It is suggested that the constrained deformation modulus of the soil treated by stone columns has to be evaluated in accordance with the stone column installation method, the relative column interdistance, and even their installation sequence. After installation of reasonably representative number of columns, the constrained deformation modulus could be even reconsidered for new measurements in between the stone columns. In such way the estimated improvement of the overall settlements also could be reevaluated, which may lead to a more economical final design.

A simple method of analysis is suggested by Alamgir, Miura and Madhav (1994) to determine the settlement response of granular column reinforced ground based on "equal strain" theory. The "unit cell" concept is employed to find out the solution of the problem. This approach incorporates the radial deformation of granular column as well as stress transfer from column to soil. Consideration is also given to the linear increase with depth of the deformation modulus of the soil. The column-soil system is divided into a number of uniformly loaded elements and the solution is obtained by applying compatibility between their vertical displacements at the and mid point of every elements. Results obtained bring out the importance of incorporating the radial deformations and stress transfer between the column and the soil. Comparing the results with Alamgir et al. (1993) in which radial deformation is neglected, it is observed that this method underpredicts the settlement reduction ratio. The results are also compared with some test results represented in the literature. The settlements and the stresses on granular column obtained from this approach agree reasonably well with the measurements.

Ranjan and Rao (1994) presented a method to estimate the ultimate capacity and a procedure to compute the settlement of the ground treated with granular piles based on the cavity expansion approach and the concept of equivalent coefficient of volume compressibility respectively. The method incorporates the contribution of load shared by the surrounding ground which improves the load carrying capacity significantly. The method of settlement analysis is versatile as it can accommodate the changing of subsoil conditions with depth. The stress-deformation behaviour of ground treated with granular piles is found to be elastic in the first initial part of the curve and fully plastic in the later part. However, a zone of elasto-plastic behaviour is introduced in between the elastic and plastic range, which is found to be more pronounced when a rigid RCC skirt is provided alround. The effective modular ratio and the area replacement ratio have been identified as the two important parameters for the predictions. The validity of the design analysis for predicting the safe bearing capacity and the settlement of improved ground has been demonstrated by utilizing these for a live structure (79m diameter and 13.5m height steel oil storage tank) founded on skirted granular piles in soft saturated deep clay deposits.

Madhav, Alamgir and Miura (1994) suggested a method of analysis to predict the ultimate capacity and the stiffness of reinforced granular column. A single granular column is considered in which reinforcement is provided in the form of layers at a given spacing in the top region to prevent bulging failure. The restraint offered to the granular material by the reinforcement, prevents the lateral deformation i.e. bulging and thus increase the ultimate capacity and the stiffness of column. The ultimate capacity of the granular column is evaluated by modifying the approach given by Hughes and Withers (1974). The stiffness of the reinforced granular column is evaluated using an approach proposed by Duncan and Chang (1970). Both the ultimate capacity and the stiffness of column increase with the increasing number of reinforcement layer and the frictional resistance at the interface of column material and reinforcement. The improvement also depends on the spacing of reinforcement layers and the depth of location of bulging failure. The predictions based on this approach agree well with small scale in-situ test results reported by Madhav (1982). The comparison indicates that the most important parameters affecting the accuracy of the proposed analysis is to identify the actual depth at which the maximum passive resistance in the surrounding soil is mobilized.

Bouassida and Hadhri (1995) proposed a method to evaluate the improvement of bearing capacity of soft purely cohesive soils reinforced by columns. Using the yield design theory, the extreme load of an isolated column imposed by rigid loaded foundation is determined for plane strain and for axi-symmetric. Considering the stresses on the soil and the column, two loading cases - undrained and drained, are presented. The influence of gravity has also been investigated. This parameter increases considerably the extreme load value in the plane strain analysis when the column material has a high friction angle. In the plane strain analysis, quasi-exact solutions have been derived for the undrained case. Acceptable bounds of the extreme loads have been established for the drained case. Under axi-symmetric analysis, bounds of the extreme load values have also been established for the undrained case. The minimum mechanical characteristics of the column material for ensuring soil reinforcement have been specified. For the case of purely cohesive soils and column materials, when considering a surface load surrounding the loaded foundation, all lower bounds (determined by the static approaches) are added by the surface load value. Quasi-exact values and the acceptable bounds for the extreme loads are compared with the result presented in the literature.

Alamgir et al. (1995) proposed a simple theoretical approach to predict the deformation behaviour of soft ground reinforced by columnar inclusions such as stone columns/granular piles, sand compactions piles, lime or cement columns, etc. The cylindrical columns, installed in a group and extended to undeformable bearing strata, are subjected to uniform flexible loading acting over the entire area. A particular displacement function is proposed to obtain the solution of the problem in a simple manner. The analysis is performed based on the elastic deformation properties of column and soil which ensures no slip at the column-soil interface. The interaction shear stresses between the column and the surrounding soil are considered to account for the stress transfer between the column and the soil. The solution is obtained by imposing compatibility between the column and the surrounding soil for each element of the column-soil system introducing a postulated displacement function. Numerical evaluations are made for a range of parameters to illustrate the influence of various parameters on the predictions. The results show that the effects of spacings and modular ratios are significant on the distribution of the shear stresses, the load sharing between the column and the surrounding soil and the settlements of the reinforced ground. However, the Poisson's ratio of soil has little influence on them. The proposed method is verified with finite element analysis using CRISP program (Britto and Gunn 1987). A reasonable agreement is obtained between the results predicted by both the methods, in predicting the distribution of shear stress at column-soil interface, variation of stress concentration ratio and the settlement profile of the treated ground.

2.6 Numerical Solutions

Balaam et al. (1977) employed both finite element and finite difference methods for the theoretical predictions of the magnitude and the rate of settlement of soft clays reinforced with granular piles installed over a large area in a regular pattern. The effects of geometric factors such as pile penetration, spacing and soil layer depth are investigated. The analysis is performed considering a regular pile-soil unit subjected to uniform vertical pressure such as might be imposed by a flexible raft foundation or an embankment. For settlement analysis finite element method is used which is applicable for taking account of elasto-plastic behaviour of the soil and of elasto-plastic and dilatant behaviour in the pile material. The finite difference method is used for the analysis of the rate of settlement using the Diffusion theory which can handle both fully and partially penetrating columns. The results reveal that significant reduction in settlement occurs for closely spaced and fully penetrating columns. The effectiveness of granular piles in increasing the rate of settlement is increased dramatically by simultaneous reduction of pile spacing and increase of pile penetration. The results also reveal that the elastic analysis and the "Diffusion Theory" can provide predictions with sufficient accuracy of the settlement and the time rate of settlement, respectively, thus avoiding lengthy calculations required for elasto-plastic and Biot's three dimensional consolidation theories. Estimates of the optimal spacing, diameter and degree of penetration of piles can readily be made from the results presented.

Schweiger and Pande (1986) proposed a numerical analysis by finite element method to evaluate the settlement and failure load of rafts resting on stone column reinforced soft clays. The influence of stone columns is assumed to be uniformly and homogeneously distributed throughout the reinforced region. It is also assumed that both column and surrounding soil undergo the same total strains i.e. no slip occurs at the column-soil interface. A constitutive model is presented for an equivalent material. It combines different elasto-plastic laws, namely, the critical state model for clay and the Mohr-Coulomb criterion for gravel. Continuity of radial stresses is ensured by an additional pseudo-yield criterion. The model is incorporated in a finite element code and the results for a circular footing placed over the reinforced ground are presented. The influence of dilatancy of columns is highlighted together with the differences in the behaviour for columns situated at the center and at the outer boundary of the footing. The results indicate a stiffer response if columns are allowed to dilate. Flexible as well as rigid foundations are considered. It is emphasized by the authors that the finite element mesh is independent of the column spacing leading to considerable advantages in carrying out the parametric studies.

Canetta and Nova (1989) presented a numerical solution using finite element method and homogenization technique for the analysis of soft ground improved by columnar inclusions. The homogenization is obtained by enforcing the fulfillment of equilibrium and compatibility at interfaces and by imposing that the second order work in the equivalent material is equal to the sum of the work done in soil and column material. Any kind of constitutive law can be dealt with. If the soil behaviour is assumed to be elastic and no lateral strain is allowed, the settlement reduction factor comes close to that of Aboshi et al. (1979). Analysis is performed to depict the influence of various design parameters. Investigation shows the efficiency of vibroflotation method in reducing settlements depends on the state of stress that is generated after the ground treatment. It is suggested by the authors that the further experimental research is needed on the topic since it is apparent that the state of stress after treatment may have a remarkable influence on computed settlements.

Poorooshasb et al. (1991) proposed a rigorous analysis of the behaviour of soft ground, reinforced by a group of end bearing gravel piles, which undergoes one dimensional consolidation due to the radial flow of water. The study takes into account the consolidation process, the load transfer between the two components of the system and the resulting settlements which are experienced simultaneously during the loading history. The soft soil is treated as a homogeneous isotropic linear elastic solid while the gravel of the pile is considered to be an elastic strain hardening plastic material following the non-associated flow rule proposed by Poorooshasb et al. (1966 & 1967). Darcy's permeability law is used to account for the expulsion of the water during the loading process. The solution is obtained by solving the governing equations using the finite difference technique. Results are presented for a typical example to show the distribution of pore water pressure with time, the settlement of the system and the nature of sharing of load between the pile and the surrounding soil. The paper concludes with the comments that the rational design of gravel piles must take into account the performance of the system as a whole i.e. it must consider such processes as consolidation, dilatation, settlement and load sharing which occurs simultaneously in any loading process.

Asaoka et al. (1994) presented finite element analysis to predict the undrained shear strength of clay improved with sand compaction piles (SCP). The displacement type sand piling method is considered in which a considerable amount of *set up* (Randolph & Worth 1979) of the undrained shear strength of the surrounding clay is anticipated. The procedure for analyzing the set-up problem is presented in two stages. The first stage is the soil-water coupled rigid plastic finite element method (RPFEM) which is employed for solving the undrained failure of clay due to pile driving, and the other a linear elastic consolidation computation which accounts for the decrease of void ratio of the clay after pile driving. The predictions from this simplified method are examined through (i) laboratory experiments on a remoulded Kawasaki clay using a triaxial apparatus and (ii) the case record of in-situ loading test on soft clay improved with the SCP method. In the latter case, extensive improvement in the soft clay is found particularly at a large depth of the soft clay layer, where the set-up ratio is more than two. Although the present analysis is only an approximation, it seems to be sufficient for use in geotechnical engineering practice, due to its simplicity, in predicting the increased shear strength due to the set-up of clay. The analysis provides the facility to compute the distribution of excess pore water pressure within the clay with no change in the diameter of the sand compaction piles.

Madhav and Van Impe (1994) proposed a numerical solution by employing finite difference technique to predict the effect of over laying gravel bed on the settlement response of the soft ground reinforced by stone columns. The Pasternak type model concept, developed by Madhav and Poorooshasb (1987) for a stiff layer over soft soil, is extended here to analyze the gravel bed over the stone columns reinforced soil. The proposed analysis is based on the assumption that (i) the gravel bed is incompressible and can distort only by shear; and (ii) the soil and the stone column deform in a linear stress-strain behaviour without slip at their interface. Numerical evaluations are made for a range of parameters to illustrate the effect of different parameters on the predicted behaviour. The variation of settlements with distance in a unit cell as shown to be dependent on the shear stiffness (Product of shear modulus and the thickness) of the gravel bed, the relative stiffness of the stone column to that of soft soil and the spacing of the stone columns. The load transfer of the stone column by the gravel bed also varies with the above specified parameters. It is observed that the gravel bed can act as flexible to rigid loading platform depending on its stiffness. A gravel bed on a stiff stone column is much more effective in reducing differential settlements. The predictions also reveal that the shear stiffness of the gravel bed is more effective at closer spacing of the stone columns.

2.7 Experimental Approaches

Hughes et al. (1975) conducted a field test on a single stone column to investigate its performance and also to verify the theory proposed by Hughes & Withers (1974) on a field scale. The column was constructed by vibro-replacement and, after the test, was excavated to check its dimensions. The cylindrical stone columns as installed were 10m long and 0.66m in diameter which was estimated on the basis of stone consumption. A standard site investigation supplemented by the Cambridge (Worth and Hughes 1973) and the Menard pressuremeter tests, provided the basic soil parameters. The column was tested by loading a concentric circular plate of 0.66m diameter-which proved to be marginally smaller than the top of the column. The column improved substantially the bearing capacity of the natural soil. The ultimate column load depends on the friction angle of the gravel used to form the column, the size of the column formed and the restraint of the clay on the gravel. The method proposed by Hughes & Withers (1974) for calculating the ultimate load apparently under predicts by a surprisingly large amount. It was also observed that the prediction is excellent if allowance is made for transfer of load from column to clay through side shear and correct column size. They commented that the accurate estimate of the column diameter is the major factor influencing the calculation of ultimate load and the settlement characteristics.

McKenna et al. (1975) reported the lack of effectiveness of stone columns constructed by vibro-replacement technique, in reducing the settlement of a high trial embankments built on soft alluvium. The alluvium was 27.5m thick, the columns were 0.90m in diameter and 11.3m long, and they were constructed on a triangular grid at 2.4m centers. The embankment was built to a height of 7.9m. The instrumentation records showed that the columns had no apparent effect on the performance of the embankment. The reasons of no improvement are, as they stated, the grading of the granular materials was too coarse to act as a filter, and as a result, the voids in the gravel backfill probably became filled with clay slurry which prevented them from acting as drains. In addition, the method of construction would probably have remoulded the adjacent soft clays and damaged the natural drainage paths, so nullifying any potential drainage provided by the stone columns. The backfill was so coarse that when the embankment load came on to the columns, the crushed stone forming each column was not restrained by the surrounding soft clay, and as the columns expanded, the soft clay squeezed into the voids.

Rao and Bhandari (1977) performed experimental investigation on single and group granular piles by skirting them at the top region to prevent the bulging and thus to increase the load carrying capacity. The rigid skirt provided at the top prevents lateral deformations and thus bulging. Therefore, bulging if at all possible, can occur below the depth of the skirt. From the results it is found that skirting the top of the piles up to a depth of 0.8m, prevented bulging of the granular piles and increased the load carrying capacity by about 1.5 times compared with that of its unskirted counterpart.

Terashi and Tanaka (1981) investigated the bearing capacity and consolidation settlement behaviour of soft ground improved by cement columns through model tests taking into account the real construction conditions. For the bearing capacity test, the relative stiffness of column and soil and the length to diameter ratio of columns are taken as 11 to 173 and 8 to 13, respectively. Both fully and partially penetrating columns were studied. The sizes of the tanks used for the tests were 3.5 x 9 x 4m and 0.5 x 1.5 x 1m. For consolidation test a large consolidometer ring of 0.3m diameter and 0.10m height, was used. It is observed that the maximum reaction of untreated part is approximately equal to the bearing capacity of the shallow foundation on clayey soil. Thus the existence of columns seems to give negligible effect on the reaction of untreated part for the present area replacement ratio. The measured time versus volumetric strain relation shows that the behaviour of composite ground is of creep nature at lower stress level and resembles to ordinary normally consolidated soil at higher stress level. The test results are predicted by simple analytical method and also by the three dimensional finite element analysis.

Madhav (1982) presented two alternative approaches to prevent bulging in the top region of granular piles either by providing reinforcement in between the granular materials or replacing the top granular material by the stiffer concrete plug. They prevent lateral strains and thus increase the vertical load carrying capacity of the piles. The results of small scale model tests on reinforced granular piles indicate that larger the number of reinforcement layers higher is the improvement in the load carrying capacity and the stiffness of the reinforced ground.

Reinforcement increased the load carrying capacity and the stiffness of the granular piles by about four times compared with its unreinforced counterparts. For the case of rigid plug, it was observed that if the top 15% to 30% of the length of pile is replaced, the load carrying capacity becomes 2-4 times compared with that of the granular piles without rigid plug.

Instrumented large scale laboratory tests were performed by Charles and Watts (1983) to assess the effectiveness of granular columns in reducing the vertical compression of soft clay. The tests modelled the situation in which a soft-clay layer reinforced with fully penetrating columns is subjected to a widespread and relatively rigid load. Five tests were carried out to assess the effect of different column diameters on vertical compression. A large floating ring oedometer of one meter diameter and 0.60 meter high, was used. The test specimens were formed with a central granular column surrounded by an annulus of clay, have been tested in it. Tests were performed with a clay remoulded at a moisture content of 19% and with a undrained shear strength of about 30 kPa. Uniformly graded gravel were used in forming the granular column. The angle of shearing resistance were measured as 47° to 53°. The length of column was 0.60m having the initial diameters of 0.045, 0.35, 0.455, 0.50 and 0.5772m. Both columns and clay were instrumented so that stresses and strains could be monitored as the samples were loaded. The test results demonstrated the complexity of the soil behaviour. It was found that the settlement reduction factor obtained using the approach of Balaam & Booker (1981) differs significantly with that of the test results. With a small diameter column the gravel was in a state of failure; dilation took place and the principal stress ratio was at, or close to, the peak value. With large diameter columns the behaviour of the gravel was quite different. There was a reduction in volume as the load was applied and the principal stress ratio was well below the peak value.

Kimura et al. (1985) conducted centrifuge tests to investigate the mechanical behaviour of clay improved by sand compaction piles under inclined loading. Kawasaki clay was normally consolidated in a centrifuge and the model sand piles were installed. Loading tests were carried by applying lateral force to a model caisson placed on improved soil for four different combinations of lateral load to vertical load. The improvement by sand compaction piles increased the bearing capacity of clay by about 200 to 700% and it was extremely effective in reducing the lateral displacement of caisson. The postmortem studies of improved soil revealed that sand compaction piles were sheared when they were subjected to inclined load. It was found that for the improvement area ratio of 70% the caisson tended to move horizontally, while for the ratio of 33 to 55%, the caisson tilted even at the early stages.

Mitchell and Huber (1985) reported the performance of vibro-replacement stone columns used to support a large waste water treatment plant founded on up to 15m of soft estuarine deposits. Column spacings ranged from a 1.2m x 1.5m pattern under the most heavily loaded areas, to a 2.1m x 2.1m pattern under the lightly loaded areas. Twenty-eight single column load tests were done during the installation of 6,500 stone columns to evaluate load-settlement behaviour. The installation of stone columns led to a reduction in settlements to about 30%-40% of the values to be expected on unimproved ground. Load test settlements calculated by the finite element method for the initial settlement conditions, using undrained clay properties and drained properties of sand and stone columns, are somewhat higher than the average settlements observed during actual field load tests conducted on similar stone column spacing patterns. However, the overall results obtained from the finite element analysis indicated reasonable agreement between the calculated and the observed settlements for the individual load tests. Settlement predictions using several other, more simplified methods gave values that agreed reasonably well with both the finite element predictions and the measured values. This lends support to the use of the simple methods in practice.

Bergado and Lam (1987) investigated the behaviour of granular piles on soft Bangkok clay with different densities and different proportions of gravel and sand. A total of 13 piles were installed with 0.30m diameter and 8.0m long using a non-displacement cased borehole method with 1.20m spacing in a triangular pattern. The completed diameter of the granular piles were 1.05 to 1.35 times the initial diameter of the hole and varied progressibly with depth. The piles were grouped into 5 categories. Group 1, 2 and 3 with 3 piles each, were constructed using the sand compacted at 20, 15 and 10 hammer blows per layer, respectively. Group 4 was made of gravel mixed with sand in the proportion of 1:0.30 by volume and group 5 was constructed with gravel; both groups consisted of two piles and each was compacted at 15 blows per layer. The soil properties were investigated by the field vane and the pressuremeter tests. Using the pressuremeter results and the method of Hughes et al. (1975), the predicted ultimate pile capacity and load-settlement curve agreed well with the experimental data. The ultimate capacity of each granular pile was determined by using full scale plate loading tests. It was found that the ultimate bearing capacity increases with the density of column and the pure gravel column indicated higher capacity than that of the mixed counterparts. The pile made of gravel with 15 blows per layer (Group 5) yielded the maximum ultimate pile capacity closely followed by the piles constructed out of sand with 20 blows per layer (Group 1). The deformed shape of the granular pile was found as of bulging type and the maximum bulge was observed to be at a depth of one pile diameter from the ground surface.

Juran and Guermazi (1988) conducted a laboratory study to investigate the effect of various parameters such as loading process and loading rate, the area replacement ratio, the group effect and the partial consolidation of the soft soil due to radial drainage through the column, on the settlement response of the soft foundation soil reinforced by compacted sand columns. Triaxial compression tests under different boundary conditions were performed on composite soil specimens made of annular silty soil samples with a central, compacted river sand column. These tests, performed in a specially modified triaxial cell, showed that the group effect, the area replacement ratio, and the consolidation of the soft soil have a significant effect on the vertical stress concentration on the column and on the settlement reduction of the foundation soil. In case of relatively high values of area replacement ratio, the group effect prevents the plastic yielding of the column and of the soft soil and, consequently, significantly decreases settlements. In case of relatively small values of area replacement ratio, effective mobilization of group effect requires larger radial strains of the column. The consolidation and partial drainage of the soil during loading have an important effect on the load-transfer mechanism and should be considered in the analysis and design procedures. The rate of generation and dissipation of excess pore-water pressure depends primarily upon the area replacement ratio and the ratio of the loading rate to the soil permeability. Analysis of the loading tests indicated that unit cell elastic solution provides a reasonable estimation of the reinforcing effect on the settlement response of soft foundation soils.

Madhav and Thiruselvan (1988) studied the effect of method of installation - cased and uncased holes, number of lifts and compactive energy per lift given to granular piles and pile spacings. The tests were performed in the field for single piles and in the laboratory for large groups. They concluded that the settlements are less in case of cased than those of uncased bore holes. It was also found that larger the compactive energy, more the number of lifts and closer the spacing, the better will be the response of granular piles.

Bergado et al. (1988) reported the performance of full scale load test on a test embankment constructed on soft Bangkok clay improved by granular piles. The test embankment had a first stage height of 2.4m (Sim 1986) and subsequently raised to a second stage height of 4.0m (Panichayatum 1987) after 345 days. The case borehole method and a hammer 1.6 kN of 0.60m falling height was used for the construction of piles. The friction angle and the compacted density of granular material varied from 39° to 45° and 17 to 18.1 kN/m^3 respectively. Piles having diameter of 0.30m and length 8.0m, fully penetrating the soft clay layer, were arranged in a triangular pattern with a spacing of 1.5m. The granular materials consists of whitish-gray, poorly graded crushed limestones with a maximum size of 20mm. A drainage blanket of 0.25m thick consisting of clean sand was laid on top of the compacted granular piles. It was observed that granular piles increased the bearing capacity, reduced the settlement and increased the stability. The comparative study indicated that the embankment on granular piles settled about 40% less than that constructed on vertical drains. These results indicated the effectiveness of granular piles over the vertical drains in improving the soft ground.

Honjo et al. (1991) reported the vertical and lateral displacements of a full scale test embankment constructed on soft Bangkok clay improved with deep mixing method-DMM (dry method) using cement powder. The embankment was 5m high and the 8m long cement columns were constructed in two different patterns such as wall type and column type. The columns were placed in rectangular grid of 2.0m x 2.8m while the walls were placed at a spacing of 4.2m. It was observed that the lateral movements and settlements of the wall type were less than that of pile type. Furthermore, the deformation patterns of the wall and pile types were observed as sliding and tilting respectively. The wall types were found to be more effective in reducing lateral and vertical deformations. The unconfined compressive strengths in the laboratory were up to 20 times the untreated values for 28 days curing with cement content of 100 kg per cubic meter of clay. The field strength was found to be one-half of the corresponding laboratory test results.

Al-Refeai (1992) conducted triaxial shear tests to study the behaviour of soft soils strengthened with fibre reinforced sand column. Test results indicate that 2% fibre content by weight effectively increases the stress resistance properties of sand without any reduction in the soil density, permeability and ductility. The fibre reinforced sand layer in the sand columns should be placed at a depth of about one column diameter for it to be most economic and effective.

DeStenphen et al. (1992) reported about the use of floating stone columns to support a processing center at the solem/Hope Creek Nuclear Power Station, located along the Delaware River in Southern New Jersey, U.S.A. This foundation system was proposed as an alternative to more costly deep driven pile foundations typically used for other structures at the station. The stone column foundation was achieved by installing short vertical columns of compacted stone into the deep hydraulic fill that was placed to reclaim the power station area known as Artificial Island. The dry method of stone column installation was used, but with a bottom feed system for deliverying the stone to the bottom of the column. The columns had the same length of 5.5m but the diameter varied from 1.1 to 1.3m as estimated from the volume of consumption of stone. The columns were installed in square grid and the area replacement ratio was estimated as 0.10. The total foundation settlement were predicted to be about 0.152m considering both area loadings by floor slab/new fill and the column loads. The actual settlements were recorded at about 0.062m over a period of 2.5 years. Half the settlement occured under a preload perior to construction. Floating stone column can be an acceptable foundation support method for moderately loaded buildings in deep compressible soils. The dry method of stone column construction has proven to be successful even in soft soils with a high water table by use of a bottom feed system.

Leung and Tan (1993) implemented laboratory investigation to examine the load distribution and consolidation characteristics of a composite soil made up of a soft clay reinforced by a sand column. The marine clay used has a composition of 15% sand, 40% silt and 45% clay. The compacted sand used has a composition of 15% fine, 75% medium and 10% coarse and was compacted to a unit weight of 18 kN/m^3. The experiments were carried out in a circular steel tank of 1m diameter and 1m high, the column installed at the center rests on the rigid base and the loads were applied through a granular fill surcharge. Five tests were conducted with compacted sand columns having diameters of 100, 150, 200, 250 and 400mm, respectively, representing a range of replacement ratios from 1% to 16%. In general, the tests were conducted for a duration of about 30 days. It was observed that the stress concentration ratio generally increased as consolidation of clay took place and reached a maximum value at the end of the primary consolidation. The maximum stress concentration ratio was found to increase approximately linearly with the replacement ratio of the sand column. Further tests revealed that the maximum stress concentration ratio appeared to be independent of the surcharge loading under working load conditions. However, the stiffness of the sand column played a significant role on the magnitude of stress concentration on the column. The findings revealed the deficiencies in the assumptions made in conventional design procedures of sand compaction piles and sand drains.

Ekström et al. (1994) presented results from full scale tests where the cement columns were constructed to improve soft clay used for the foundation of buildings. Cement columns were installed in two sites having column diameter of 0.80m, length 9.0m and 7.0m respectively and placed in a square grid with a spacing of 1.5m. Hereby 20% of soil volume was stabilized. The columns extended up to 1.5m below the ground surface. The upper 1.5m of soil consisted mainly of sand. The undrained shear strength of soil determined by vane shear test was approximated 30-60 kPa in the top clay layer and 60 kPa in the upper part of the lower clay layer and increases to about 100 kPa at 15m depth. The compressive strength of the columns varied from about 100 kPa to 260 kPa. The average strength of 38 samples was 930 kPa and the standard deviation 800 kPa. The load steps of 18 and 34 kPa acted for 50 and 110 days, respectively. The measurements revealed that the installation of cement columns reduced settlement of the natural ground significantly.

Pan et al. (1994) carried out the laboratory model and field loading tests on single cement column and columns in group. The finite element method was employed to study the engineering behaviour of the soft clay ground improved with cement columns. The results from both the laboratory model tests and the field loading tests indicated that the load distribution between the cement columns and the surrounding soil are affected by the area replacement ratio, the length and the stiffness of cement columns as well as the stress level. The smaller the spacing and higher the cement content, the greater the stress concentration ratio will be. The results from FEM showed that the settlement of the composite ground decreases with the increase of the length, the stiffness and the relative area of the cement columns. However, there exists optimal values for both the length and the stiffness of cement columns from economical point of view.

Miura and Madhav (1994) reported the use of cement column as an design alternative of timber piles to improve soft sensitive Ariake clay. To construct the column, the dry jet mixing (DJM) method was used in which the in-situ soil is mixed with cement introduced in dry powder form. The partially penetrated DJM piles were 1.0m in diameter and 7.5m long. They were spaced at 1.6m in square arrangement. The improvement ratio defined as the area covered by the DJM piles to that of the treated ground, was around 30%.

2.8 Conclusions

A critical review of available literature pertaining to the improvement of soft ground by columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., is presented in the previous sections. The chronological development of analytical methods, numerical solutions and the experimental works after the beginning of modern applications of columnar inclusions as a ground improvement technique till to the recent past, have been discussed briefly. From the results of a large number of laboratory and field tests conducted till very recently and also from the results presented through several analytical and numerical studies of the behaviour of soft ground reinforced by columnar inclusions, it is observed that this ground improvement technique is successful in (i) improving stability of both embankments and natural slopes, (ii) increasing bearing capacity, (iii) reducing total and differential settlements, (iv) reducing the liquefaction potential of sands and (v) increasing the time rate of settlement.

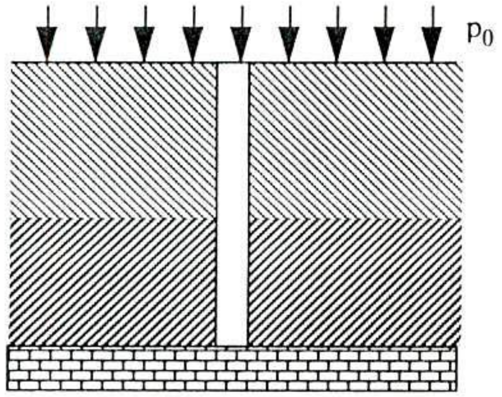
The literature review reveals that the practicing engineers still use the simple analytical method based on the 'equal strain' theory that horizontal sections of the treated ground remain horizontal during the course of settlement. This consideration does not take into account the actual loading conditions i.e. the reinforced ground may be subjected to either rigid or flexible loading. In most cases it violates the actual situation and keeps the analysis in a stage where the interaction between column and surrounding soil along the depth does not come into the picture. Stress concentration ratio, a very important concept, which accounts for much of the beneficial effect of improving marginal ground with columnar inclusions, is considered to have a single value for a particular problem. This consideration may lead to considerable errors in predictions as this ratio varies with a number of variables including the relative stiffness between the two materials, spacing and length diameter ratio of columns and the characteristics of the granular fill placed over the reinforced ground. Until now, the influence of granular fill placed over the reinforced ground is not considered in the design by the practicing engineers. It is also observed that it is possible to categorized the various types of columnar inclusions i.e. stone columns/granular piles, sand compaction piles, lime or cement columns, etc., as a single group from the standpoint of foundation analysis with some idealizations in order to develop a unified approach. From the foregoing discussions, it can be recognized that there is a need to develop a general foundation model that can be used by the practicing engineers with a high degree of accuracy to estimate the response of the soft ground reinforced by columnar inclusions considering most of the factors governing the behaviour under specified field situations.

CHAPTER THREE

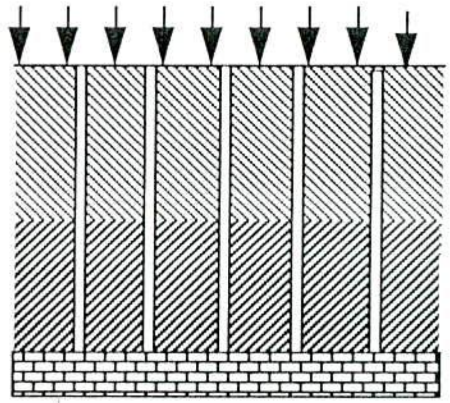
DEVELOPMENT OF THE METHOD OF ANALYSIS

3.1 General

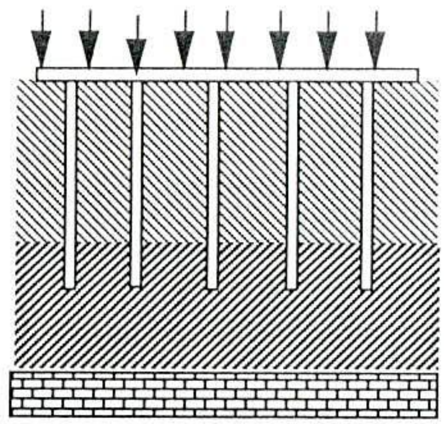
A general method of analysis to solve an important class of problems encountered in the field of geotechnical engineering is developed in this chapter. The analysis is formulated from the fundamental equation of equilibrium. It leads to develop a simple integro-differential equation to characterize the overall behaviour of the system. The behaviour of rigid columns (e.g. concrete/timber piles, lime/cement columns), deformable columns (e.g. stone columns/ granular piles, sand compaction piles), pile groups, pile-raft foundations are the type of situations that can be formulated by the proposed method of analysis. These foundation systems are being idealized and the governing equations in conjunction with the appropriate boundary conditions are developed. A numerical scheme using the finite difference method is proposed to solve the governing equations with the aid of the relevant boundary conditions. A systematic process of trial is proposed to identify the possible slip and its magnitude developed along the column-soil interface. The numerical scheme can be used for floating and end bearing columns and for the situation of soil stratification. The merit of this method is that data required for the analysis can be obtained from routine laboratory tests and the required computational facilities are available in any moderate size professional office or even in personal level. It can handle rather complicated situations such as certain type of inhomogeneity (e.g. radial inhomogeneity), different geometry of column and time-dependent behaviour of the reinforced ground.



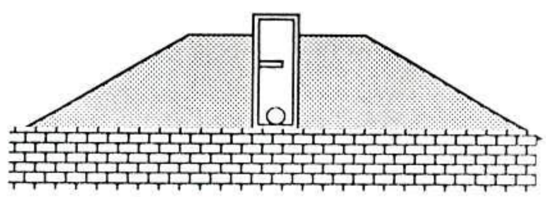
(a) Single column



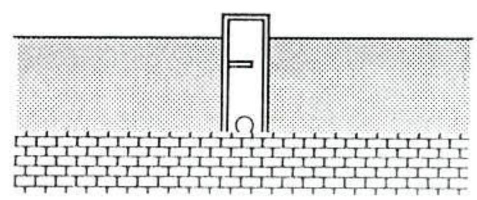
(b) Columns in group



(c) Pile-raft foundation



(d) Crusher unit of a mine
processing complex



(e) Idealization of (d)

Figure 3.1 Typical soil-structure systems to which the present analysis is applicable.

3.2 Idealization of the Problem

An important class of axi-symmetric problems generally encountered in the field of geotechnical engineering are idealized here from the standpoint of kinematics and foundation analysis. Figure 3.1 shows several types of structures to which the analysis outlined in this chapter is applicable. Figure 3.1(a) shows a single column (rigid or deformable) extended to bedrock embedded in a soft soil deposit. The stratified soil system is shown in this figure. Uniform load/surcharge (p_o) is applied over the entire area. The surcharge produces a negative skin friction along the column-soil interface leading to a downdrag force which can indeed be of a considerable magnitude. Figure 3.1(b) shows the end bearing columns (rigid or deformable) installed in groups to reinforce the soft ground. The columns are extended to bedrock. This composite ground is subjected to uniform load over the entire area which results the mobilization of negative skin frictions along the column-soil interface. In this case, the soil in between the columns is experienced arching effect. Now, it is necessary to quantify the extend of arching and provide the other data such as stress transfer between the column and soil, regarding the behaviour of reinforced ground required for a rational design. Figure 3.1(c) shows a so-called pile-raft foundation scheme. The load sharing between the pile and the surrounding soil due to their interaction is the main governing feature in this situation. The exact role of mat, placed over the pile, in the distribution of stresses among the components of the system is also required to identify. The mat placed on the top of pile can act as rigid or flexible loading platform based on its shear stiffness. This can be quantified easily with the aid of same method of formulations. Finally, in Fig.3.1(d), the crusher unit of a mine processing complex is shown. Here the downdrag on the vertical shaft must be evaluated in order to obtain a rational structural design. For this case, shown in Fig.3.1(d), the approximation applies if it is assumed that the side slopes are far enough from the shaft walls that an equivalent system, shown in Fig.3.1(e), may be used instead. These are the types of problems that can be handled by the analysis described in this chapter. The relevant units, for example an individual column surrounded by its zone of influence in the column reinforced ground shown in Fig.3.1(b), in all these systems enjoy one common condition: that is, the condition of radial symmetry.

3.3 Method of Solution of the System

In a paper entitled "Skin Friction on a Single Pile to Bedrock" by Poorooshasb and Bozozuk (1967) advocated the use of a single kinematically admissible displacement field whereby the soil surrounding the pile was assumed to move along concentric cylinders with their vertical axis along the axis of the pile. Stated otherwise it was suggested that for the type of problem under study it was accurate enough to assume that the lateral (radial) component of the displacement vector was sufficiently small to be ignored. The analysis followed the route proposed by Hill (1963), the central feature of which was to satisfy as many overall conditions of equilibrium as possible in order to achieve a solution of adequate accuracy. The paper did not attract much attention mainly because as, at that time, the evaluation of the basic equation was obtained in a closed form and necessarily dealt with a very simple soil model i.e. linear constitutive law for soil behaviour. With the advance and availability of computers it is now possible to obtain numerical solution of the governing equation and to handle rather complicated situations such as material nonlinearity as well as time dependency. The fundamental difference between the present work and those of Poulos and Davis (1975), Kuwabara and Poulos (1989), Chow et al. (1990) and Lim et al. (1993) is that while these authors start from a statement of equilibrium the present work starts from a statement of geometry (kinematics).

The motivation for undertaking the expansion and further elaboration of the ideas stated in the paper of Poorooshasb and Bozozuk (1967), referred to above, has been twofold. First, it is felt that although very sophisticated constitutive equations in conjunction even more sophisticated numerical techniques can be used to obtain "elegant" solutions to even simple cases such endeavors are hardly rewarding (or even relevant!) considering the complex nature of natural soil deposits and the uncertainties still involved in even the most advanced constitutive models. The present analysis makes use of nothing more than an e-log(p) curve and can handle stratified soil system as easily as uniform deposits. Second, the formulation of the problem, as presented in this chapter, uses a very elementary concept capable of simple physical interpretation. Furthermore, the numerical technique is straight forward and hence simple to grasp and to use. In this respect the study may merit some attention from an educational point of view and with concern to professional engineers.

3.4 Derivation of the Governing Equation

The derivation of the desired equation based on the ideas stated in the section 3.3, is developed in the following sections. A simple situation is considered for the development of the required governing equation. The kinematics of the problem, the relevant boundary conditions and the criterion for the consideration of possible slip along the column-soil interface are also stated.

3.4.1 Kinematics

Consider a single vertical column of circular cross section surrounded by a layer of soft ground and subjected to uniform loading over the entire area, shown in Fig.3.2(a). To analyze this system, the use of cylindrical coordinates is a natural choice. Let the three components of the displacement vector along the reference axes (r, θ, z) be denoted by w_r, w_θ and w_z. In view of the axi-symmetry of the problem w_θ is zero. Therefore, only the two displacement components, w_r, the radial and w_z, the axial, are present. The main kinematic constraints may now be stated as: for the type of problem under consideration the radial component, w_r is small and can be neglected. Thus, the only component of the displacement

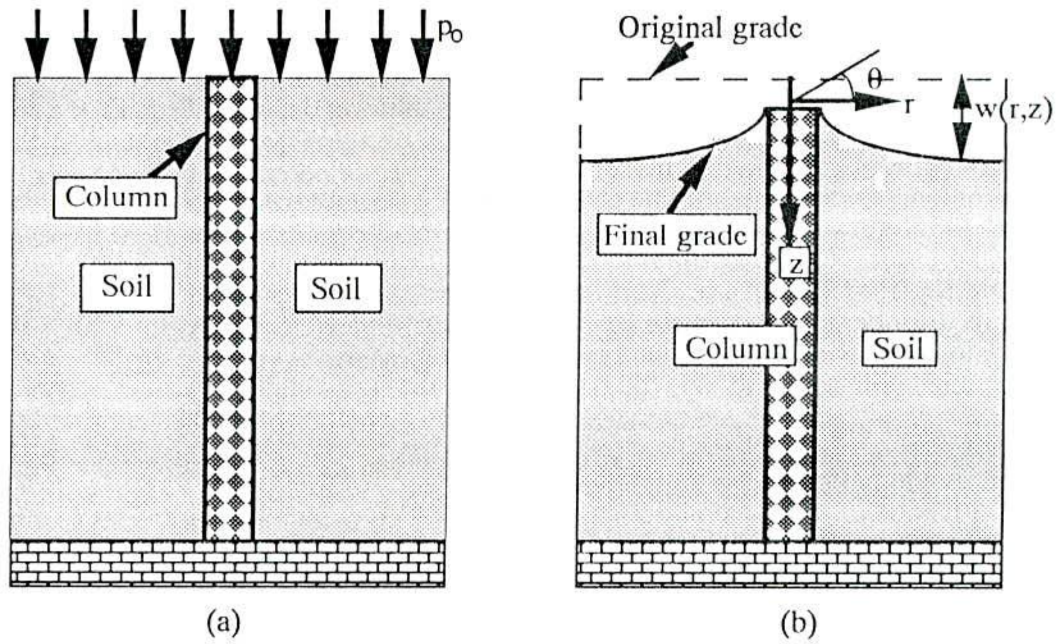


Figure 3.2 (a) Key diagram; and (b) Mode of displacement.

vector that is considered in the analysis is the vertical component w_z. For convenience in what follows the non-zero component of the displacement vector will be denoted by w in place of w_z. The component w is obviously a function of the space variables r and z i.e. $w=w(r,z)$ and in time dependent problem a function of time t, also i.e. $w=w(r,z,t)$. The rationality of this approximation in the cases stated in section 3.2 is most evident. The following phenomena can be realized from the pattern of deformation of the system, shown in Fig.3.2(b) as the mode of displacement. After the application of load on the system, as the consolidation proceeds, the surrounding soil layer moves downwards but not sideways.

3.4.2 Equilibrium

From the equilibrium of vertical forces acting on a typical soil element at a point (r,z), shown in Fig.3.3, in the absence of body forces, the following differential equation

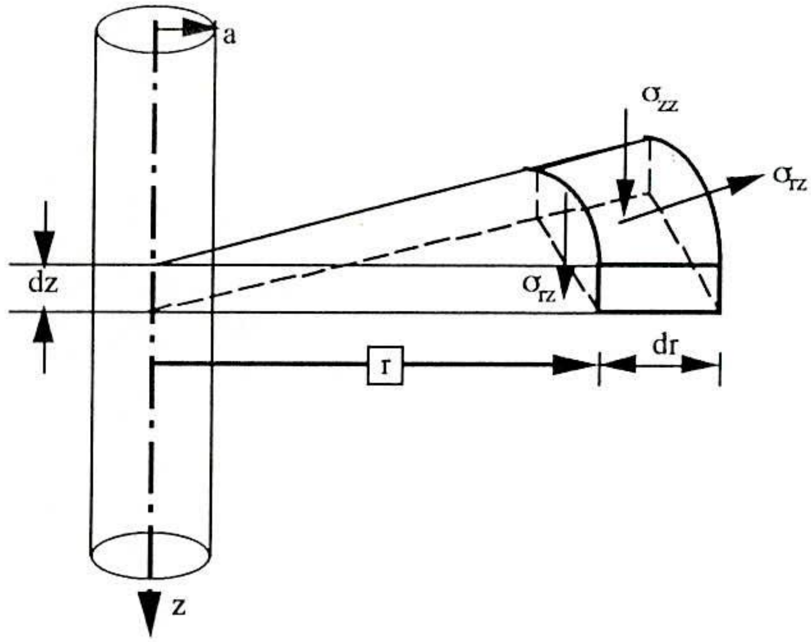


Figure 3.3 Equilibrium of a soil element around the column.

is obtained where the stress system is symmetric about z-axis. The derivation of this equation is presented in details in Appendix-I.

$$\frac{\partial \sigma_{zz}}{\partial z} + \frac{\partial \sigma_{rz}}{\partial r} + \frac{\sigma_{rz}}{r} = 0 \tag{3.1}$$

where σ_{zz} and σ_{rz} are the normal and the shearing stress components of the stress vector acting on a z plane. Integrating Eq. (3.1) between the limits 0 to z yields the value of σ_{zz}, as the following equation;

$$\sigma_{zz} = -\int_0^z \left[\frac{\partial \sigma_{rz}(r,\xi)}{\partial r} + \frac{\sigma_{rz}(r,\xi)}{r} \right] d\xi + p_o \tag{3.2}$$

where p_o is a constant equal to the value of the surcharge or applied load on the ground surface. Since the radial displacement component is assumed to be negligible, the shear strain γ_{rz}, can be expressed as

$$\gamma_{rz} = \frac{\partial w(r,z)}{\partial r} \tag{3.3}$$

where $w(r,z)$ is the vertical component of displacement and it is also a function of space variables r and z. Let the relationship between the stress component σ_{rz}, and the above strain component γ_{rz}, is expressed by the following stress-strain equation

$$\sigma_{rz} = G\,(r,z)\,\gamma_{rz} = G\,(r,z)\frac{\partial w(r,z)}{\partial r} \tag{3.4}$$

where the symbol $G(r,z)$ represents the shear modulus of soil. It is represented here by this form to emphasize that it is not to be treated as a constant but rather as a variable which may be a nonlinear function of space variables r and z. In general, it can also be a function of time and stress level also. This type of nonlinearity will not, however, be dealt with here. Substituting for σ_{rz} in Eq.(3.2) from the last equation one obtains the following integro-differential equation;

$$\sigma_{zz} = -\int_0^z \left[\frac{\partial}{\partial r} \left\{ G(r,\xi) \frac{\partial w(r,\xi)}{\partial r} \right\} + \frac{1}{r} \left\{ G(r,\xi) \frac{\partial w(r,\xi)}{\partial r} \right\} \right] d\xi + p_o \tag{3.5}$$

which represents the vertical component of the stress σ_{zz}, at a point (r,z) .

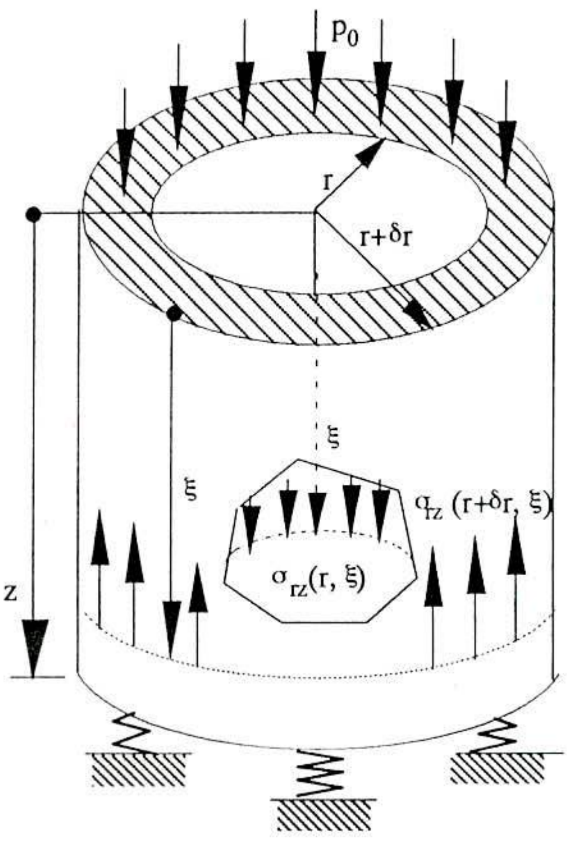


Figure 3.4 Physical interpretation of the basic equation.

At this juncture it is useful to give a physical explanation of Eq.(3.5). Figure 3.4 shows the free body diagram of a cylindrical element of the case shown in Fig3.1(a) with an inner radius r and an outer radius $(r + \delta r)$. A portion of the cylinder has been removed to reveal the situation inside the cylinder i.e. at radius r. The cylinder is assumed for the sake of argument, to be supported at the base by a set of imaginary springs having the deformation modulus of $E(r,z)$. The right hand side of Eq.(3.5) multified by the area of the ring denote the total unbalanced forces which are produced by the sum of the components $\sigma_{rz}(r, \xi)$, $\sigma_{rz}(r+\delta r, \xi)$ and p_o, and transmitted to the plane located at a depth z. These unbalanced forces give rise to the stress component σ_{zz} which, in turn, compresses the springs and produces a vertical strain of magnitude $[\sigma_{zz}/E(r,z)]$ and that can be expressed in the following form;

$$\frac{\partial w(r,z)}{\partial z} = \frac{\sigma_{zz}}{E(r,z)} \tag{3.6}$$

where the deformation modulus $E(r,z)$ is not constant but a variable which may be a nonlinear function of space variables r and z. From Eq.(3.6), one can also express the value of vertical stress component σ_{zz}, as;

$$\sigma_{zz} = E(r,z)\ \frac{\partial w(r,z)}{\partial z} \tag{3.7}$$

Now, combining Eqs.(3.5) and (3.7), one can obtain the following expression;

$$\frac{\partial w(r,z)}{\partial z} E(r,z) = -\int_0^z \left[\frac{\partial}{\partial r} \left\{ G(r,\xi) \frac{\partial w(r,\xi)}{\partial r} \right\} + \frac{1}{r} \left\{ G(r,\xi) \frac{\partial w(r,\xi)}{\partial r} \right\} \right] dz + p_o \tag{3.8}$$

After proper manipulation and the differentiation of the two terms of the right hand side of Eq.(3.8), with respect to r, one can derive the following expression;

$$\frac{\partial w(r,z)}{\partial z}+\frac{1}{E(r,z)}\int_0^z\left[\frac{\partial G\,(r,\xi)}{\partial r}\,\frac{\partial w(r,\xi)}{\partial r}+G\,(r,\xi)\,\frac{\partial^2 w(r,\xi)}{\partial r^2}+\frac{G\,(r,\xi)}{r}\,\frac{\partial w(r,\xi)}{\partial r}\right]d\xi=\frac{p_o}{E(r,z)} \tag{3.9}$$

which is the desired equation.

At this stage, it is appropriate to discuss the dependency of the two functions E and G on the radial coordinate, r. In time dependent problems (for example those that include radial drainage) these parameters change with time and are functions of the r coordinate also. As an example the elements nearest to a granular column, towards which the drainage is taking place gets "stronger" and acquire higher E and G values. The same type of statement holds true for the situations where these functions are stress level dependent. In problems where time and stress level do not play a role it may be assumed that E and G are not functions of r and hence $[\partial G/\partial r\,]=0$. Under the above stated conditions, one can reduce Eq.(3.9) as

$$\frac{\partial w(r,z)}{\partial z}+\frac{1}{E(z)}\int_0^z\left[G\,(\xi)\left\{\frac{\partial^2 w(r,\xi)}{\partial r^2}+\frac{1}{r}\,\frac{\partial w(r,\xi)}{\partial r}\right\}\right]d\xi=\frac{p_o}{E(z)} \tag{3.10}$$

which is the governing equation. This equation must be solved in conjunction with certain boundary conditions stated in the following section.

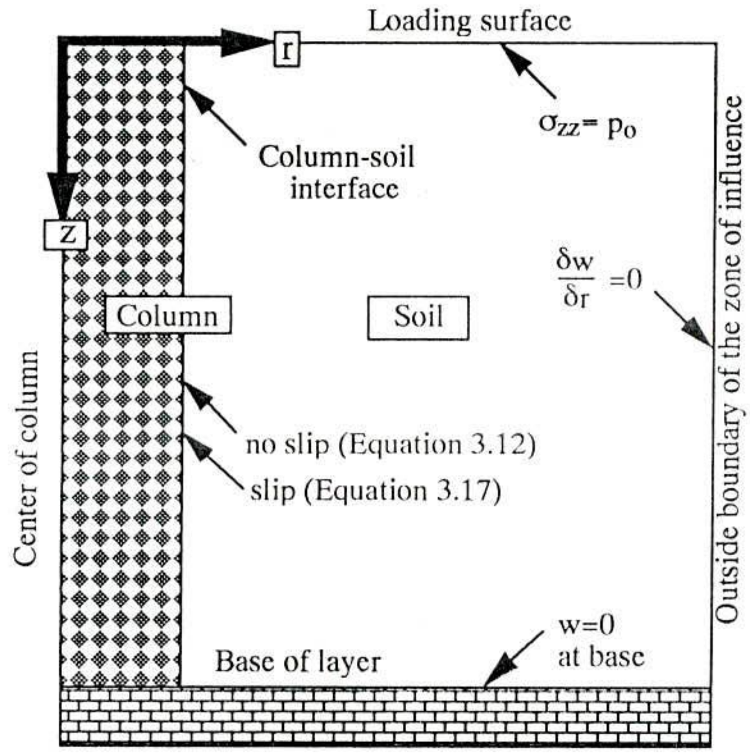


Figure 3.5 Boundary conditions.

3.4.3 Boundary conditions

The following boundary conditions, indicated diagrammatically in Fig.3.5, provide extra number of equations needed for a unique solution.

(i) Boundary condition at the grade level: This boundary condition has already been incorporated in the equations (the term involving the applied stress at grade level, p_o).

(ii) Boundary condition at the base: The base is considered as an undeformable bearing strata and at this level all the displacements are zero i.e. $w=0$.

(iii) Boundary condition at large radial distance from the column: At a "large" radial distance from the column, known as the radius of influence, it may be assumed that $[\partial w/\partial r]$=0. If the column in question belongs to a "group" of closely spaced columns, then the radius of influence is related to the column spacing. From the symmetry of load and geometry, there will be no shear stress at the radius of influence and hence the term $[\delta w/\delta r]$ must be equal to zero at this point.

(iv) Boundary condition at the column-soil interface: A downdrag force will be developed as a result of the surface loading. The magnitude of this downdrag force F_d is given by the equation;

$$F_d = 2\pi a \int_0^z G(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi \tag{3.11}$$

where a is the radius of column and $w(a,z)$ is the vertical displacement component at a radial distance $r=a$ and at depth z .The force F_d causes an additional axial strain in the column of magnitude $F_d/[A_c E_c(z)]$; where A_c is the cross sectional area of column and $E_c(z)$ is its deformation modulus which may vary with depth. The axial strain in the column at a depth z is equal to $[\partial w(a,z)/\partial z]$ assuming that no slip takes place between the column and the surrounding soil. Thus one can obtain the boundary condition at the column-soil interface in the following form;

$$\frac{\partial w(a,z)}{\partial z} = \frac{p_o}{E_c(z)} + \frac{2\pi a}{A_c E_c(z)} \int_0^z G(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi \tag{3.12}$$

which is the required equation to consider the equilibrium at the column-soil interface in case of no slip condition.

3.4.4 Consideration of slip at the interface

The boundary condition expressed above, in Eq.(3.12), is required for the analysis if no slip occurs at the column-soil interface. However, since real soils have a finite shear strength and the column-soil interface has finite strength, slip or local yield will occur when the shear stress reaches the yield strength. Due to this phenomena, one may encounter two possibilities at the column-soil interface. Along the lower portion of the column, slip is unlikely, the soil adheres to the column and no slip takes place. The upper portion of the soil, in all likelihood, will fail and lead to slip between the column and its surrounding soil. To account for this possible slip a criterion of failure must be introduced. One such criterion which is very commonly used in geotechnical engineering problems is that the shear stress that is mobilized along the column-soil interface can not exceed a limiting value, say τ_f, expressed below, and would flow at constant stress i.e. would behave as a plastic material.

$$\tau_f = K_0(\gamma' z + p_o)\tan\delta; \quad K_0 = 1 - \sin\phi; \quad \delta = 0.5\phi \ \ to \ \ \phi \tag{3.13}$$

where K_0 is the coefficient of earth pressure at rest; δ is the angle of friction between the column and soil; ϕ is the angle of friction of the soil and γ' is the effective unit weight of the soil. The range of δ stated here may cover all the possible situations, i.e., the slip between the very smooth to very rough column-soil interface.

In the light of what has just been discussed the soil element in the immediate vicinity of the column experiences a "stress-strain" response similar to that shown in Fig.3.6(a). The relationship also can be modified as shown in Fig.3.6(b). It differs from the

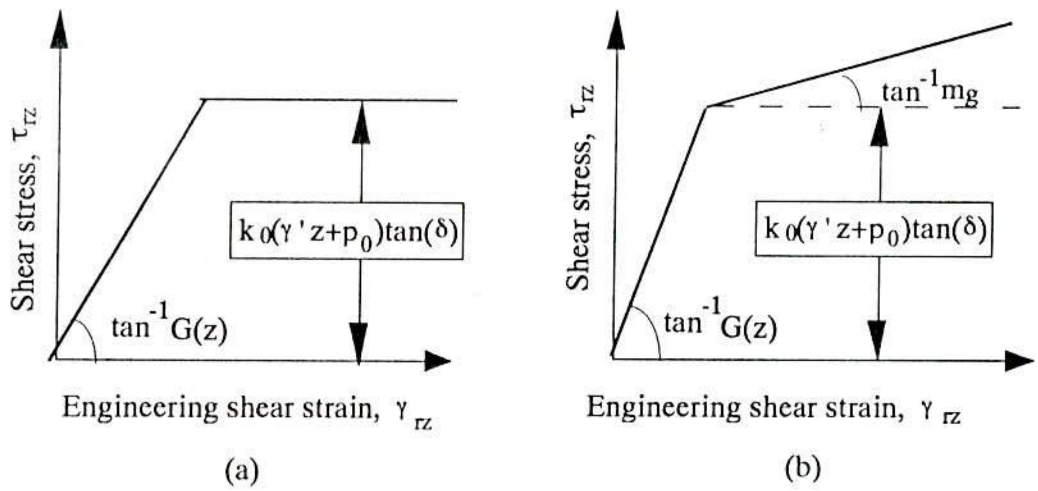


Figure 3.6 Stress-strain curve at the column-soil interface.

situation shown in Fig.3.6(a), in that the plastic portion of the curve has a positive slope m_g (for example m_g is of the order of $0.0001G$). This artifice ensures that at all times a one to one relation exists between the stress and the strain and greatly simplifies the computational efforts. This stress and strain relationship may now be stated as

$$\tau = \begin{cases} G(z)\dfrac{\partial w(a,z)}{\partial r} & \text{if} \quad \dfrac{\partial w(a,z)}{\partial r} < \dfrac{\tau_f}{G(z)} \\[2ex] m_g(z)\dfrac{\partial w(a,z)}{\partial r} + \tau_f\left(1 - \dfrac{m_g(z)}{G(z)}\right) & \text{if} \quad \dfrac{\partial w(a,z)}{\partial r} \geq \dfrac{\tau_f}{G(z)} \end{cases} \tag{3.14}$$

Eq.(3.11) may now be modified as to evaluate the magnitude of the force exerted on the column due to the consideration of possible slip at the column-soil interface;

$$F_d = 2\pi a \int_0^{z'} \tau_f(\xi) d\xi + \int_{z'}^{z} G(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi \tag{3.15}$$

where z' is the depth above which slip will take place. This depth is determined by a process of systematic evaluation which will be described later. Now substitute the value of τ_f from Eq.(3.14), the above equation can be modified as the following

$$F_d = 2\pi a \left[\int_0^{z'} \left\{ 1 - \frac{m_g(\xi)}{G(\xi)} \right\} \tau_f(\xi) d\xi + \int_0^{z'} m_g(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi + \int_{z'}^{z} G(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi \right] \tag{3.16}$$

With the aid of Eq.(3.16), Eq.(3.12) may now be modified to take into account the forces exerted by the soil on the column due to the possible slip at column-soil interface. Thus the boundary condition at the column-soil interface may be stated as;

$$\frac{\delta w(a,z)}{\delta z} = \frac{p_o}{E_c(z)} + \frac{2\pi a}{A_c E_c(z)} \left[\int_0^{z'} \tau_f \left\{ 1 - \frac{m_g(\xi)}{G(\xi)} \right\} d\xi + \int_0^{z'} m_g(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi + \int_{z'}^{z} G(\xi) \frac{\partial w(a,\xi)}{\partial r} d\xi \right] \tag{3.17}$$

which is the desired equation to solve the situation of slip at column-soil interface.

3.5 Development of the Numerical Scheme

The governing integro-differential equation in conjunction with the appropriate boundary conditions can not be solved analytically. A numerical solution must be employed. The development of such a numerical scheme using finite difference method with the procedure to determine slip zone is discussed in the following sections. The effect of the size of finite difference mesh on the predictions is also quantified.

3.5.1 Finite difference form of formulations

The development of the scheme will be demonstrated using the case shown in Fig.3.1(a) for the case of a single column extended to bedrock. The network, shown in Fig.3.7 in the *(r,z)* plane, is assumed to have *jmax* columns i.e. $j=1, 2, 3 \ldots jmax$, spaced

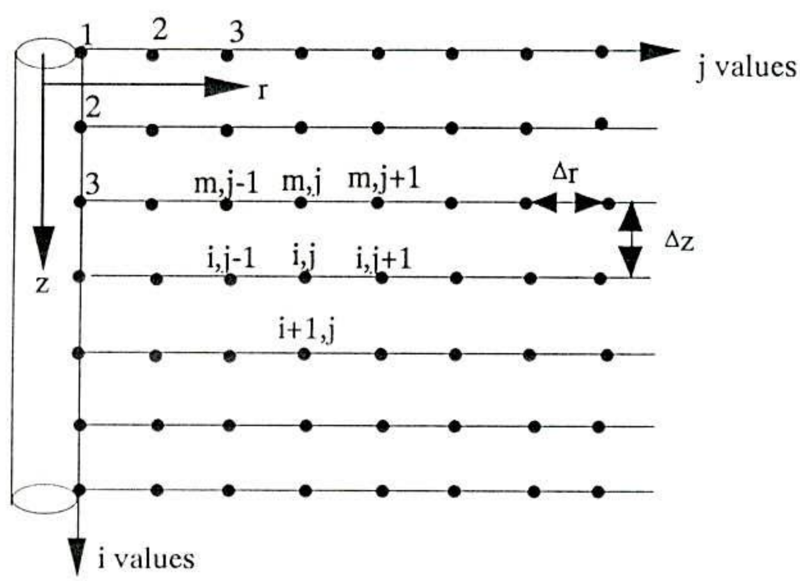


Figure 3.7 Grid system of the finite difference mesh.

regularly at Δr intervals, and *imax* rows i.e. $i=1, 2, 3 \ldots imax$, spaced regularly at Δz intervals. The origin of the r axis is the axis of the column and that of z is the top of the column, assumed to be flush with the soil surface. Here the value of j increases in the radial direction from the center of column while the value of i increases in the vertical direction from the top of soil surface. Considering a general nodal point *(i,j)* in the network, the general finite difference form of the governing equation, Eq.(3.10), may be written as;

$$w(i+1,j)-w(i,j)+\sum_{m=1}^{i}\alpha(m,j)\left[w(m,j-1)-2w(m,j)+w(m,j+1)\right]+ \\ \sum_{m=1}^{i}\beta(m,j)\left[w(m,j+1)-w(m,j-1)\right]=\frac{p_o}{E(i)}\Delta z \tag{3.18}$$

where $\alpha(m,j)=\dfrac{G\,(m-1)(\Delta z)^2}{E(i)(\Delta r)^2}$ and $\beta(m,j)=\dfrac{G\,(m-1)(\Delta z)^2}{E(i)(2r\Delta r)}$. The finite difference form of the boundary condition at the column-soil interface for no slip case i.e. Eq.(3.12), may be written as;

$$w(i+1,1)-w(i,1)+\sum_{m=1}^{i}\psi(m,1)\left[\frac{3}{2}w(m,1)-2w(m,2)+\frac{1}{2}w(m,3)\right]=\frac{\Delta z}{E_c(i)}p_o \tag{3.19}$$

where $\psi(m,1)=\dfrac{2\pi a(\Delta z)^2G(m-1)}{A_c(\Delta r)E_c(i)}$. Similarly, the boundary condition at the column-soil interface for taking account possible slip i.e. Eq.(3.17), can be written in finite difference form as the following.

$$w(i+1,1)-w(i,1)+\sum_{m=1}^{i'}\zeta(m,1)\left[\frac{3}{2}w(m,1)-2w(m,2)+\frac{1}{2}w(m,3)\right]+ \\ \sum_{m=1}^{i'}\eta(m,1)\tau_f(i)+\sum_{m=i'}^{i}\psi(m,1)\left[\frac{3}{2}w(m,1)-2w(m,2)+\frac{1}{2}w(m,3)\right]=\frac{\Delta z}{E_c(i)}p_o \tag{3.20}$$

where $\zeta(m,1) = \dfrac{2\pi a(\Delta z)^2 m_g(m-1)}{A_c(\Delta r)E_c(i)}$ and $\eta(m,1) = \dfrac{2\pi a(\Delta z)^2}{A_c E_c(i)}\left[1 - \dfrac{m_g(m-1)}{G(m-1)}\right]$. In finite difference form, the differential equation representing the boundary conditions (iii) i.e. at the outer boundary of the zone of influence, can be written as;

$$\frac{w\,(i,jmax+1) - w(i,jmax-1)}{2(\Delta r)} = 0 \quad i.e.\ \ w\,(i,jmax+1) = w(i,jmax-1) \tag{3.21}$$

where *jmax* indicates the node at the boundary of the zone of influence. It is the object of the analysis to evaluate the magnitude of $w(i,j)$ at each and every nodal point of the network which may be done by solving simultaneous equations. But it remains to express Eqs.(3.18) to (3.21) in a form suitable for coding. This is done by reducing the above equations and deriving the coefficients of a set of linear simultaneous equations for $w(1)$, $w(2)$, $w(3)$...........$w(nmax)$. Where the digit in the bracket indicate the nodal number, n, which is given for a general node (i,j) as

$$n = (i-1)\,jmax + j \tag{3.22}$$

The value of n varies as *1, 2, 3.....nmax*, where $nmax = (imax-1)*jmax + jmax$, is the maximum number of nodes i.e. total number of unknowns to be determined. Thus the nodal number assigned to the point (m,j) is expressed as

$$k = (m-1)\,jmax + j \tag{3.23}$$

and the node directly below the point (m,j) will have a nodal number of

$$s = m\,jmax + j \tag{3.24}$$

Thus Eq.(3.18) can be rewritten as

$$\begin{aligned} w(s) - w(n) + \sum_{k=1}^{i} \alpha(k)\left[w(k-1) - 2w(k) + w(k+1)\right] + \\ \sum_{k=1}^{i} \beta(k)\left[w(k-1) - w(k+1)\right] = \frac{p_o}{E(i)} \end{aligned} \tag{3.25}$$

Similarly, the Eqs.(3.19) to (3.21) can be rewritten in a similar form for suitable coding. The object of the above coding is to find out the values of constants $a(n,1)$, $a(n,2)$, ... $a(n,nmax+1)$, so that the simultaneous equations required for the solution of $w(1)$, $w(2)....w(nmax)$ can be obtained easily. One such equation corresponding to the node n, may be formed as

$$a(n,1)w(1) + a(n,2)w(2) + \ldots . a(n,n)w(n) + a(n,n+1)w(n+1) + \ldots . = a(n,nmax+1) \tag{3.26}$$

which is the desired form of the governing equation. In this way n number of simultaneous equations can be derived, where n is total number of nodes takes into consideration (i.e. existing nodes in the considered finite difference mesh). This set of simultaneous equations can be solved easily to obtain the values of $w(n)$ which, in turn, give the settlement profile of the treated ground, the mobilized shear stresses at the column-soil interface and the stress distribution between the components of the foundation system. The implementation of the above numerical scheme in a computer code is quite simple and straight forward. It is commented here, however, that the matrix of the coefficients of the unknowns (w) is rather full distinct from that of a finite element analysis wherein the matrix is banded.

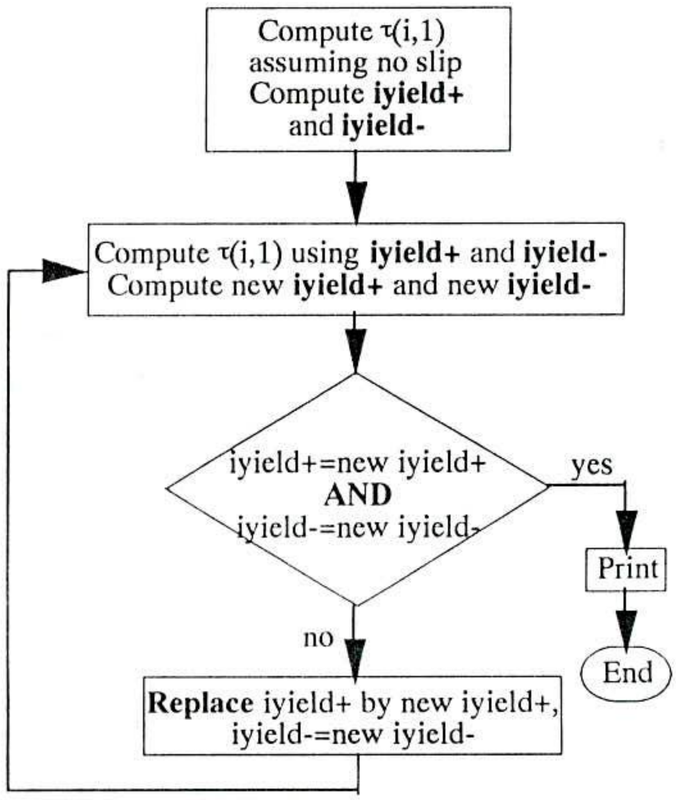


Figure 3.8 Flow chart for the trials determining the slip zones.

3.5.2 Procedure to determine slip zone

The possible slip at the column-soil interface and the depth of slip zone is not known a priory. So, it is obvious, a systematic process of evaluation and the identification of nodes at which slip occurred are required. In this section, a procedure is described that can be followed to determine the depth of slip zone between the column-soil interface. The scheme used in the present method of analysis involves the following steps, shown in Fig.3.8. First a run is made assuming no slip at all nodes and the largest nodal number at which the shear stress exceeds the permissible value, i_{yield+} , is noted. In some cases and in the case of partially penetrating column, the calculation may indicate a second slip point near the tip of the column where the shear stress exceeds the maximum value of negative shear (i.e. acting upwards). These values of i, i_{yield-}, are also noted. The second run of the program incorporates both i_{yield-} and i_{yield+} and the permissible value of shear stress at these levels. With these new constraints a check is made to identify the changes of the depth of slip zone, if any. Often there are and a third trial becomes necessary. Convergence to a unique situation, however, is fast and a fourth trial is seldom required.

3.5.3 Effect of finite difference mesh size

Based on the above stated formulations and the numerical scheme, a computer programme is written in Quick Basic for the prediction of results. The execution of this programme is performed using a personal computer, Macintosh LC 575. The results are obtained within a few minutes. Time required to get the response depends on the size of finite difference network. The mesh size is identified by the value of *imax* i.e. maximum number of vertical nodes and *jmax* i.e. maximum number of radial nodes. As the number of node increases, the time required for computation is also increased. Therefore, it is needed to establish the effect of mesh size. A typical example of column reinforced soft ground is considered for this checking. The columns are installed in group and extended up to the bed rock. A uniform flexible loading is acting over the entire area. The values of the parameters are taken as p_o/E_{so}=0.10, L_c/d_c=10, d_e/d_c=5.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. Here, p_o is the applied pressure, L_c and d_c are the length and diameter of column, respectively, d_e is the diameter of the zone of influence, E_{so} and m_s are the deformation modulus of soil at surface and its rate of increase with depth, respectively and ν_s is the Poisson's ratio of soil. The value of *imax* is considered to vary from 11 to 31 while the value *jmax* is varied from 6 to 11.

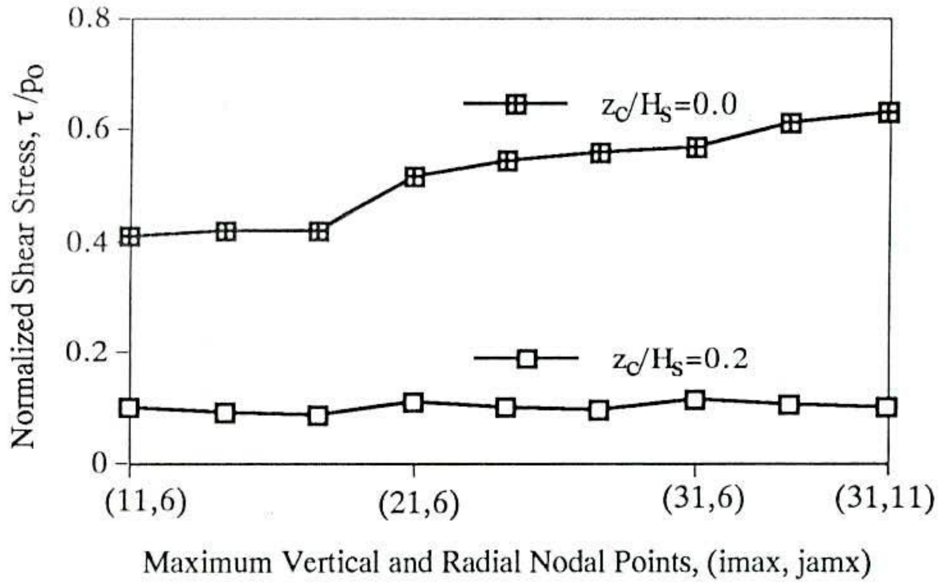


Figure 3.9 Effect of finite difference mesh size on the mobilized shear stress at column-soil interface.

Predictions are performed in nine combinations having different values of *imax* and *jmax* . The variation of predicted values of normalized shear stress at column-soil interface, τ/p_o, the normalized vertical stress in column, p_c/p_o, and normalized settlement of the reinforced ground at the surface, S_t/H_s, is presented in Figs.3.9, 3.10 and 3.11, respectively. In these presentations, τ, p_c, S_t, H_s and z_c are the shear stress, normal stress in column, settlement of composite ground, depth depth of soil layer and the depth measured from the surface, respectively. Figure 3.9 shows that at the top of column (i.e. at z_c/H_s=0.0), the value of τ/p_o increases with the increase of *imax* and *jmax* but at a depth z_c/H_s=0.20, the variation of

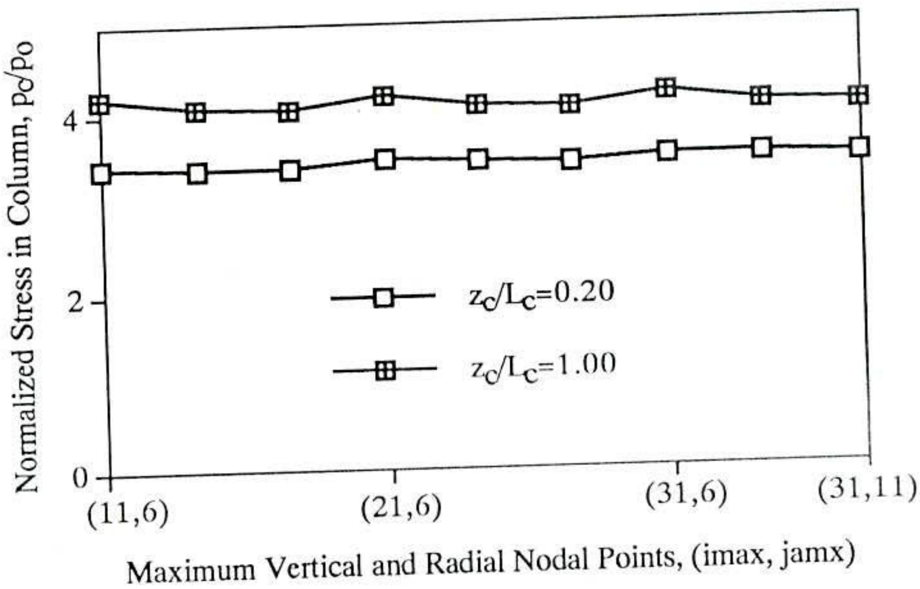


Figure 3.10 Effect of finite difference mesh size on the stress concentration in column at a depth z_c/L_c=0.20 and z_c/L_c=1.00.

the value of τ/p_o is negligible. The increase of τ/p_o is less for the increase of *imax* from 21 to 31 than that of from 11 to 21. The variation of stress concentration in column with the increase of finite difference mesh size is shown in Fig.3.10. The value of normalized vertical stress in column, p_c/p_o, with (*imax, jmax*) is plotted for the depth z_c/L_c=0.20 and 1.0. The variation of p_c/p_o is found to be insignificant with increase of mesh size from (*imax, jmax*)=(11,6) to (*imax, jmax*)=(31,11). Figure 3.11 shows that there is an increase of S_t/H_s for the increase of *imax* from 11 to 21 and *jmax* from 6 to 11 at $r/a=n$ i.e. at the boundary of the zone of influence. But the variation of the value of S_t/H_s is negligible at r/a =1.0 i.e. at the column-soil interface. It is also observed that the variation of settlement for the increase of *imax* from 21 to 31 is negligible. These predictions reveal that any value of imax, maximum

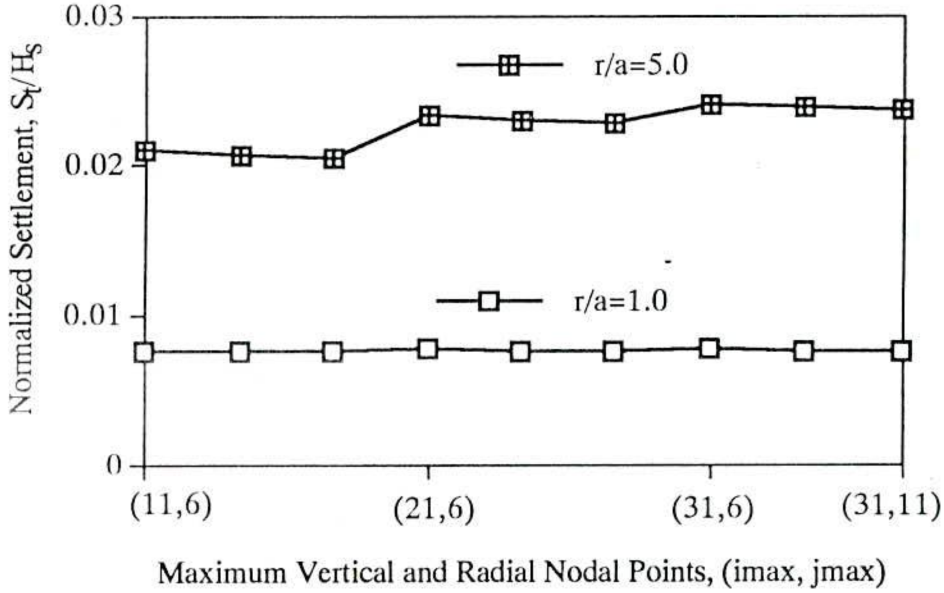


Figure 3.11 Effect of finite difference mesh size on the settlement of improved ground.

number of vertical nodes, from 21 to 31 and *jmax*, maximum number of horizontal nodes, from 9 to 11, provide an acceptable degree of accuracy in the predictions of the behaviour of column-reinforced soft ground. It is also noted here, only ten elements have been found to give the results of satisfactory accuracy in solutions for an incompressible pile by Poulos and Davis (1968), for a compressible pile by Mattes and Poulos (1969) and for granular column-reinforced ground by Alamgir et al. (1993).

3.6 Conclusions

A method of analysis is presented here to solve an important class of problems which the practicing engineers generally encounter in the field of geotechnical engineering. A numerical scheme is proposed for the solution of the governing equation. The method is simple both in concept and computations but versatile in application. The formulation of the problem as considered in the proposed method of analysis, uses a very elementary concept i.e. equilibrium of state of stress in an infinitesimal soil element. The numerical technique considered for the solution of governing equation in conjunction with the boundary conditions, is straight forward and provides the results with a high degree of accuracy even for a moderate finite difference mesh size. The implementation of the above numerical scheme in a computer code is very simple and requires few minutes to get the response with the kind of facilities generally available in a moderate size professional office or any academic institution. Minimum amount of data is required for the analysis and the conventional test facilities available in soil mechanics laboratory can provide the necessary information about the required design parameters.

This method of analysis can be treated as a fairly versatile tool. It is usable in various kinds of problems for both time-independent and time-dependent analyses. The formulations and the numerical scheme provide a fair guide line so that the proposed method can be used to solve the problem of soft ground reinforced by columnar inclusions. The columns either end bearing or floating can be analyzed by this method of analysis. The analysis incorporates the situations of possible slip or no slip at column-soil interface. It can also handle rather complicated situations such as certain type of inhomogeneity (e.g. radial inhomogeneity) and soil stratification. Though the minimum data and simple constitutive model of material is used rather complicated constitutive models of soil and the respective structures can also be accomplished easily in the proposed method to meet with the necessity and the required accuracy of predictions.

The main constraint which exists in the proposed method is clarified here. Although it satisfies all the possible boundary conditions, the main kinematic constraint is that the proposed method considers: the radial displacement component to be considerably small and hence can be neglected. It may be valid for some cases but in general may lead to some errors in predictions.

CHAPTER FOUR

SETTLEMENT RESPONSE

4.1 General

The reinforcement of soft ground by columnar inclusion such as stone columns/granular piles, lime or cement columns, etc., has become a common practice for ground improvement throughout the world. This reinforced ground is usually covered by a layer of granular fill to serve various purposes. The load is generally applied over the entire area and the loading conditions may vary from flexible (e.g. embankments) to rigid (e.g. raft foundation). Several methods for the determination of the supporting capacity and load-settlement behaviour of column-reinforced ground ranging from experience-based empirical estimates to sophisticated finite element analysis have been developed from the beginning of the modern phase of the use of columnar inclusions as a ground improvement technique. Their chronological development and limitations have been stated briefly in Chapter Two in the literature review. The existing design approaches do not take care of all the possible phenomenon as a whole that the system experiences simultaneously at any loading stage. Therefore, it is realized that there is a strong need to develop a foundation model that can be used successfully with minimum computational efforts to solve this type of problem considering all the possible mechanisms that the system may exhibit.

The development of an idealized foundation model to predict the behaviour of the above mentioned foundation system and its numerical predictions are described in this chapter. The formulations and the numerical scheme developed in Chapter Three are employed here. The proposed model incorporates nonlinearity of the material properties, the interaction as well as the stress transfer between the column and the surrounding soil along the depth, the compressibility of the granular fill and the possible slip at the column-soil interface. The stratification of the soil layer and the end bearing or floating columns are taken into consideration. Numerical evaluations are made to identify the effects of granular fill placed on the reinforced ground, distribution of load between the components of the system and the settlement profile of the improved ground. Predictions are also made for a wide range of parameters to illustrate the influence of different parameters such as (i) thickness and deformation modulus of granular fill, (ii) spacing and length to diameter ratio of columns, (iii) modulus of deformation and Poisson's ratio of soil, (iv) relative stiffness of column and soil, and (v) angle of friction between column and soil. The parametric study reveals that the proposed model can be used successfully to demonstrate the effect of the variation of different parameters on the overall response of the soft ground reinforced by a group of columnar inclusions.

4.2 Statement of the Problem

Consider a general case in which a soft ground reinforced by a group of columnar inclusions is covered by a layer of granular fill and is subjected to a uniform load over the entire area. The plan and section of this type of foundation system is shown in Fig.4.1(a) in which the columns are extended up to the bedrock and the soil strata is considered as homogeneous. The arrangement of columns may be found as square, triangular and hexagonal. In some instances the deposits of soft soil are so deep that it is not economically feasible to extend the column up to the full depth of soft soil layer. The soil deposit may also be stratified. The foundation systems in such cases are shown in Figs.4.1(b), (c) and (d) for end bearing

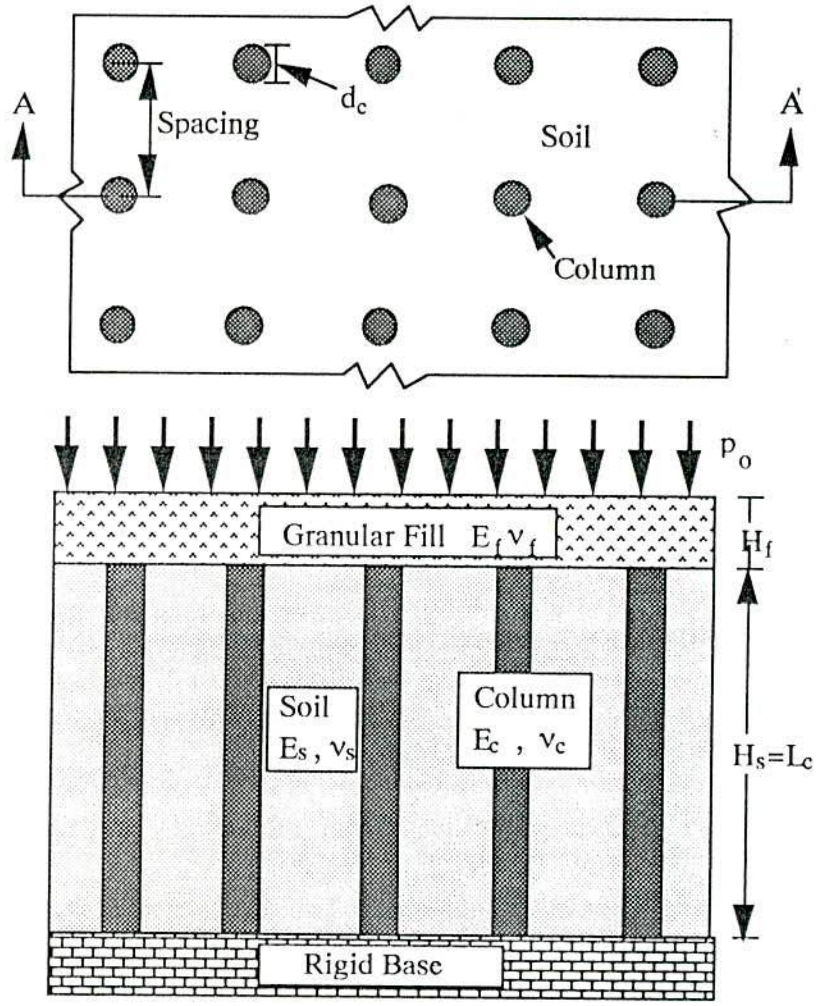


Figure 4.1(a) The system to be analyzed: Plan and section of nonstratified soft soil system reinforced by end bearing columnar inclusions.

columns in stratified soil, floating columns in nonstratified and stratified soil systems, respectively. In these figures, p_o is the uniform pressure applied over the reinforced ground, H_f is the thickness of the granular fill, L_c and $d_c(=2a)$ are the length and diameter of the cylindrical column, respectively, H_s is the total thickness of soil media. The granular fill, the column and the surrounding soil are characterized by the deformation moduli E_f, E_c and E_s that may vary with depth linearly or in any other form and the constant Poisson's ratios ν_f,

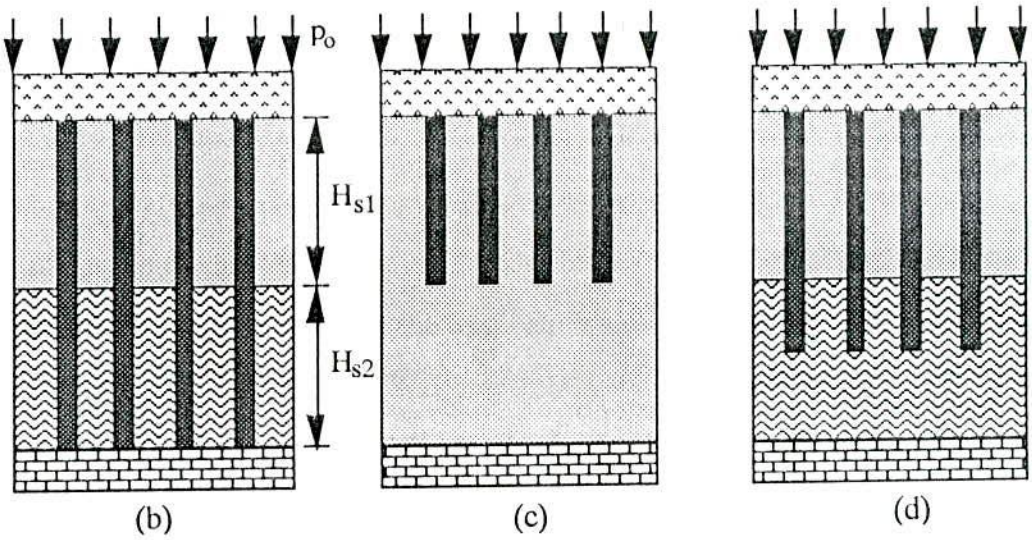


Figure 4.1 Foundation system: (b) End bearing columns in stratified; (c) Floating columns in nonstratified; and (d) Floating columns in stratified soil systems.

ν_c and ν_s, respectively. These properties are not influenced by the presence of columns and remain constant throughout the loading process. In case of stratified soil system, the digit 1 and appear in the subscript of the symbol of parameters to indicate the properties of the top and bottom soil layers, respectively. In stratified soil system, H_{s1} and H_{s2} represent the thicknesses of upper and lower soil layers, respectively.

4.3 Idealization of the Foundation System

Different types of columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., may exhibit different phenomenon in the post installation phase and during loading. This occurs due to the inherent differences in their physical and mechanical properties and their techniques of installation which depend on soil types, technical ability, efficiency and local conditions. But for foundation analysis these types

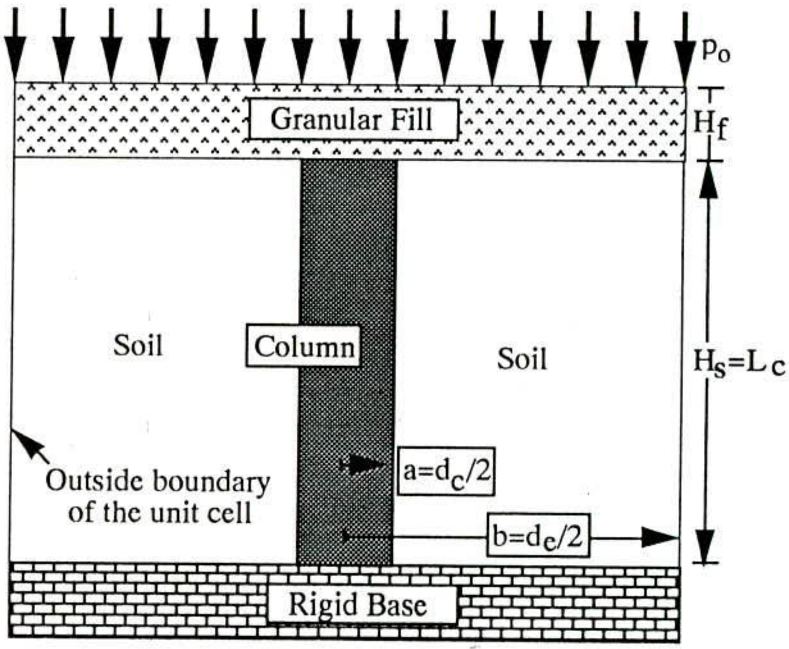


Figure 4.2 "Unit cell" idealization of the foundation system.

of columnar inclusions are categorized into a single foundation type i.e. a composite ground which consists of stiffer and stronger column and the surrounding softer soil media. This consideration allows the development of a unified model with some idealization for the analysis of column-reinforced soft ground.

The "unit cell" concept, which consists of the column and the surrounding soil within the zone of influence of the column, is employed here for the development of the proposed model. The 'unit cell' has the same area as the actual domain and its perimeter is shear free and undergoes no lateral displacement. It is recognized that the behaviour of the 'unit cell' adequately represents the behaviour of the soft ground reinforced by a group of columnar inclusions (Barksdale & Bachus 1983, Balaam & Booker 1985, Juran & Guermazi 1988, Enoki et al. 1991, Madhav & Van Impe 1994 and Alamgir et al. 1995). The consideration of

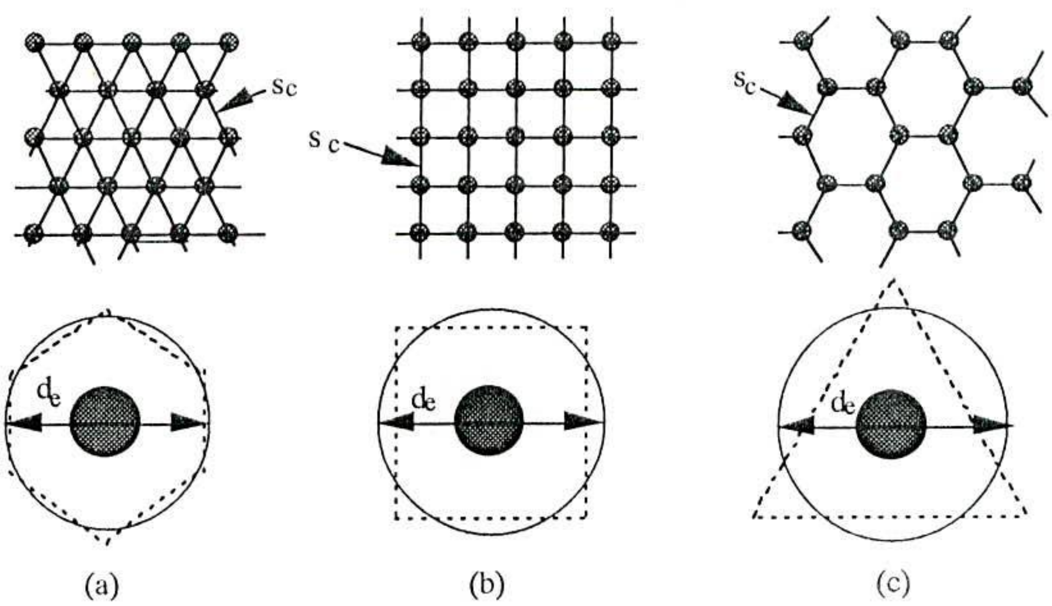


Figure 4.3 Various arrangements of columns and the zone of influence:
(a) Triangular; (b) Square; and (c) Hexagonal.

such a concept leads to a considerable simplification of the geometry of the problem, as shown in Fig.4.2 for the case stated in Fig.4.1(a). The three possible regular arrangements of columns generally practiced for ground improvement, are illustrated in Fig.4.3. The columns may lie on the vertices of an equilateral triangle, a square or a regular hexagon. The last case is of limited practical importance. In order to reduce the complexity of the analysis each influence zone is approximated by a circle of effective diameter, $d_e(=2b)$. Balaam and Booker (1981) relate the diameter, d_e, to the spacing of the columns, s_c, as

$$d_e = c_g s_c \tag{4.1}$$

where c_g is the geometry dependent constant equal to 1.05, 1.13 and 1.29 for triangular, square and hexagonal arrangement, respectively.

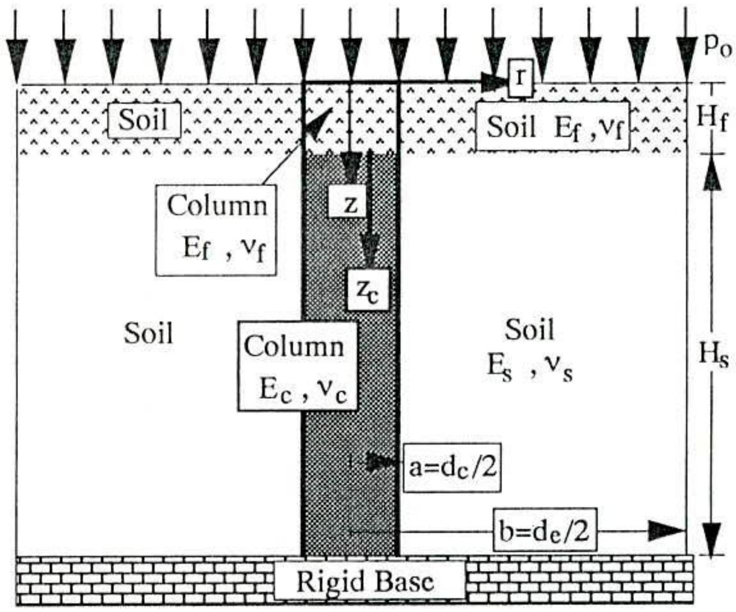


Figure 4.4 Foundation model for soft ground reinforced by columnar inclusions and covered by a layer of granular fill.

Further idealization is necessary to model the granular fill placed over the reinforced soft ground. The area of granular fill is divided into two regions: on the top of the column and the surrounding soil. This last idealization completes the proposed foundation model for the system and is shown in Fig.4.4 along with the coordinate axes considered in the derivation of the governing equation. In this model, the column-reinforced soft ground and the overlaying granular fill have been represented by the column and the surrounding soil. The column and the surrounding soil now consist of two layers instead of a single layer. In the top layer of the column and the surrounding soil, the dimensions and material properties are the same as those of granular fill but for the other regions they are the same as those of column and soil, respectively.

4.4 Response Function of the Model

As stated before, it is assumed here that the radial displacement component is very small and hence can be neglected. The rationality of this approximation in the cases of the above considered foundation system is evident. As the consolidation or the displacement of the system proceeds on or after the application of load, it is expected that the surrounding soil layer moves downwards but not sideways. Since the proposed foundation model enjoys the condition of radial symmetry and for the approximation just stated, the method of analysis and the numerical scheme developed in Chapter Three for the solution of an important class of problem in the field of geotechnical engineering, are employed here for the analysis and predictions. The developed numerical scheme is straight forward and can be used without any modification for the solution of the considered foundation system.

The analysis for the situation of no slip at column-soil interface, is performed by assigning the parameters $E(z)$ as the deformation moduli E_f, $E_c(z)$ and $E_s(z)$ for the granular fill, column and the surrounding soil, respectively. The deformation modulus of granular fill is considered as constant along the depth. But it is considered to increase linearly with depth for column (in some cases) and the surrounding soil: the variation is

$$E_s(z_c) = E_{so} + m_s z_c \tag{4.2}$$

where $E_s(z_c)$ is the deformation modulus of soil at a depth z_c measured from the top of soil layer, E_{so} is the deformation modulus at the top of soil layer, m_s is the rate of increase of modulus with depth. To represent the variation of the deformation modulus of column the subscript, s, of Eq.(4.2) is replaced by c. To account for possible slip between the column and the surrounding soil, the soil and the granular fill are treated as an elastic-perfectly plastic material and the criteria used for the evaluation of limiting shear stress is given as

$\tau_f = K_0(\gamma' z + p_o)\tan\delta$, where δ=0.5ϕ to ϕ. The parameter of soil/granular fill i.e. ϕ, is to be assigned. The value of shear modulus i.e. $G(z)$ is evaluated from the value of E(z) by the fundamental relationship: $G(z)=[E(z)/2(1+\nu)]$, where ν is the Poisson's ratio. It is necessary to evaluate deformation modulus, $E(z)$, and hence shear modulus, $G(z)$, using the test results from a simple odeometer test, in the form of compression index, C_c, and initial void ratio, e_o, and by including the effect of surcharge. To evaluate the parameter $E(z)$, a procedure is suggested in this study which may be followed. The details of the evaluation of $E(z)$ by this procedure are presented in the Appendix-II.

In the following sections, the predictions are made for a wide range of values of design parameters. The values of parameters are taken as p_o=100 kPa, E_{so}=1000 kPa, m_s=100 kPa, E_f=1000 to 100000 kPa, H_f=0.4 to 2.0 m, L_c=3.0 to 12.0 m, d_c=0.6 m, d_e=0.9 to 12.0 m, ν_f=0.30 and ν_s=0.25 to 0.49. The obtained results are presented in the nondimensional form. The above mentioned parameters are also expressed in the nondimensional form. The range of values of various nondimensional parameters are shown in the following Table.

Table 4.1 Ranges of Values of Nondimensional Parameters.

Sl. No.	Nondimensional Parameters	Ranges of Values
1	Applied load intensity, p_o/E_{so}	0.01
2	Length to diameter ratio of column, L_c/d_c	5.00 to 20.00
3	Spacing ratio of columns, $n=d_e/d_c$	1.50 to 20.00
4	Degree of penetration of columns, L_c/H_s	0.25 to 1.00
5	Relative thickness of granular fill, H_f/H_s	0.00 to 0.333
6	Relative moduli of granular fill and soil, E_f/E_{so}	1 to 100
7	Relative stiffness of column and soil, E_c/E_s	10 to 1000
8	Poisson's ratio of granular fill, ν_f	0.30
9	Poisson's ratio of soil, ν_s	0.25 to 0.49

4.5 Effect of Granular Fill on the Reinforced Ground

In the field, it is observed that the improvement of soft ground by columnar inclusions usually involves providing a layer of granular fill at the natural ground surface over the entire area. The purpose of this layer of granular fill may be outlined as (Madhav & Van Impe 1994): (i) to provide a working platform for the machinery, (ii) to level the site and increase the elevation, (iii) to prevent upheaval during column installation by vibro-displacement technique, (iv) to provide a facility for drainage of water, since the granular columns act as drains as well, and (v) to distribute the load from structures on to the soils and the columns and to minimize the differential settlements. The role of granular fill on the overall response of the soft ground reinforced by columnar inclusions, depends on its thickness and the modulus of deformation. Some of the recent available data regarding the thickness and the type of granular materials involved are shown in Table 4.2.

Table 4.2 Description of granular fill placed over the column-reinforced soft ground (after Madhav and Van Impe 1994).

Granular fill material	Thickness, H_f (m)	References
Sand and thin layer of gravel	2.0	Brons & De Kruijff (1985)
Gravel	0.30-1.0	Mitchell & Huber (1983)
Sand and gravel	0.70	Mitchell & Huber (1983)
Sand	2.0	Venmans (1990)
Sand, gravel or crushed stone	0.30-1.0	Barksdale & Bachus (1983)
Sand-medium to fine	0.90	Bachus & Barksdale (1984)
Coarse sand and crushed stone	1.0	Bhandari (1983)

Results are obtained for a range of parameters to illustrate the influence of deformation modulus and thickness of granular fill on the settlement response of soft ground reinforced by columnar inclusions. The evaluations are made for the column extended to bedrock through a homogenous soil layer considering no slip along the column-soil interface. Although the proposed model can be used for floating column, stratified soil and for possible slip at column-soil interface, the simple case is considered since the aim of this section is to quantify the influence of granular fill placed over the reinforced ground. The values of the parameters are p_o/E_{so}=0.10, L_c/H_s=1.0, L_c/d_c=10, d_e/d_c=5.0, E_c/E_{so}=50, m_s/E_{so}=0.10, ν_f=0.30 and ν_s=0.40. The variation of deformation modulus and thickness of granular fill and their influences on the response of soft ground reinforced by columnar inclusions, are discussed in the following sections.

4.5.1 Settlement of reinforced ground

The influence of thickness and deformation modulus of granular fill on the settlement response of soft ground reinforced by columnar inclusions is presented in Figs.4.5 to 4.10. The changes of settlement profiles for the variation of relative thickness of granular fill

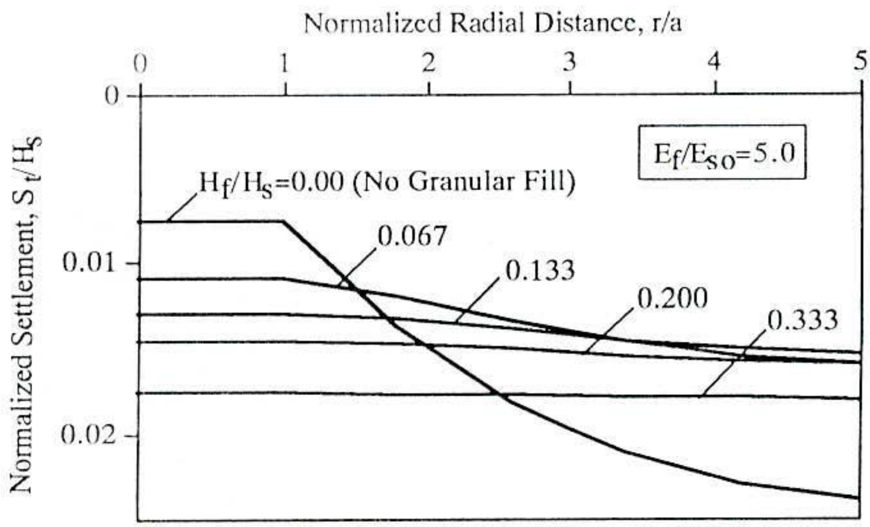


Figure 4.5 Effect of thickness of granular fill on the settlement profiles of soft ground reinforced by columnar inclusions for E_f/E_{so}=5.0.

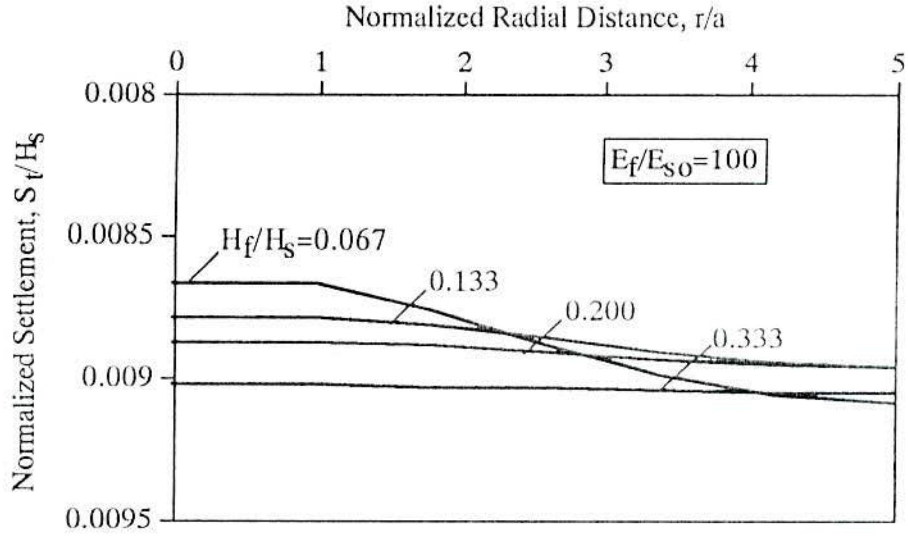


Figure 4.6 Effect of thickness of granular fill on the settlement profiles of soft ground reinforced by columnar inclusions for $E_f/E_{so}=100$.

i.e. H_f/H_s from 0.00 to 0.333 are presented in Figs.4.5 and 4.6 for very low and high values of relative moduli of granular fill i.e. E_f/E_{so}=5 and 100, respectively. In these figures, the variation of the normalized settlement of the reinforced ground, S_t/H_s, are plotted with normalized radial distance r/a, where S_t is the settlement of the reinforced ground. The settlement profile of the reinforced ground changes significantly with the placement of granular fill over it. In case of low relative moduli of granular fill i.e. E_f/E_{so}=5.0, the value of S_t/H_s increases in the near column region i.e. at r/a=0.0~1.5 but beyond this radial distance, it decreases up to the boundary of the zone of influence (i.e. $r/a=n$), shown in Fig.4.5. The value of S_t/H_s changes from 0.0076 to 0.0177 at r/a=0.0 and 0.024 to 0.0181 at r/a=5.0 for the value of H_f/H_s changing from 0.000 to 0.333. It can also be seen that the overall settlement of the reinforced ground tends to become uniform for higher value of H_f/H_s. The influence of thickness of granular fill for a high value of its relative moduli i.e. E_f/E_{so}=100, on the settlement response of composite ground is presented in Fig.4.6. The overall settlement decreases considerably and gives almost uniform response of settlement even for a moderate thickness of granular fill. The value of S_t/H_s is found as 0.0087 to 0.009 at r/a =0.0 and 0.0091 to 0.009 at r/a =5.0 for H_f/H_s=0.000 to 0.333 while it is found as 0.0076 and 0.024 for the no granular fill case. These figures show that the presence of a layer of granular fill effects the magnitude of settlement significantly. These results also reveal that the use of thick granular fill is not necessary to obtain the uniform settlement if the granular fill is having high value of deformation modulus.

The influence of relative moduli of granular fill over the settlement response of the reinforced ground is presented in Figs.4.7 and 4.8 for very low and high values of relative thicknesses of the granular fill i.e. H_f/H_s=0.067 and 0.333, respectively. The settlement profile of column reinforced ground for no granular fill situation is also plotted in these figures. In this evaluation, the relative moduli of the granular fill E_f/E_{so}, is considered to vary from 1 to 100.

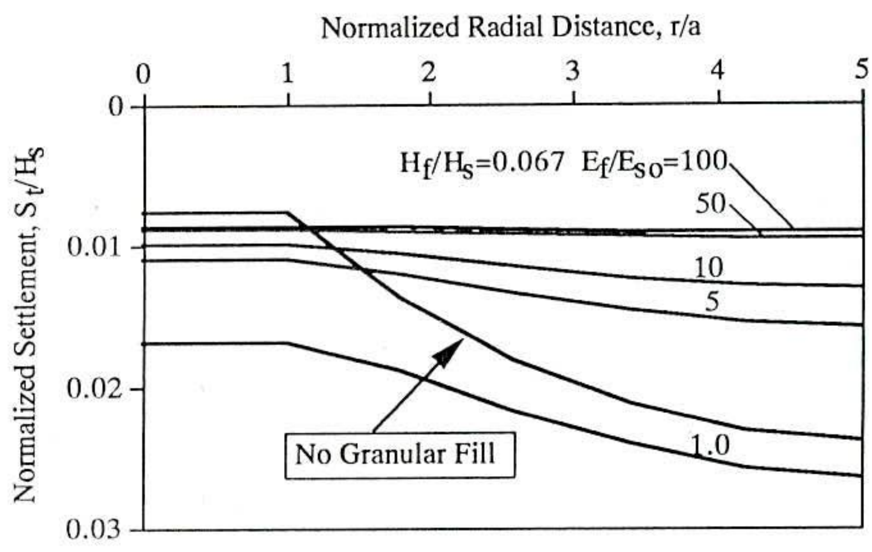


Figure 4.7 Effect of deformation modulus of granular fill on the settlement profiles of soft ground reinforced by columnar inclusions for H_f/H_s=0.067.

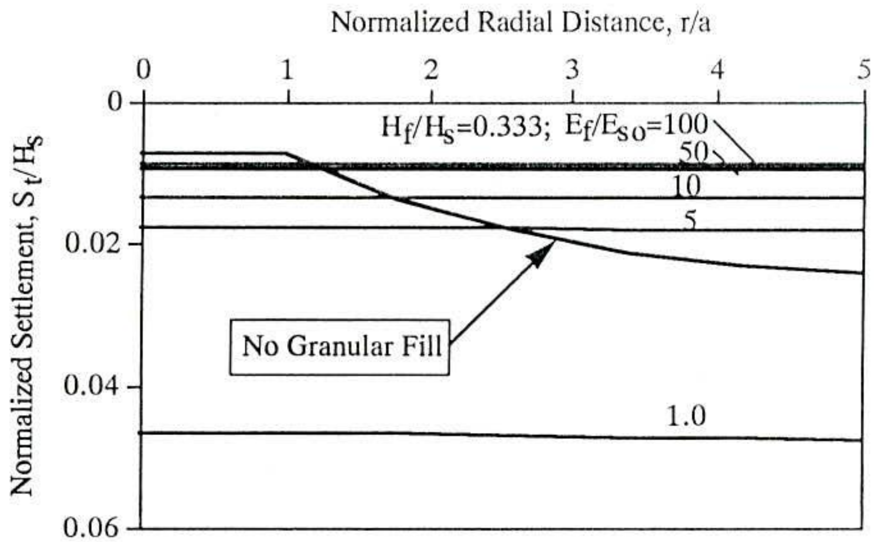


Figure 4.8 Effect of deformation modulus of granular fill on the settlement profiles of soft ground reinforced by columnar inclusions for H_f/H_s=0.333.

Settlement profiles of the reinforced ground are plotted in the S_t/H_s versus r/a diagrams. In all the cases, the settlement of the reinforced ground decreases with the increase of relative moduli of granular fill. From Fig.4.7, it can be seen that the settlement profile of the reinforced ground remains nonuniform as long as the value of E_f/E_{so} is less than 100. The value of S_t/H_s varies from 0.017 to 0.0087 at r/a=0.0 and 0.026 to 0.0091 at r/a=5.0 for increasing the value of E_f/E_{so} from 1 to 100. The result presented in Fig.4.8 reveals that the overall settlement on the reinforced ground becomes almost uniform at the values of H_f/H_s=0.333 for any value of E_f/E_{so} ranges from 1 to 100. But the value of S_t/H_s increases noticeably with the decreasing value of E_f/E_{so} . The value of S_t/H_s increases from 0.009 to 0.0468 at r/a=0.0 and 0.009 to 0.0476 at r/a=5.0 for decreasing the value of E_f/E_{so} from 100 to 1. The lower value of E_f/E_{so} produces higher overall settlement of the improved ground which is not desirable for the success of this type of soil improvement technique. Both figures show that granular fill with

E_f/E_{so}=1.0, produces higher settlement of reinforced ground than that of no granular fill situation. These findings reveal that the compressibility of the granular fill should not be neglected for the rational evaluation of its role over the settlement response of reinforced ground. Shukla (1995) also observed that the compressibility of the granular fill has an appreciable influence on the settlement response of the geosynthetic-reinforced granular fill-soft soil system as long as the stiffness of the granular fill is less than fifty times that of the soil.

The effectiveness of granular fill in minimizing the differential settlement of reinforced ground is shown in Figs.4.9 and 4.10 for the relative moduli of granular fill E_f/E_{so}=5 and 100 respectively. The relative thickness of granular fill is considered to vary from H_f/H_s=0.000 to 0.333 and the spacing of columns $n(=d_e/d_c)$ varies from 2.5 and10, respectively. In this presentation, S_t are predicted at the center of column i.e. at r/a=0 .0 and at the boundary of the zone of influence i.e. at $r/a=n$. Because the maximum differential settlement in reinforced ground is obtained when these two points are considered. From these

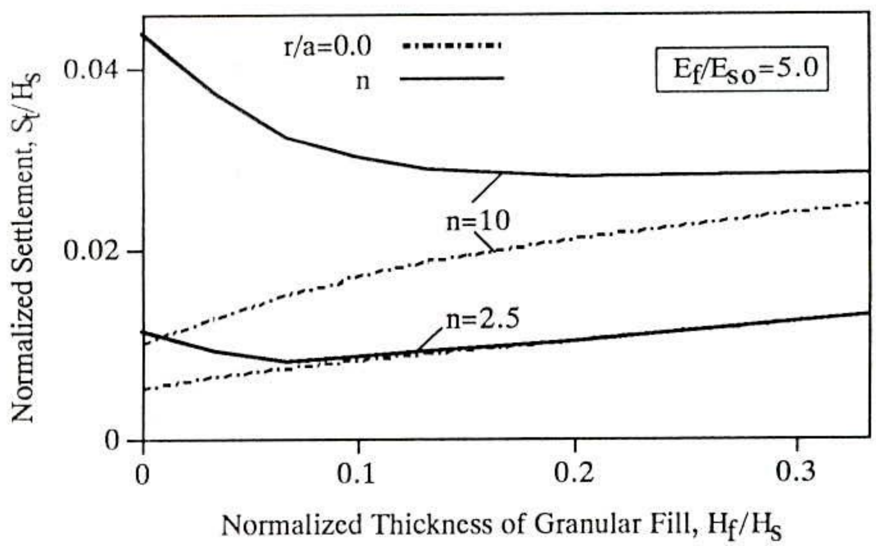


Figure 4.9 Influence of granular fill thickness in reducing the differential settlement of column-reinforced soft ground for E_f/E_{so}=5.0.

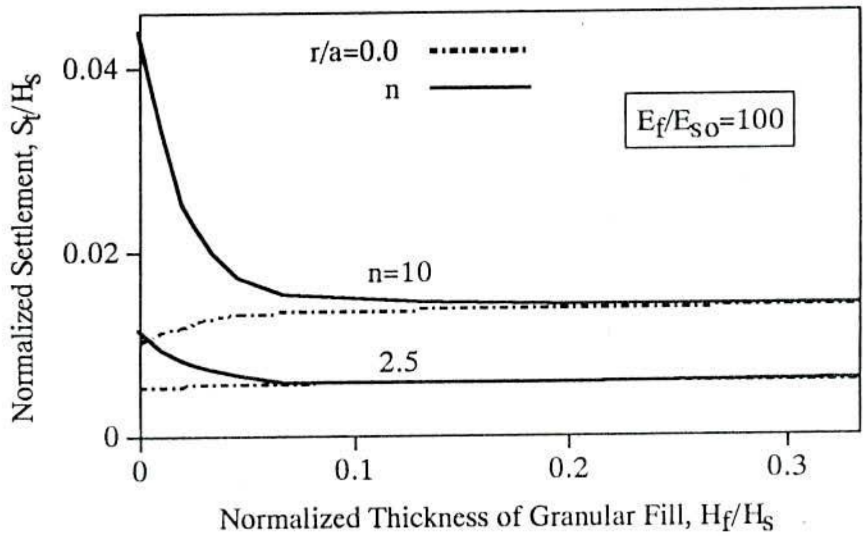


Figure 4.10 Influence of granular fill thickness in reducing the differential settlement of column-reinforced soft ground for E_f/E_{so}=100.

results, it is revealed that the differential settlement is large for H_f/H_s=0.00 i.e. for no granular fill, but it reduces with the presence of granular fill over the reinforced ground. Figure 4.9 shows that the differential settlement becomes almost zero at H_f/H_s=0.133 for n=2.5 while the settlement still remain nonuniform even at H_f/H_s=0.333 for n=10. In case of high relative moduli of granular fill i.e. E_f/E_{so}=100, shown in Fig.4.10, the differential settlement reduces considerably for any value of n even for low value of H_f/H_s. At H_f/H_s=0.075, the differential settlement is found almost zero for n=2.5 while in case of n=10, the value of H_f/H_s needs to be around 0.30 to get uniform settlement. This behaviour may be explained from arching effect of soil between the columns. Closer spacing of columns offer higher arching effect than that of for the higher spacing which, in turn, effects the effectiveness of granular fill. However, these results reveal that the granular fill is more effective when it has high deformation modulus and for the closer spacing of columns.

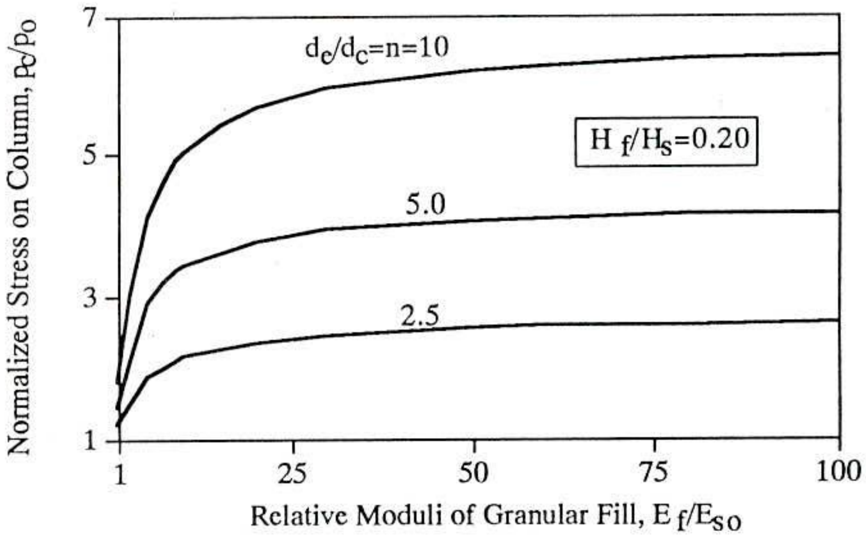


Figure 4.11 Variation of normal stress on the top of column with the relative moduli of granular fill for H_f/H_s=0.20.

4.5.2 Stress concentration in column

The effect of granular fill placed over the reinforced ground on the concentration of applied stress i.e. the stress taken by the relatively stiffer and stronger column p_c, is shown in Figs.4.11 and 4.12. Figure 4.11 shows the variation of magnitude of normalized stress p_c/p_o, at the top of column with the relative moduli of granular fill E_f/E_{so}=1 to 100 for H_f/H_s=0.20 and column spacings n =2.5, 5.0 and 10. The results indicate that the load taken by column increases nonlinearly with the increase of E_f/E_{so} and become almost constant at a value of E_f/E_{so}=50. The rate of increase of p_c/p_o is considerably higher at the range of E_f/E_{so}=1 to 20 than that of at E_f/E_{so}=20 to 100. The value of p_c/p_o increases from 1.0 at E_f/E_{so}=0, to 2.62, 4.19 and 6.39 at E_f/E_{so}=100, for the value of n=2.5, 5.0 and 10, respectively.

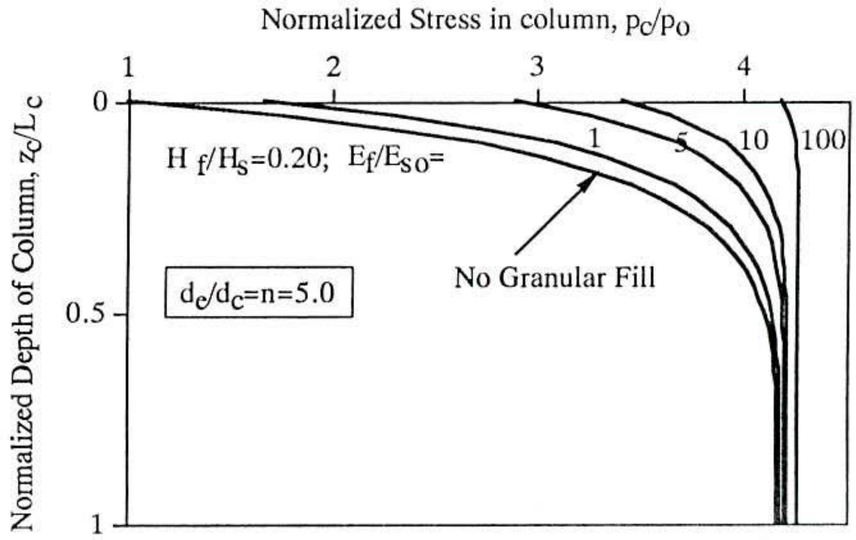


Figure 4.12 Variation of normal stress in column along the depth with the relative moduli of granular fill for H_f/H_s=0.20.

Figure 4.12 shows the variation of p_c/p_o along the depth of column for the value of E_f/E_{so}=0, 5, 10 and 100, H_f/H_s=0.20 and n =5.0. The value of p_c/p_o increases with depth and the rate of increase decreases with increasing value of E_f/E_{so}. The value of p_c/p_o is almost constant along whole length of column for E_f/E_{so}=100. A high constant value at this level indicates that no stress gets transferred along the depth for the higher value of E_f/E_{so}, which is, of course, as expected because the surface settlement is found almost uniform for the value of E_f/E_{so}=100. From Fig.4.12, it is found that p_c/p_o changes from 1.0 to 4.17 for z_c/L_c=0.0 and 1.0 for no granular fill condition but these values are 4.185 and 4.19 for E_f/E_{so}=100. Since the success of this ground improvement technique depends considerably on the concentration of stress in the columns, theses findings reveal the encouraging response of the presence of a layer of granular fill over the soft ground reinforced by columnar inclusions.

4.6 Settlement Response of End Bearing Column

The mobilization of shear stress along column-soil interface, the sharing of loads between the components of the system i.e. the column and the surrounding soft soil and the settlement profile of the reinforced ground are presented and discussed in the following sections. The ground is reinforced by a group of columns extended to bedrock. The predictions are made for two loading conditions: (i) Flexible loading i.e. no granular fill over the reinforced ground and (ii) Rigid loading i.e. a granular fill having high stiffness is placed over the reinforced ground which ensured uniform settlement. In the analysis both the no slip and possible slip situations along the column-soil interface and the stratification of soil layer, are considered. The friction angle of soil, ϕ, is considered as 30°. The value of δ is taken as equal to ϕ, to determine the limiting shear stress at the column-soil interface. Because for the considered foundation system, slip generally occurs at soil-soil interface. The values of parameters of granular fill are taken as E_f/E_{so}=100, H_f/H_s=0.333 and ν_f=0.30.

4.6.1 Distribution of shear stress

The distribution of shear stress along the depth of column for a typical example of soft ground reinforced by a group of columnar inclusions is shown in Figs.4.13(a) and (b) for nonstratified and stratified soil system respectively. The values of parameters are p_o/E_{so}=0.10, $n=d_e/d_c$=5.0, L_c/d_c=10, L_c/H_s=1.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. For nonstratified soil system, in Fig.4.13(a), the mobilized shear stress at the top region of column is different for no slip and slip situations. This behaviour is, of course, expected due to the consideration of limiting shear stress at column-soil interface. For no slip situation, the interface shear stress is maximum at the top of column i.e. τ/p_o=0.567 at z_c/L_c=0.0, then decreases and tends to zero at the bedrock level. But, for slip situation, as expected, the shear stress at the top of column is

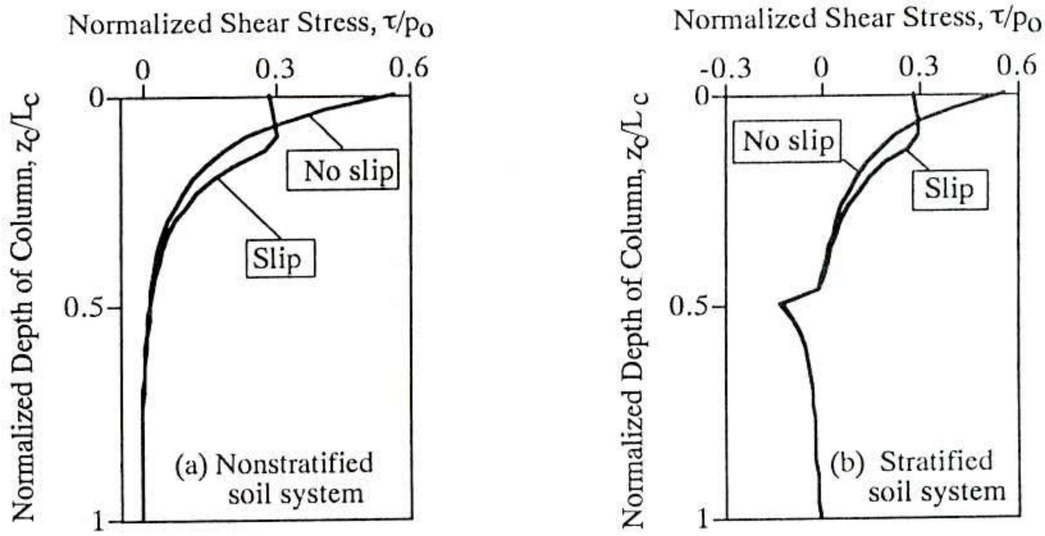


Figure 4.13 Distribution of shear stress at column-soil interface along the depth for end bearing column-reinforced soft ground.

less than that of no slip situation and increases slightly up to a certain depth beyond which decreases and tends to zero at the bedrock level. At the top, the value of τ/p_o=0.289, it increases to a value 0.303 and then decreases and tends to zero at bedrock level. The depth of slip zone is found as $0.10L_c$ measured from the top of column. This figure shows that the shearing stresses are developed at the upper portions of column and a considerable portions of lower part of column remains almost shear stress free.

The mobilized shear stress along the column-soil interface in stratified soil system is shown in Fig.4.13(b). The thickness of upper and lower soil layers is the same i.e. H_{s1}/H_{s2}=1.0. The same values of parameters stated above for Fig.4.13(a), are considered here, only the modulus of lower soil layer is taken as five times higher than that of the upper soil layer i.e. E_{s2}/E_{s1}=5.0. Fig.4.13(b) shows that the depth of slip zone and the interface shear stress at the upper portions of column are almost the same as observed for the nonstratified soil system. But at just above and below the interface of two soil layers, the distribution of shear stress is somewhat different when compared with nonstratified soil systems. The influence of soil stratification on the interface shear stress is evident which is observed here near the interface of two soil layers. For no slip case the values of τ/p_o are -0.006 , -0.126 and -0.078 at the depth z_c/L_c=0.45, 0.50 and 0.55, respectively. In case of slip for the same depths, the values of τ/p_o are -0.003 , -0.119 and -0.075, respectively.

4.6.2 Variation of stresses in column and soil

The variation of normal stress in column, p_c, along the depth is shown in Figs.4.14(a) and (b) for the case of nonstratified and stratified soil systems, respectively. The same examples stated in section 4.6.1, are considered for this demonstration. Figure 4.14(a) shows that the value of normalized vertical stress in column, p_c/p_o, is unity at the surface, z_c/L_c=0.0. It increases with depth and beyond a certain depth becomes almost constant. The increase of stress in column along the depth indicates the stress transfer from softer soil media towards the stiffer column along the depth. In the upper portion of column the value of p_c/p_o, obtained from no slip and slip situations, is clearly distinct. Higher value of p_c/p_o is obtained for no slip case comparing with slip. At a depth z_c/L_c=0.25, the values of p_c/p_o are 3.62 and 3.43 for no slip and slip situations, respectively. The consideration of limiting shear stress restricts the magnitude of mobilized shear stress which, in turn, reduces the amount of stress transfer from soil to column. However, at the lower portion, the value of p_c/p_o becomes almost the same, which is around 4.15, for no slip and slip situations. In Fig.4.14(b), the value of p_c/p_o increases with depth and reached a maximum value just above the interface of soil layers beyond which it decreases gradually up to bottom of column. In the upper portion of column the value of p_c/p_o, obtained from no slip and slip situations, is clearly distinct. But in the lower

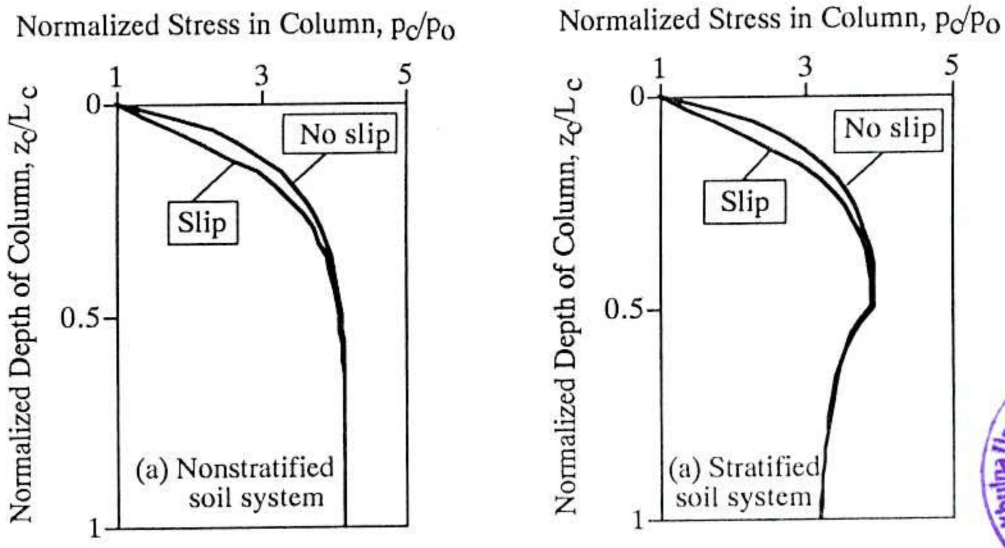


Figure 4.14 Variation of normal stress in column along the depth for end bearing column-reinforced soft ground.

portion of column, the differences decreases and it becomes almost the same at the bottom. At a depth z_c/L_c=0.45, the values of p_c/p_o are found as 3.96 and 3.90 for no slip and slip situations, respectively. But at z_c/L_c=1.0, the value of p_c/p_o is found as around 3.21 for both slip and no slip cases.

The variation of normal stress in soil, p_s, along the depth, z_c/H_s, is shown in Figs.4.15(a) and (b) for nonstratified and stratified soil systems, respectively. Figure 4.15(a) shows that the value of normalized vertical stress in soil, p_s/p_o, is unity at the surface i.e. z_c/H_s=0.0. It decreases along the depth and beyond a certain depth becomes almost constant. At the upper portion of soil layer, the value of p_s/p_o obtained from no slip and slip situations, differs considerably from each other. But at the lower portion, it attains almost a unique value, which is around 0.13. At a radial distance r/a=2.60 and at a depth z_c/L_c=0.25, the values of

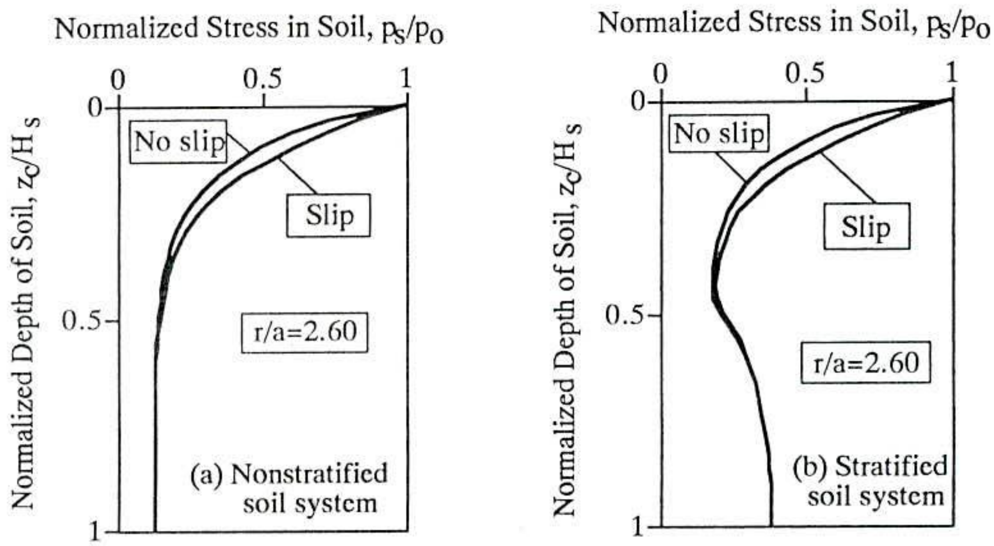


Figure 4.15 Variation of normal stress in soil along the depth at a radial distance r/a=2.60 for end bearing column-reinforced soft ground.

p_s/p_o are 0.23 and 0.28 for no slip and slip situations, respectively. The variation of p_s/p_o with z_c/H_s for stratified soil system, is shown in Fig.4.15(b). The value of p_s/p_o decreases with depth and reached a minimum value just above the interface of soil layers beyond which it increases gradually up to the base of soil layer. In the upper portion of column the value of p_s/p_o, obtained from no slip and slip situation, is clearly distinct. But in the lower portion of column, the differences decreases and it becomes almost the same at the base. At a depth z_c/H_s =0.45 and at a radial distance r/a =2.60, the values of p_s/p_o are 0.182 and 0.195 for no slip and slip situations, respectively. But at z_c/H_s=1.0, the value of p_s/p_o is around 0.39 for both slip and no slip cases.

The variation of normal vertical stresses in soil, p_s/p_o, with normalized radial distance, r/a, is presented in Figs.4.16(a) and (b) for nonstratified and stratified soil systems,

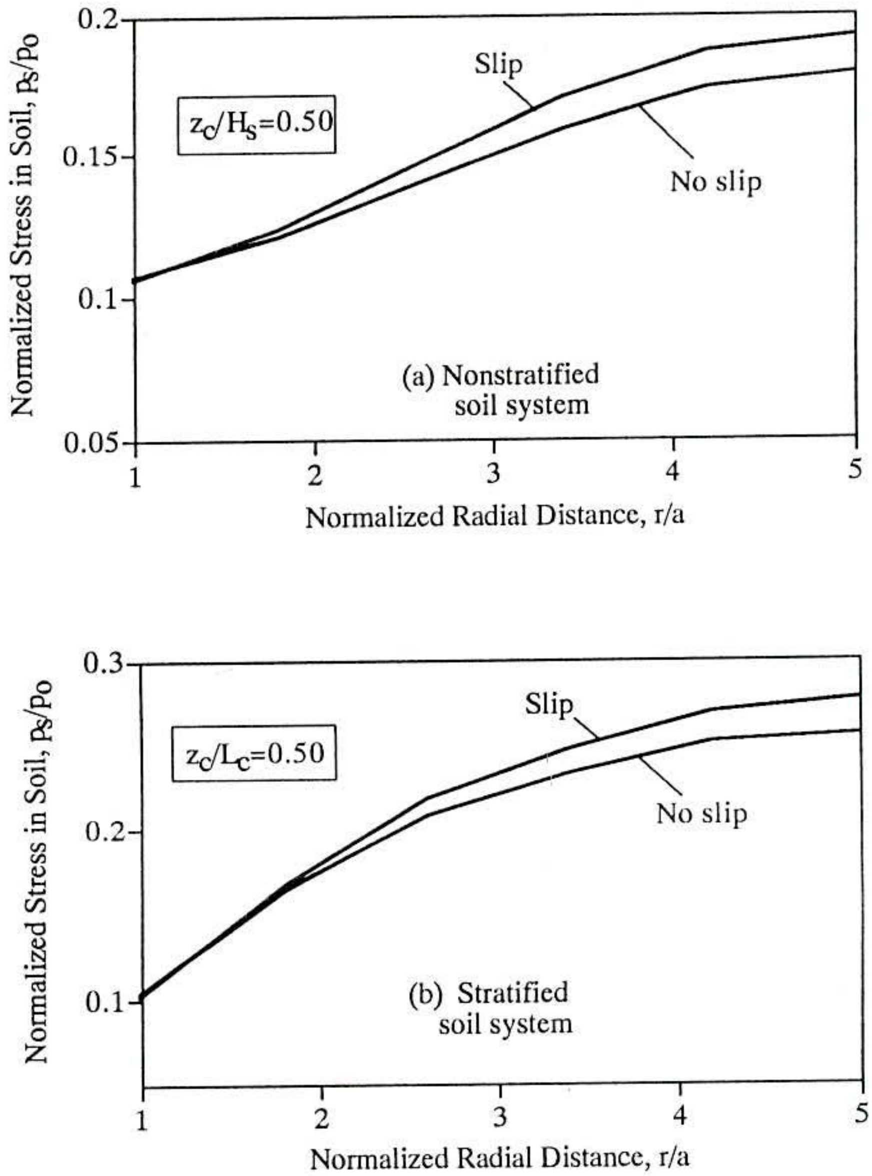


Figure 4.16 Variation of normal stress in column along the radial distance at a depth z_c/H_s=0.50 for end bearing column-reinforced soft ground.

respectively. From both figures, it can be seen that the value of p_s/p_o increases with r/a and acquired the ximum value at the boundary i.e. at $r/a=n$. The value of p_s/p_o for slip case always higher than its no slip counterpart. Which is, of course, as expected. The stress transfer from soil to column restricts due to introducing the limiting shear stress at column-soil interface. Which, in turn, increases the magnitude of p_s/p_o in soil. Comparing the magnitudes of p_s/p_o in these two figures, the influence of a stiffer soil layer at the lower portion is evident. Figure 4.16(b) shows the higher magnitude of p_s/p_o than its nonstratified counterpart.

4.6.3 Settlement profile of the treated ground

The settlement profile of the column-reinforced ground is presented in Figs.4.17(a) and (b) for nonstratified and stratified soil systems, respectively. The results are presented for flexible and as well as rigid loading conditions. The same example and the magnitudes of parameters as taken in the previous two sections are considered. In case of flexible loading, the differences between the normalized settlement of composite ground, S_t/H_s, at the column region, $0 \leq r/a \leq 1$, and that of the surrounding soil region, $1 \leq r/a \leq n$, are found to be noticeable. The differential settlement increases with radial distance r/a and attains a maximum value at the boundary i.e. at $r/a=n$. This observation is valid for both the nonstratified and stratified soil systems. The differential settlement is more for slip situation than its no slip counterpart. For slip analysis, the differential settlement increases as the limiting shear stress at column-soil interface decreased. Because, the amount of load transfer to the stiffer column gets reduced. As a result, soil settles more but column settles less. For nonstratified soil system, in Fig.4.17(a), the values of S_t/H_s at a radial distance $r/a=1$ are 0.0076 and 0.0146 for no slip and slip situations, respectively. At a radial distance $r/a=5$, these values are 0.024 and 0.0255 for no slip and slip situations, respectively.

For the stratified soil systems, in Fig.4.17(b), the values of S_t/H_s are 0.007 and 0.0137 for no slip and slip situations, respectively, at a radial distance $r/a=1$. At $r/a=5$, these values are 0.023 and 0.0245 for no slip and slip cases, respectively. From these predictions, it

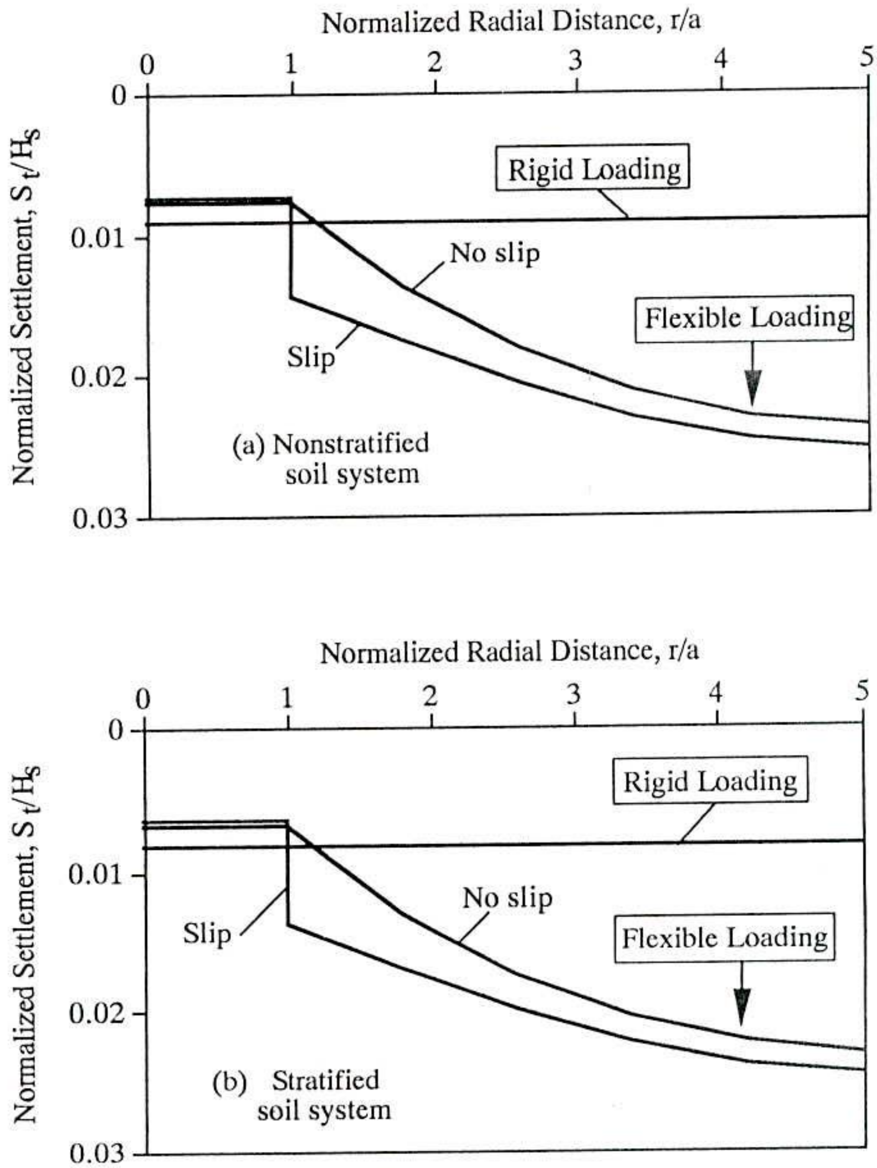


Figure 4.17 Settlement profiles of soft ground reinforced by end bearing columnar inclusions subjected to flexible and rigid loading.

is revealed that the value of S_t/H_s at r/a=1 to 5, is decreased only from 8.57 to 4.17% for no slip and 7.14 to 4.86% for slip cases, respectively, comparing the nonstratified case. For rigid loading the uniform settlement is observed over reinforced ground. The values of S_t/H_s are obtained 0.009 and 0.008 for nonstratified and stratified soil systems, respectively. In all cases, the amount of settlement reduction is not significant at all as the modulus of lower soil layer is five times higher than that of the upper layer. The results indicate that the role of upper soil layer to be pronounced than that of the lower layer. These findings reveal that in case of uniform surface loading, the soft ground reinforced by columnar inclusions exhibits considerable amount of differential settlement if no granular fill is placed on the top.

4.7 Settlement Response of Floating Column

The penetration of column up to the full depth of a deep soft soil layer may not be viable economically and technically, in some instances. In such cases the improvement of soft ground is often done by partial penetration of columns into the deep soil layer. It is also revealed in some cases that the full penetration of column is not necessary to achieve the required degree of improvement (DeStephen et al. 1992). However, the settlement response of floating column is somewhat different from its end bearing counterpart. The predictions are made for soft ground reinforced by a group of floating columns subjected to either flexible or rigid loading. The stratification of soil system and the possible slip at column-soil interface are taken into consideration. The friction angle of soil ϕ, is considered as 30° and the values of δ is taken as ϕ, to determine the limiting shear stress at the column-soil interface. The values of the parameters of granular fill are taken as E_f/E_{so}=100, H_f/H_s=0.333 and ν_f=0.30. The distribution of shear stress, the variation of normal stresses in column and soil and the settlement profile of the treated ground are presented and discussed in the following sections.

4.7.1 Distribution of shear stress

The distribution of shear stress along the depth of column for a typical example of

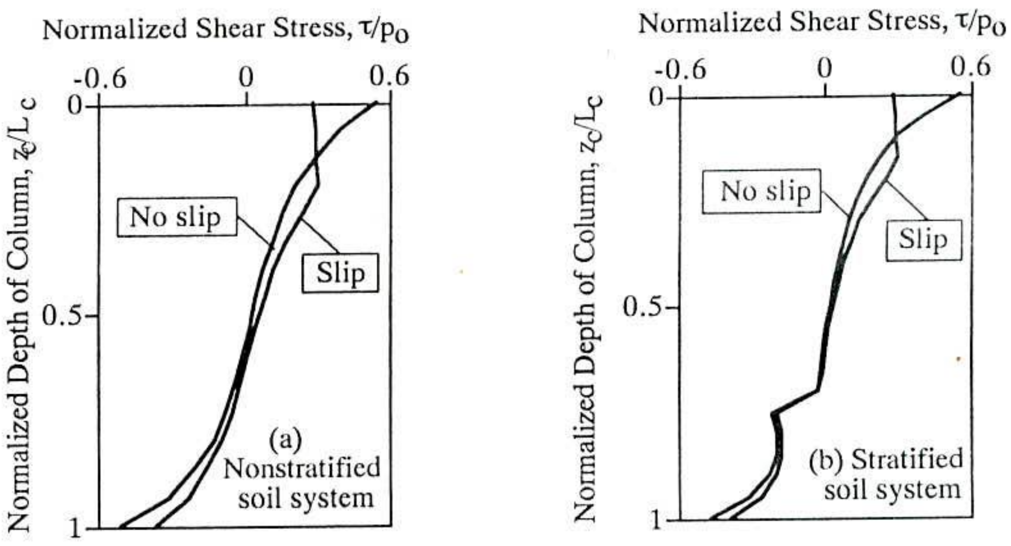


Figure 4.18 Distribution of shear stress at column-soil interface along the depth for floating column-reinforced soft ground.

soft ground reinforced by floating column is shown in Figs.4.18(a) and (b) for nonstratified and stratified soil systems, respectively. The values of parameters are p_o/E_{so}= 0.10, d_e/d_c =5.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. Flexible load is acting over the entire area. The values of L_c/H_s and L_c/d_c are 0.50 and 5 for the nonstratified and 6.67 and 0.667 for the stratified soil systems, respectively. In stratified soil systems, equal thicknesses for the upper and the lower soil layers are considered i.e. H_{s1}/H_{s2}=1.0. The magnitudes of parameters are the same for nonstratified and stratified soil systems. Only the modulus of lower soil layer is five times higher than that of the upper soil layer i.e. E_{s2}/E_{s1} =5.0. The predictions presented in Figs.4.18(a) and (b) show that the interface shear stress is positive (i.e. downwards) on the upper portion of the column but becomes negative (i.e. upwards) after a certain depth. The elevation at which this change in sign of the shear stress takes place is known as the neutral depth (Vesic 1977). From Fig.4.18(a), it can be seen that the magnitude of neutral depth obtained from slip case is higher than that of no slip case. The values of neutral depth are $0.55L_c$ and $0.65L_c$ for no slip and slip situations respectively. The influence of soil stratification on the distribution of shear stress is evident. The shear stress at column-soil interface in stratified soil system is somewhat different than that of nonstratified soil systems, which is shown in Fig.4.18(b). The depths of neutral plane do not change significantly for no slip and slip conditions and are found to be $0.62L_c$ and $0.63L_c$, respectively. In Figs.4.18(a) and (b), the depths of slip zones are predicted as $0.2L_c$ and $0.15L_c$ for nonstratified and stratified soil systems, respectively. These are higher than those of obtained for end bearing columns. This is, of course, expected. These results reveal that while for the case of end bearing columns a considerable portion of column remains almost shear stress free but in case floating columns, the whole length is subjected to shear stress either positive (i.e. downwards) or negative (i.e. upwards).

4.7.2 Variation of stresses in column and soil

The same example and magnitudes of parameters as stated in the section 4.7.1 are used here to illustrate the variation of normal stresses in column, p_c, and soil, p_s, in case of floating column. The nonstratified and stratified soil systems are considered for no slip and slip situations. Figure 4.19(a) and (b) represent the variation of normalized vertical stress in column p_c/p_o, along the depth z_c/L_c, for nonstratified and stratified soil systems respectively. From unity at the surface z_c/L_c=0.0, the value of p_c/p_o increases with depth and beyond a certain depth decreases up to the end of column. The variation of p_c/p_o, obtained from no slip and slip situations, is clearly distinct. The value of p_c/p_o increases with the increasing value of limiting shear stress and higher value is obtained for no slip case comparing with slip up to a depth around $0.0<zc/Lc\leq 0.85$. Beyond this depth, this behaviour is reverse. In Fig4.19(a), the depths of maximum values of p_c/p_o are $0.60L_c$ and $0.65L_c$ for no slip and slip situations, respectively. The maximum values of p_c/p_o are 3.50 and 3.48 for no slip and slip situations,

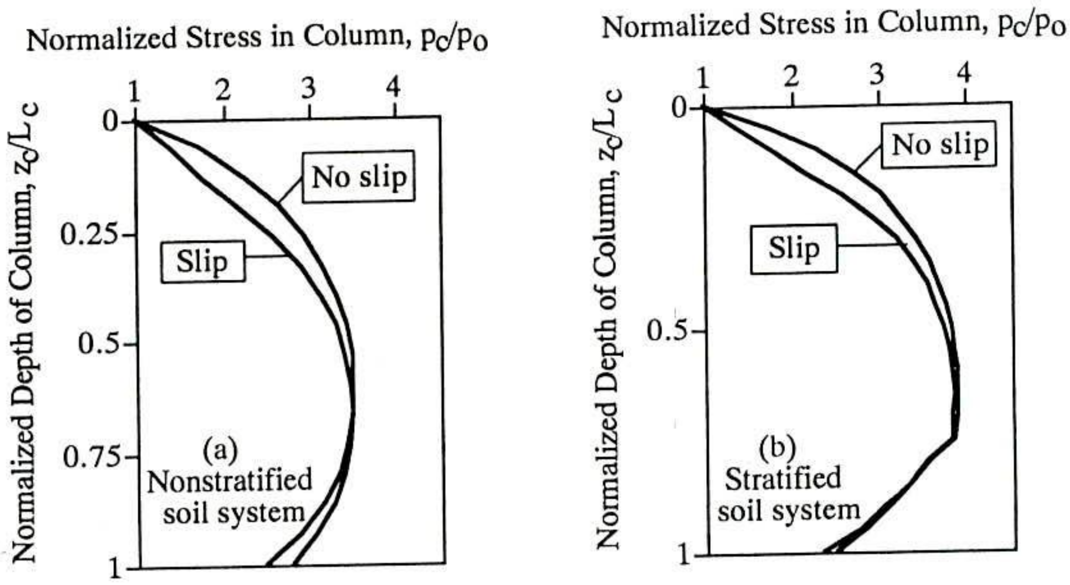


Figure 4.19 Variation of normal stress in column along the depth for floating column-reinforced soft ground.

respectively. But at the bottom of column i.e. z_c/L_c=1.0, these values are 2.47 and 2.77, respectively. For stratified soil system, in Fig.4.19(b), the depths of maximum values of p_c/p_o are obtained almost the same i.e. 0.65L_c for the both no slip and slip situations, respectively. The maximum values of p_c/p_o are 3.90 and 3.85 for no slip and slip cases, respectively. But at the bottom of column i.e. at z_c/L_c=1.0, these values are 2.36 and 2.50, respectively. This changing pattern of p_c/p_o is evident as the interface shear stress changes their sign from positive to negative. Stress is transferred from soil to column up to the neutral depth but beyond this stress transferring occurs from column to soil. The presence of a stiffer soil layer shows the rapid decrease of stress in the lower region of column which is shown in Fig.4.19(b).

The variation of normal stress in soil, p_s, along the depth is shown in Figs.4.20(a)

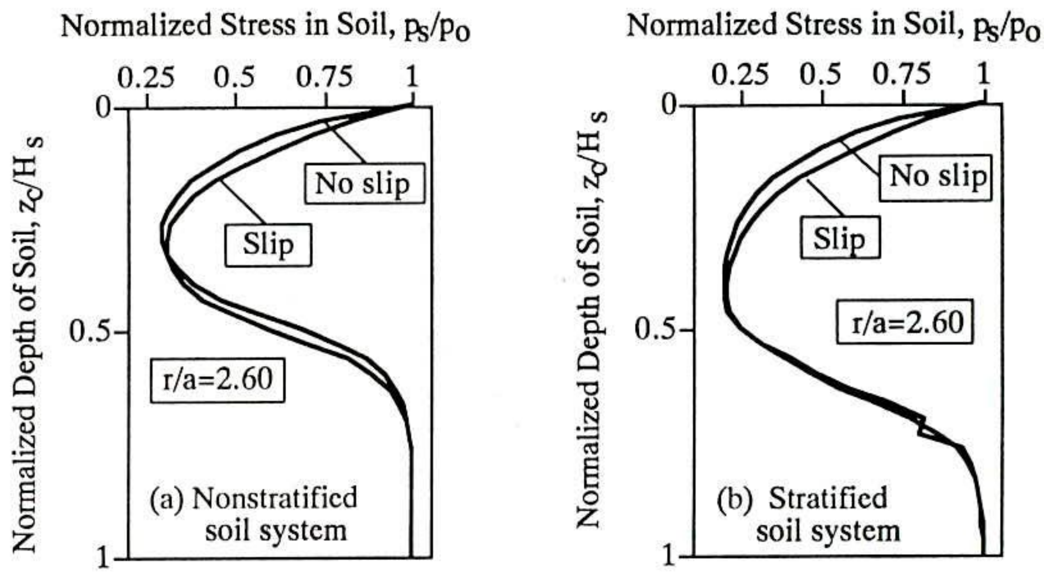


Figure 4.20 Variation of normal stress in soil along the depth at a radial distance r/a=2.60 for floating column-reinforced soft ground.

and (b) for nonstratified and stratified soil systems, respectively. The value of normalized stress in soil, p_s/p_o, is unity at the surface i.e. at z_c/H_s=0.0. It decreases along the depth and reached a minimum value at a certain depth. Beyond this depth, it increases and tends to be unity at the base. At the upper portion of soil layer, the value of p_s/p_o obtained from no slip and slip situations, differs considerably from each other. But at the lower portion, $z_c/H_s>0.5$, the differences are negligible. In Fig.4.20(a), the depths of minimum values of p_s/p_o are almost the same i.e. $0.30L_c$ for the both no slip and slip situations, respectively, in case of nonstratified soil system. The minimum values of p_s/p_o are 0.298 and 0.308 for no slip and slip situations, respectively. For stratified soil system, in Fig.4.20(b), the depths of minimum values of p_s/p_o are also almost the same i.e. $0.40L_c$ for no slip and slip situations, respectively. The minimum values of p_s/p_o are 0.196 and 0.21 for no slip and slip cases, respectively. This changing pattern of stress in soil along the depth is quite expected. As the interface shear stresses change their sign from positive to negative, stress transferring occurs from column to surrounding soil beyond the neutral depth. These two figures also reveal that near the base of soil layer, the stress transferring is almost zero at a radial distance r/a=2.60.

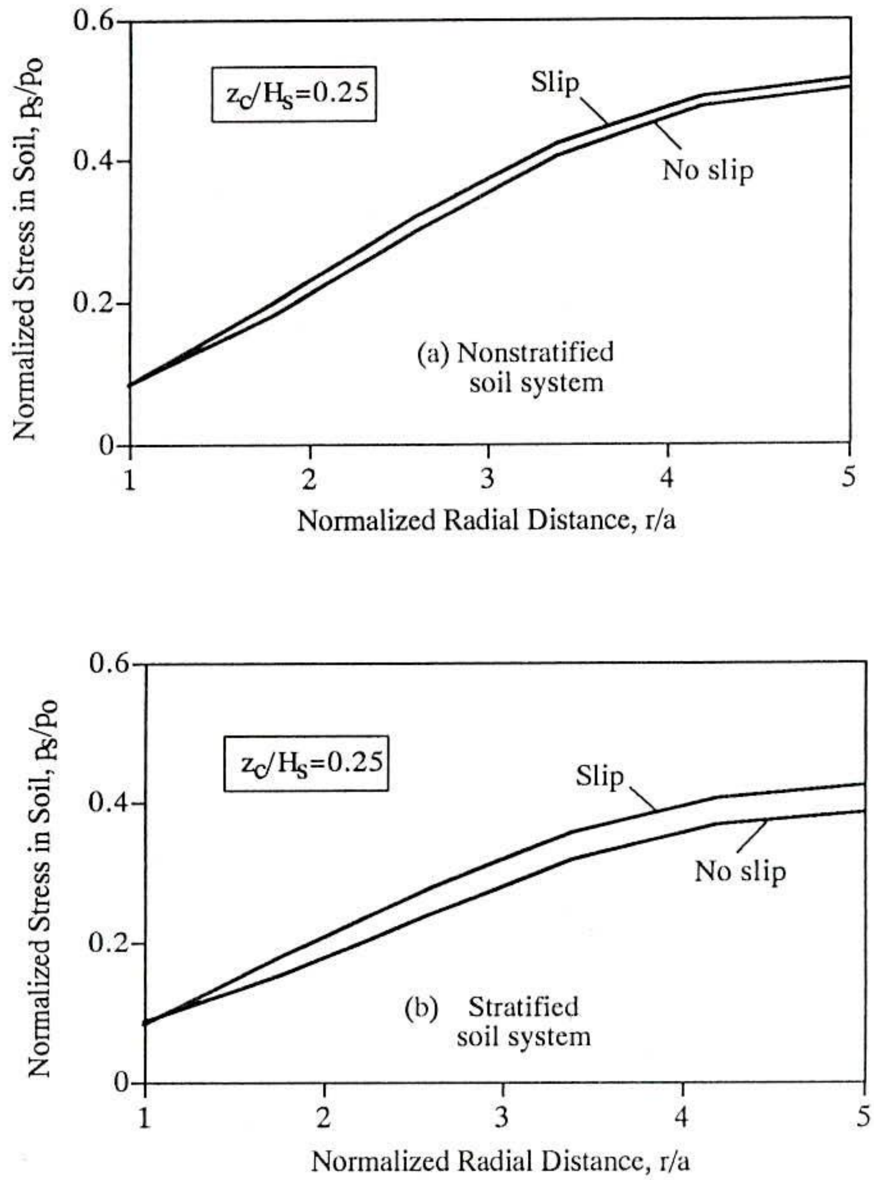


Figure 4.21 Variation of normal stress in soil along the radial distance at a depth z_c/H_s=0.25 for floating column-reinforced soft ground.

The variation of normalized vertical stresses in soil, p_s/p_o, with normalized radial distance, r/a, is presented in Figs.4.21(a) and (b) for nonstratified and stratified soil systems, respectively. From both the figures, it can be seen that the value of p_s/p_o increases with r/a and acquired the maximum value at the boundary i.e. at $r/a=n$. For slip case, the value of p_s/p_o is always higher than its no slip counterpart, as expected. The stress transfer from soil to column decreases due to introducing limiting shear stress at column-soil interface. This, in turn, increases the magnitude of normal stress in soil.

4.7.3 Settlement profile of the treated ground

The settlement profile of the soft ground reinforced by floating column in nonstratified and stratified soil systems is presented in Figs.4.22(a) and (b), respectively. The same example and the magnitudes of parameters as taken in the previous two sections are considered for the predictions. In case of flexible loading, the normalized settlement of composite ground S_t/H_s, at the column region, $0 \le r/a \le 1$, is less than that of the surrounding soil region, $1 \le r/a \le n$. The differential settlement increases with radial distance r/a and attains a maximum value at the boundary i.e. at $r/a=n$. This observation is valid for both the nonstratified and stratified soil systems. The differential settlement is more for slip situation than its no slip counterpart. The limiting shear stress at column-soil interface decreases the amount of load transfer from soil to column. Therefore, the normal stress in column gets reduced compared to no slip case. As a result, soil settles more but column settles less. In Fig.4.22(a), at a radial distance $r/a=1$, the values of S_t/H_s are 0.0427 and 0.0474 for no slip and slip situations, respectively. At radial distance $r/a=n=5$, these values of S_t/H_s are obtained as the same, which is 0.058, for no slip and slip situations, respectively. For the stratified soil system, in Fig.4.22(b), at a radial distance $r/a=1$, the values of S_t/H_s are 0.0111 and 0.0177 for no slip for slip situations, respectively. At $r/a=5$, these values are 0.0273 and 0.0285 for no slip and slip situations, respectively. For rigid loading, the uniform settlement is observed over the entire reinforced ground. The values of S_t/H_s are obtained 0.0445 and 0.0126 for nonstratified and stratified soil systems, respectively. These results indicate that the influence of relatively stiffer soil layer below the soft soil layer is significant in reducing the overall

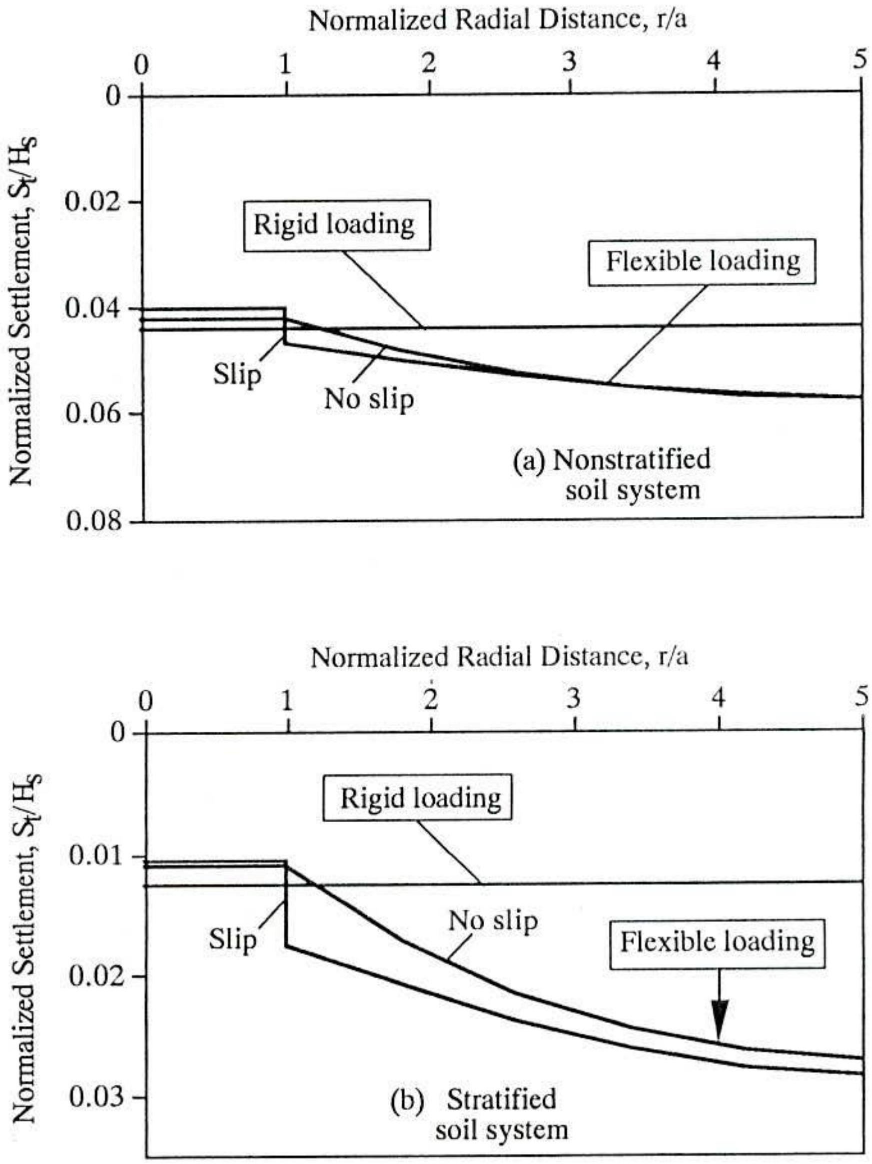


Figure 4.22 Settlement profiles of soft ground reinforced by floating columnar inclusions subjected to flexible and rigid loading.

settlement of the improved ground while compared with the findings presented in Figs.4.17(a) and (b) for nonstratified and stratified soil system, rspectively. In the case of floating columns the whole length of column is subjected to shear stress and the role of lower soil layer is become important since beyond the neutral plane stress transferring occurs from column to soil. The settlement profiles reveal that the soft ground reinforced by even floating columns and subjected to uniform surface loading exhibits considerable amount differential settlement if no granular fill is placed on the top.

4.8 Influence of Various Design Parameters

The predictions are made to illustrate the influences of various design parameters such as spacing of columns, length to diameter ratio of column, degree of penetration of column into soft ground, relative stiffness of column and surrounding soil, Poisson's ratio of soil and angle of friction between column and soil. The results are presented to show the distribution of shear stresses along the column-soil interface, the variation of stress concentration into column and the settlement profile of the treated ground. The evaluations are made and presented in the following sections for the case of flexible loading i.e. granular fill over the reinforced ground. The columns are installed into a nonstratified soft soil system and the analysis is performed for no slip condition. This simple situation is considered as the cardinal aim of this section to illustrate the influence of the variation of values of different parameters on the predictions.

4.8.1 Spacing of columns

The spacing of columns is an important parameter which accounts for the effectiveness of soft ground improvement by columnar inclusions. The wider spacing gives economically viable solution but may not satisfy the design requirements while on the other hand closer spacing provides opposite effects. The distribution of shear stress, the variation of

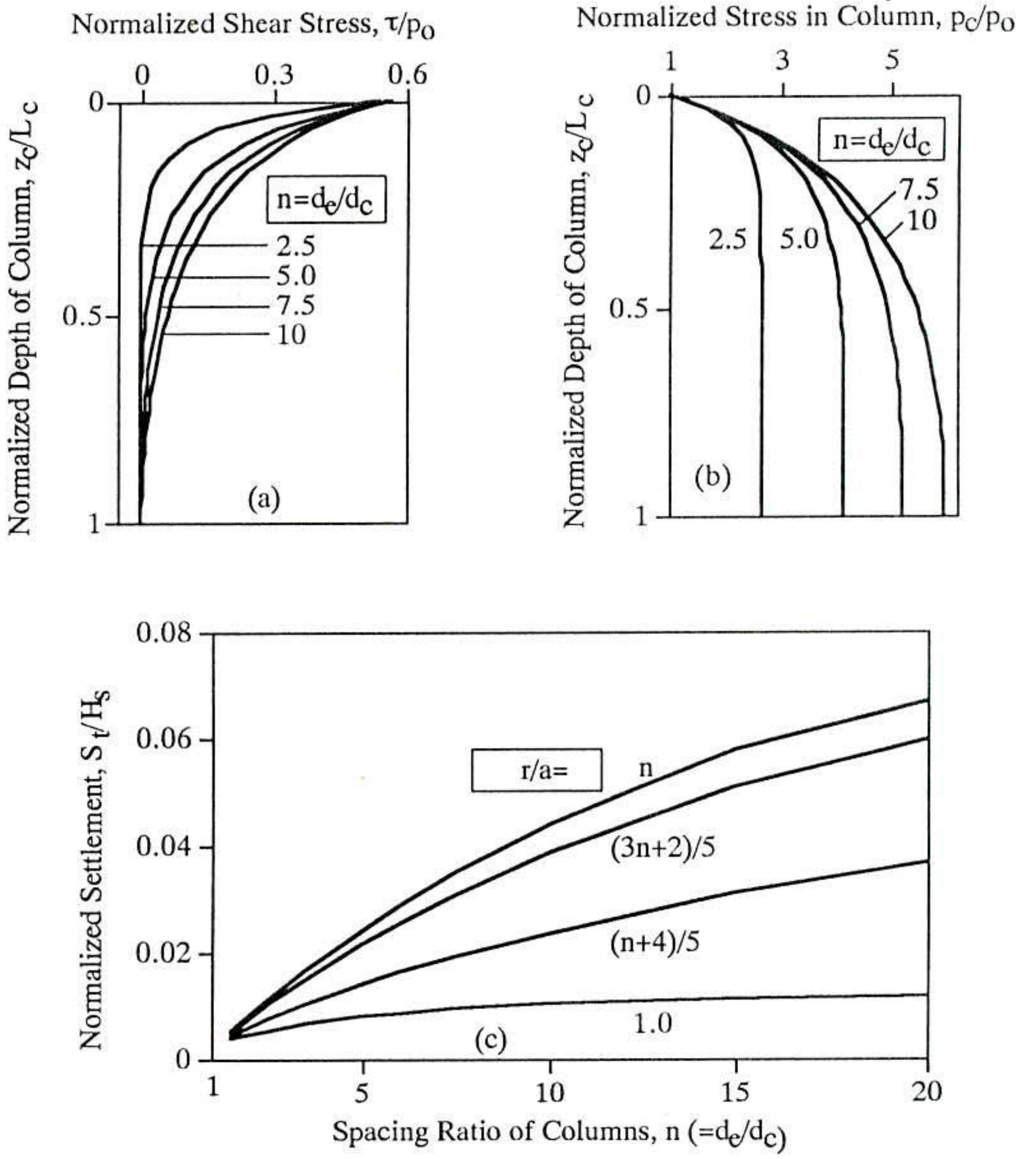


Figure 4.23 Influence of spacing of columns: (a) Distribution of shear stress at column-soil interface; (b) Variation of normal stress in column; and (c) Settlement of reinforced ground.

stress concentration in column and settlement of the treated ground for different column spacings are shown in Figs.4.23(a), (b) and (c), respectively. The predictions are made for the values of p_o/E_{so}=0.10, L_c/d_c=10, L_c/H_s=1.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. The values of $d_e/d_c(=n)$, which represent the spacing of columns, are varied from 1.5 to 20. Figure 4.23(a) shows that for all the cases, the shear stresses are high at the top of the column and decrease gradually with depth and tend to zero at bedrock level. The shear stresses at the interface increase with the increasing value of column spacings. At a depth z_c/L_c=0.20, the value of τ/p_o increases from 0.025 to 0.207 for n=2.5 to 10. From this figure, it can be seen that the shear stress free portion of column increases with the decreasing value of column spacing. The soil arches between the columns and thus prevents the mobilization of shear stresses. The arching of soil increases with the closer spacing of columns. Figure 4.23(b) shows the variation of normalized vertical stress in column, p_c/p_o, for different column spacings n. From this figure, it is revealed that at a certain depth, the magnitude of p_c/p_o increases with column spacing. At a depth z_c/L_c=0.50, the value of p_c/p_o increases from 2.675 to 5.478 for n=2.5 to 10. This tend of increasing p_c/p_o is valid along the whole length of column. As the magnitude and the zone of interface shear stress increase with spacing, the transfer of stress from soil to column extends over larger length of column and increases the value of p_c/p_o, with spacing. In the stated case, the value of p_c/p_o varies from 2.65 to 5.97 at the bottom of column for n=2.5 to 10, respectively. This result reveals that for a particular situation, the stress in column depends significantly on its spacing.

The influence of column spacings on the settlement of the reinforced ground is shown in Fig.4.23(c). The settlements of the surrounding soil are predicted at radial distances r/a=1.0, $(n+4)/5$, $(3n+2)/5$ and n. For very close spacing i.e. $1<n<1.5$, the settlements are almost the same all over the area of the composite ground, which occurs from the high arching effect of soil in between the columns. The overall and the differential settlements of the reinforced ground increase with column spacing and radial distance. The effect of column spacing is more prominent on the soil region than that of on the column region. The value of S_t/H_s increases from 0.0034 to 0.0115 for $0\le r/a\le 1$ and 0.0049 to 0.066 at $r/a=n$ for the increase of n value from 1.5 to 20. This result indicates the insignificant role of column in reducing the settlement of the reinforced ground for n>10. Closer spacing of columns is effective in reducing the overall and the differential settlements of the reinforced ground.

4.8.2 Length to diameter ratio of columns

The length and the diameter of columns have significant influence on the overall settlement response of the soft ground reinforced by columnar inclusions. To illustrate this influence, results are presented for different length to diameter ratio of column, L_c/d_c, for the columns extended to bedrock. The evaluations are performed for the values of p_o/E_{so}=0.10, d_e/d_c=5.0, L_c/H_s =1.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. The values of L_c/d_c vary from 5 to 20. The influence of L_c/d_c on the distribution of shear stress, the variation of normal stress in column and the settlement of the treated ground are presented in Figs.4.24(a), (b) and (c), respectively. From Fig.4.24(a), it can be seen that the pattern of distribution of shear stress along the column-soil interface is similar for all values of L_c/d_c. The lower value of length to diameter ratio offers better interaction between the column and the surrounding soil and thus causes the mobilization of shear stress along whole length of column. For L_c/d_c=20, the two-thirds of the column remains almost shear stress free but L_c/d_c=5, the shear stress is acting along the whole length of the column. At the mid point of column i.e. z_c/L_c=0.25, the values of τ/p_o are 0.211, 0.085 and 0.016 for L_c/d_c=5, 10 and 20, respectively. Figure 4.24(b) shows the variation of normalized stress in column along the depth for L_c/d_c=5, 10 and 20. For all the values of L_c/d_c, the values of p_c/p_o increase with depth in the similar manner. From this figure, it can be seen that at a particular depth of column, the value of p_c/p_o increases with L_c/d_c. It is because the length of column, in which the downdrag force is acting, increases with L_c/d_c and thus increases the magnitude of p_c/p_o. At a depth z/L_c=0.25, the values of p_c/p_o is found as 2.95, 3.64 and 4.09 for L_c/d_c=5, 10 and 20, respectively.

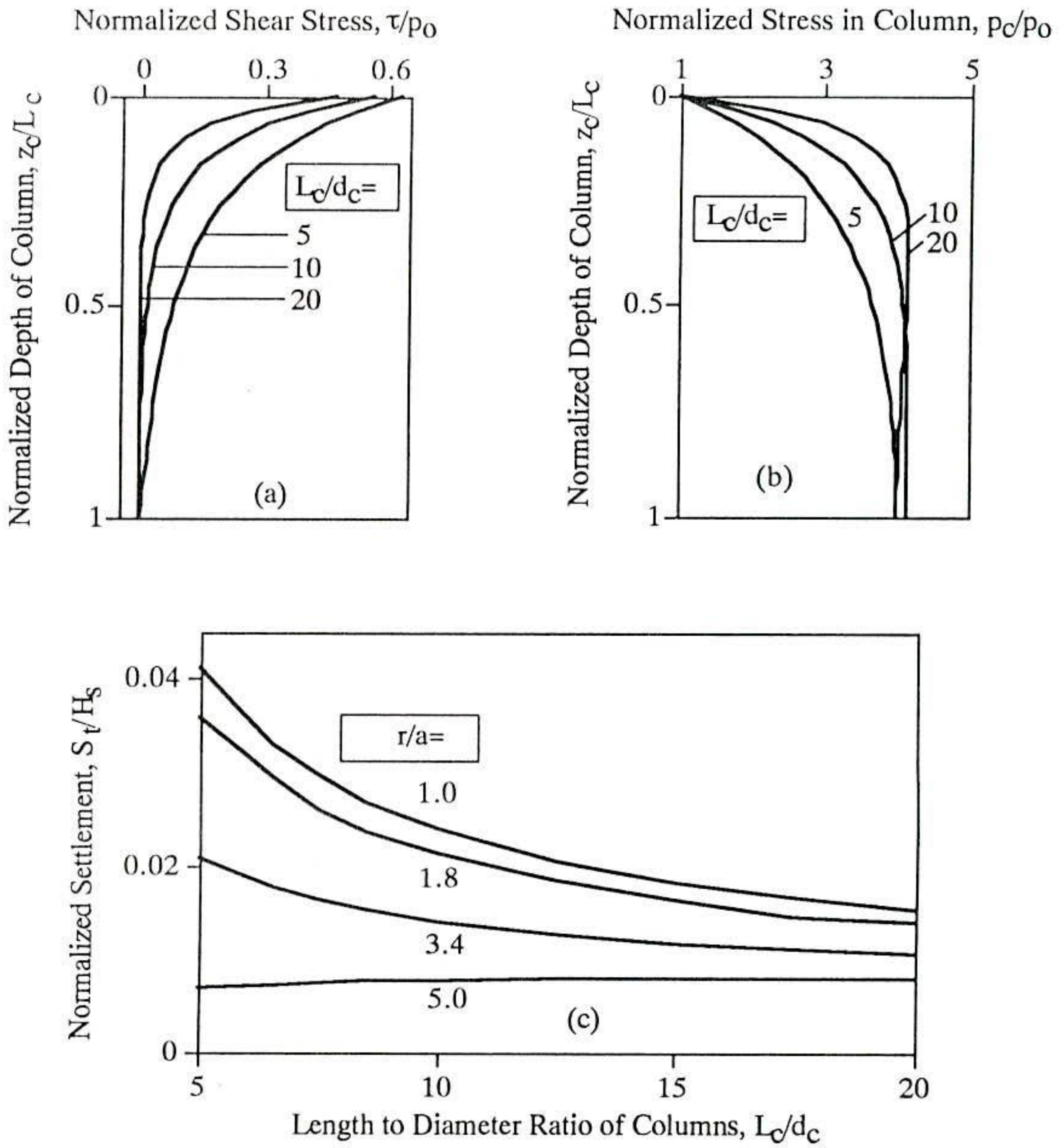


Figure 4.24 Influence of length to diameter ratio of column: (a) Distribution of shear stress at column-soil interface; (b) Variation of normal stress in column; and (c) Settlement of reinforced ground.

The normalized settlements of the improved ground, S_t/H_s, with L_c/d_c at r/a=1.0, 1.8, 3.4 and 5.0, are shown in Fig.4.24(c). The overall and the differential settlements decrease with the increasing value of L_c/d_c. The values of S_t/H_s are obtained as 0.0068 to 0.0079 at r/a=1.0 and 0.0411 to 0.0153 at r/a=5.0 for L_c/d_c=5 to 20. This figure shows that the decrease of differential settlement is significantly high for the increase of L_c/d_c. It is around 75% for the increase of L_c/d_c from 5 to 20. It also reveals that the influence of L_c/d_c on settlement reduction is more sensitive for lower value of length to diameter ratio i.e. L_c/d_c=5 to 10, than that of higher value of length to diameter ratio i.e. L_c/d_c=10 to 20. This may be explained from the variation of stress concentration in the column along the depth as observed in Fig.4.24(b). The higher value of L_c/d_c provides larger length of column for the interaction with surrounding soil and thus causes the reduction of normalized settlement.

4.8.3 Degree of penetration of columns

The influence of degree of penetration of column, L_c/H_s, on the settlement behaviour of soft ground reinforced by columnar inclusions, is presented in this section. Predictions are made considering the values of parameters are p_o/E_{so}=0.10, d_e/d_c=5.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. The values of L_c/H_s vary from 0.25 to 1.00 (L_c/d_c=2.5 to 10). The distribution of shear stress, the variation of normal stress in column and the settlement of the treated ground are presented in Figs.4.25(a), (b) and (c), respectively. Fig.4.25(a) shows that the positive (i.e. downwards) and negative (i.e. upwards) shear stresses increase with the decreasing value of L_c/H_s. This may happen as the relative movement of column and surrounding soil at the bottom of column increases with the decrease of L_c/H_s. For all values of L_c/H_s (=0.25 to 0.75), the column experiences both positive and negative values of τ/p_o. It is interesting to note that the depth of neutral plane is almost at the same location of column for the values of L_c/H_s=0.25 to 0.75. It is around $0.6L_c$ from the top of column as shown in Fig.4.24(a). The variation of normal stress in column along the depth with the degree of penetration of column is presented in Fig.4.25(b). The value of p_c/p_o increases

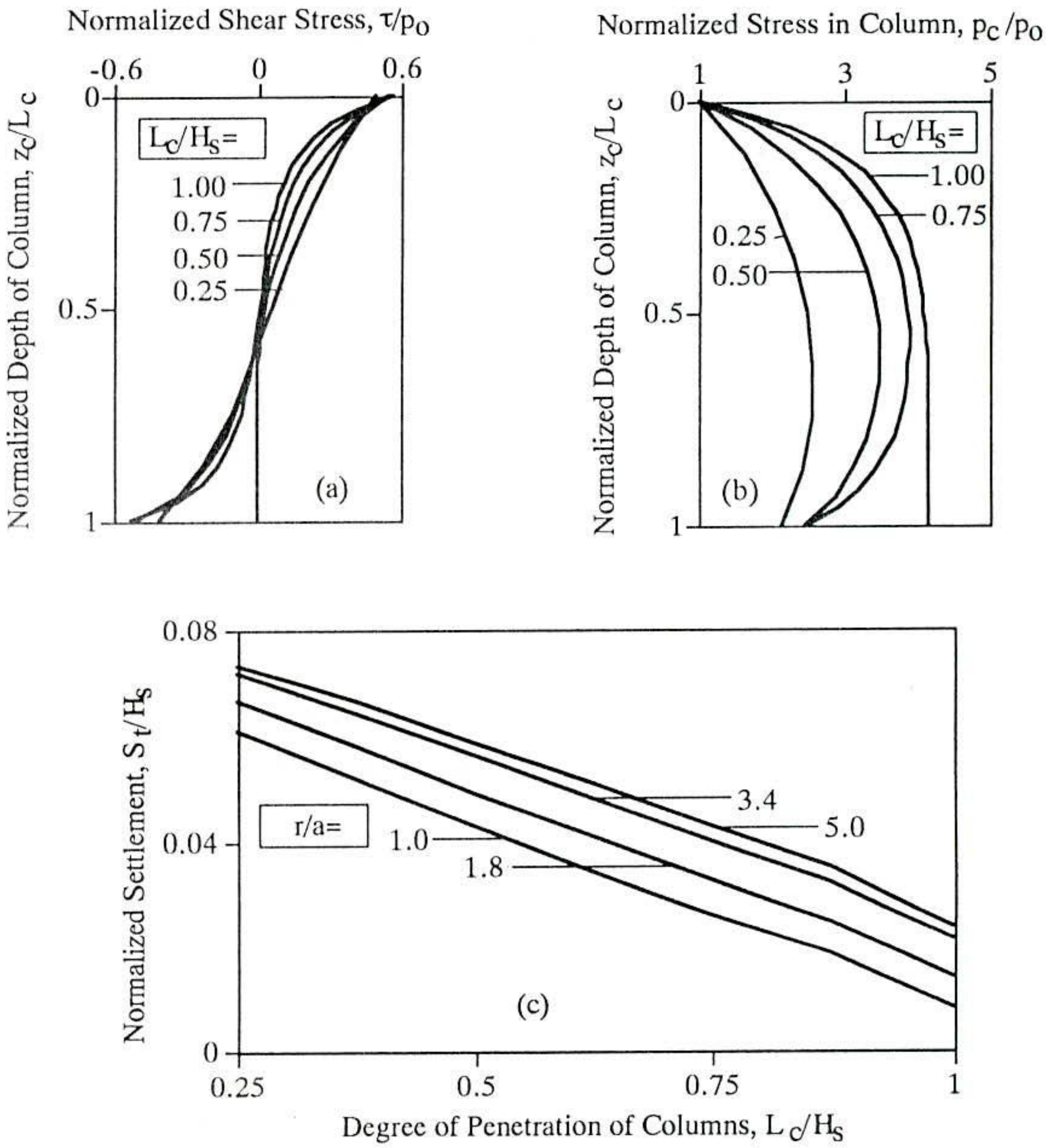


Figure 4.25 Influence of degree of penetration of column: (a) Distribution of shear stress at column-soil interface; (b) Variation of normal stress in column; and (c) Settlement of reinforced ground.

with the increasing value of L_c/H_s. The maximum values of p_c/p_o are 2.595, 3.507, 3.908 and 4.176 for L_c/H_s=0.25, 0.50, 0.75 and 1.0, respectively. The location of maximum value of

p_c/p_o for any value of L_c/H_s=0.25 to 0.75, is almost the same i.e. $0.60L_c$ measured from the top of column, which is the location of neutral plane. This result reveals that despite the identical value for other parameters, only the degree of penetration can change the magnitude of p_c/p_o significantly.

The variation of S_t/H_s with the degree of penetration, L_c/H_s, at radial distances r/a=1, 1.8, 3.4 and 5 are shown in Fig.4.25(c). The reduction of settlement of the improved ground increases significantly and almost linearly with the increasing value of L_c/H_s. From this figure, it is also revealed that although the overall settlement reduces significantly with increasing value of L_c/H_s, the magnitude of differential settlements remain almost the same for any value of L_c/H_s ranging from 0.25 to 1.0. At r/a=1.0, the values of S_t/H_s are 0.0605 to 0.0076 while at r/a =5.0, these values are 0.073 to 0.024 for the value of L_c/H_s ranging from 0.25 to 1.0. The maximum differential settlements are 0.013 to 0.016 for L_c/H_s increasing from 0.25 to 1.0. This result reveals that as the value of L_c/H_s does not play any role for the minimization of differential settlement, the higher value of L_c/H_s is desirable considering its effectiveness in reducing the overall settlement of the column-reinforced ground.

4.8.4 Relative stiffness of column and soil

The relative stiffness of column and soil (modular ratio i.e. ratio of deformation modulus of column to that of soil), is an important parameter which controls the improvement of soft ground reinforced by columnar inclusions. Its variation influences the distribution of shear stress, the sharing of stress by column and the settlement of the reinforced ground. As the column gets stronger and stiffer, the interaction between the column and the surrounding soil increases resulting in the mobilization of higher shear stress, higher stress concentration in the column and greater reduction of overall settlement of the reinforced ground. Results are obtained for the foundation system having the values of parameters as p_o/E_{so}=0.10, L_c/d_c=10,

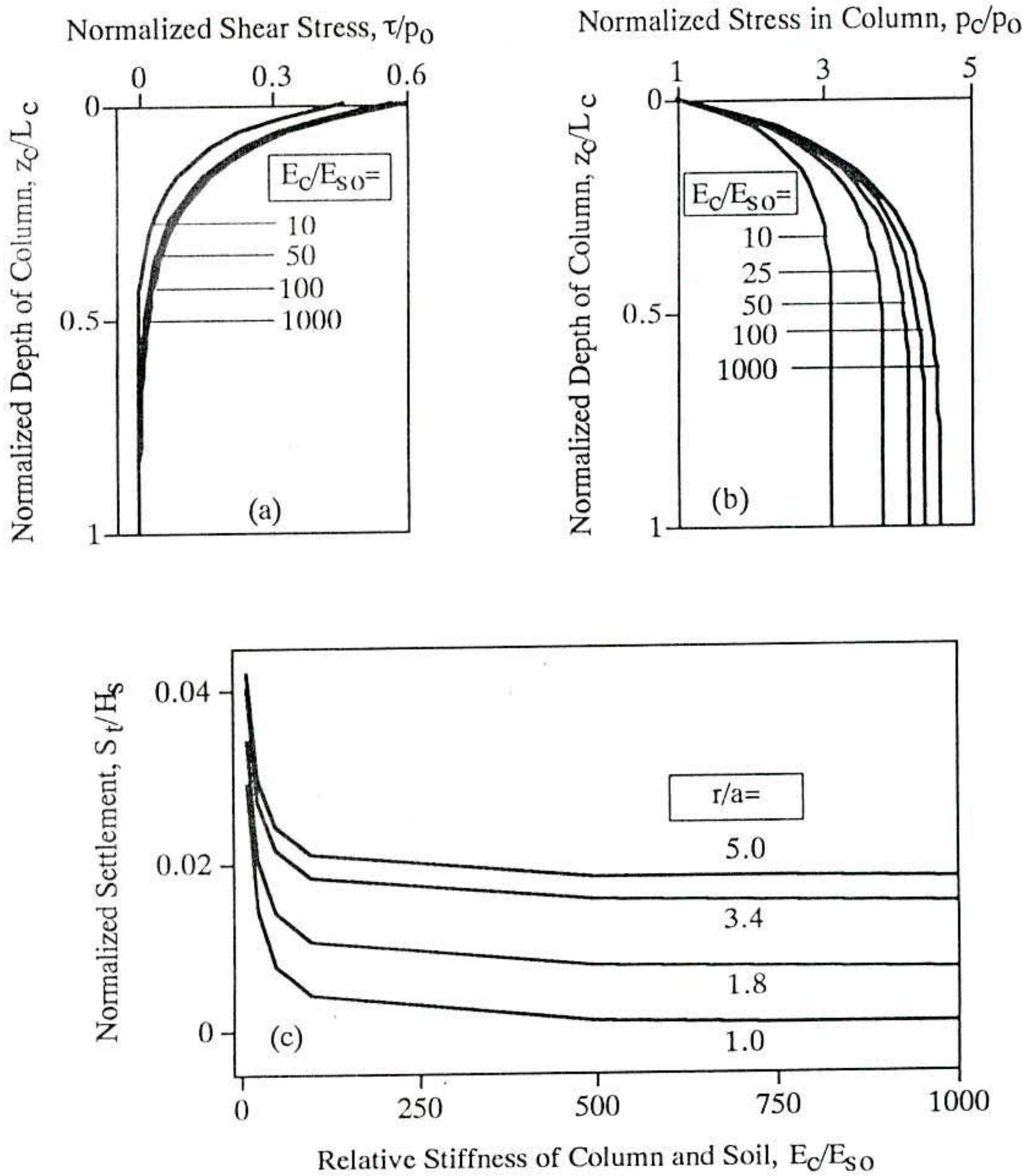


Figure 4.26 Influence of relative stiffness of column and soil: (a) Distribution of shear stress at column-soil interface; (b) Variation of normal stress in column; and (c) Settlement of reinforced ground.

d_e/d_c=5.0, L_c/H_s=1.0, m_s/E_{so}=0.10 and ν_s=0.40. The values of E_c/E_{so} are considered to vary from 10 to 1000. A wide range of E_c/E_{so} i.e. E_c/E_s=10 to 1000, is taken into account because the columnar inclusions range from comparatively softer sand column to relatively stiffer lime or cement columns. The results are presented in Figs.4.26(a), (b) and (c) to show the distribution of shear stress, the variation of normal stress in column and the settlement of the treated ground, respectively. Figure 4.26(a) shows that the distribution of shear stress along the column-soil interface increases with the increase of modular ratio. The normalized shear stress, τ/p_o, varies from 0.457 to 0.599 at the top, and 0.045 to 0.095 at z_c/L_c=0.25 for E_c/E_{so} increasing from 10 to 1000. This figure also indicates that the influence of modular ratio on the rate of increase of shear stress is more for lower values of modular ratio than that of higher values of modular ratio. Figure 4.26(b) shows the influence of modular ratio on the variation of normal stress in column along the depth. The value of p_c/p_o increases with the increase of E_c/E_{so}. At the bottom of the column the values of p_c/p_o are 2.98 to 4.60 for the variation of E_c/E_{so} from 10 to 1000. The result reveals that for 100 times increase of modular ratio, the increase of τ/p_o and p_c/p_o are only 1.31 and 1.54 times, respectively. The properties of soil, not the modular ratio, control the magnitudes of shear stresses at column-soil interface, which, in turn, transfer the stress from soil to column.

The reduction of settlement of the reinforced ground with the variation of E_c/E_{so} at radial distances r/a=1, 1.8, 3.4 and 5.0 are plotted in Fig.4.26(c). The overall settlement of the reinforced ground decreases and the differential settlement increases significantly with the increase of modular ratio. These changes are found more pronounced at the lower range of modular ratio i.e E_c/E_{so}=10 to 100, than that of at the higher range of modular ratio i.e. E_c/E_{so}=100 to 1000. This figure shows that the maximum differential settlement increases from 0.013 to 0.017 and 0.017 to 0.0175 for the increase of E_c/E_{so}=10 to 100 and 100 to 1000 respectively. For the same ranges of E_c/E_{so}, the normalized settlements S_t/H_s at r/a=3.4 reduces from 0.04 to 0.018 and 0.018 to 0.015 respectively. These findings reveal that the relative stiffness of column more than 100 does not play any significant role in the reduction of overall settlement of reinforced ground.

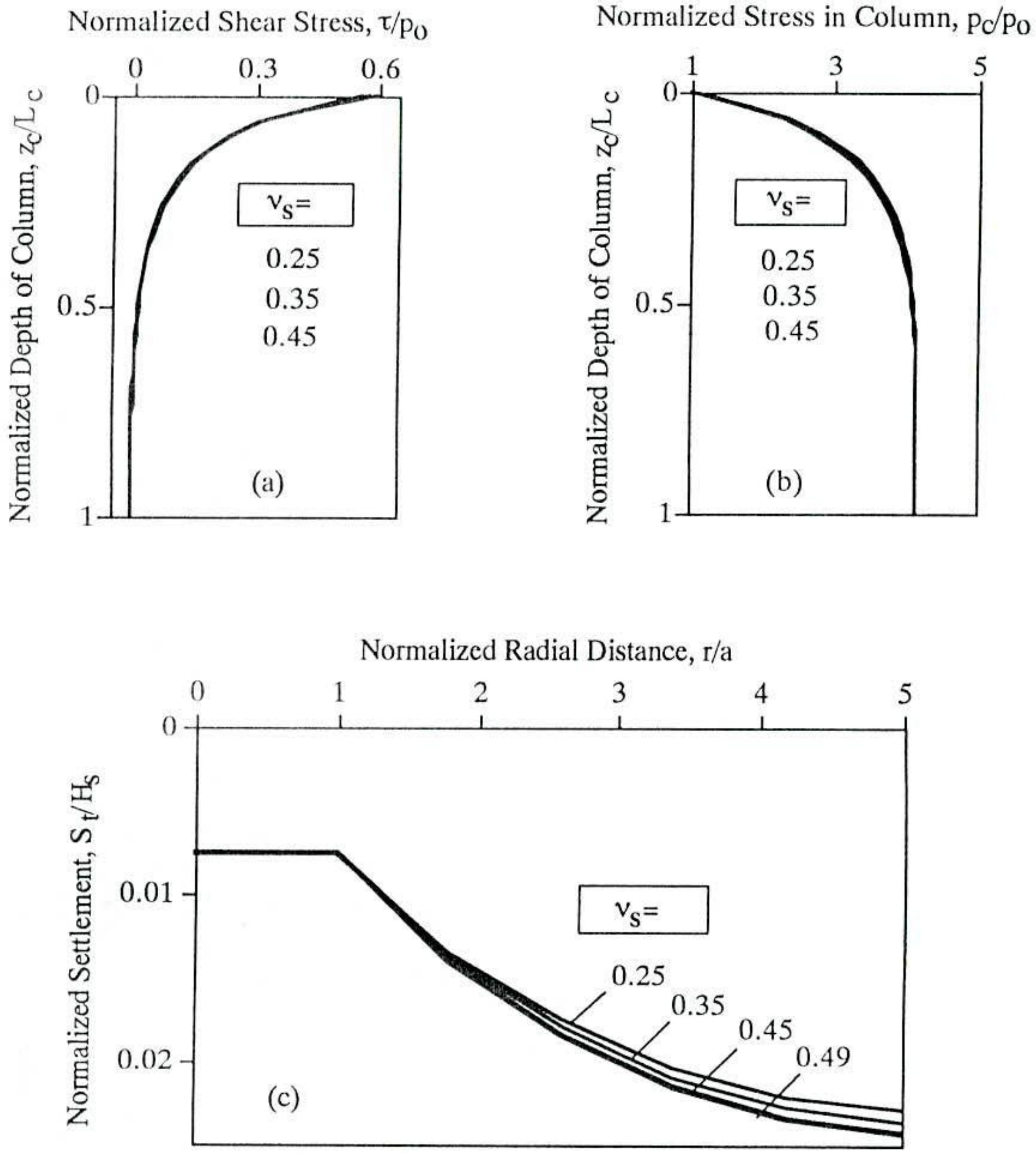


Figure 4.27 Influence of Poisson's ratio of soil: (a) Distribution of shear stress at column-soil interface; (b) variation of normal stress in column; and (c) Settlement of reinforced ground.

4.8.5 Poisson's ratio of soil

The effect of Poisson's ratio of soil on the settlement response of soft ground reinforced by columnar inclusions is presented in this section. The evaluations are performed for the foundation system having the values of parameters as p_o/E_{so}=0.10, L_c/d_c=10, d_e/d_c=5.0, L_c/H_s=1.0, E_c/E_{so}=50 and m_s/E_{so}=0.10. Results are obtained for ν_s varying from 0.25 to 0.49. The mobilization of shear stress, the variation of normal stress in column and the settlement of the reinforced ground are plotted in Figs.4.27(a), (b) and (c), respectively. These results reveal that the variation of Poisson's ratio of soil has little influence on the predicted values of τ/p_o, p_c/p_o and S_t/H_s. The result from Fig.4.27(c) shows that the value of S_t/H_s remains almost the same in the column region i.e. at $0 \le r/a \le 1$. But it increases slightly, from 0.023 to 0.0245, in the outer boundary of the zone of influence i.e. at r/a=5.0, for the increase of Poisson's ratio of soil from 0.25 to 0.49.

4.8.6 Angle of friction between column and soil

The limiting shear stress at column-soil interface depends on the value of angle of friction between column and soil, δ. The limiting shear stress increases with the increasing value of δ. If slip occurs at soil-soil interface, the value of δ is equal to ϕ, angle of friction of soil. But if slip occurs others than soil-soil interface, the value of δ may differ from ϕ. To show the effects of δ on the predictions, the value of δ is considered to vary from 0.5ϕ to ϕ, while the value of ϕ is 30°. In this evaluation, the values of others parameters are considered as p_o/E_{so}=0.10, L_c/d_c=10, d_e/d_c=5.0, L_c/H_s=1.0, E_c/E_{so}=50, m_s/E_{so}=0.10 and ν_s=0.40. The influence of δ on the distribution of shear stress at column-soil interface, variation of normal stress in column and the settlement of improved ground are shown in Figs.4.28(a), (b) and (c), respectively. The distribution of normalized shear stress, τ/p_o, along the depth of column is

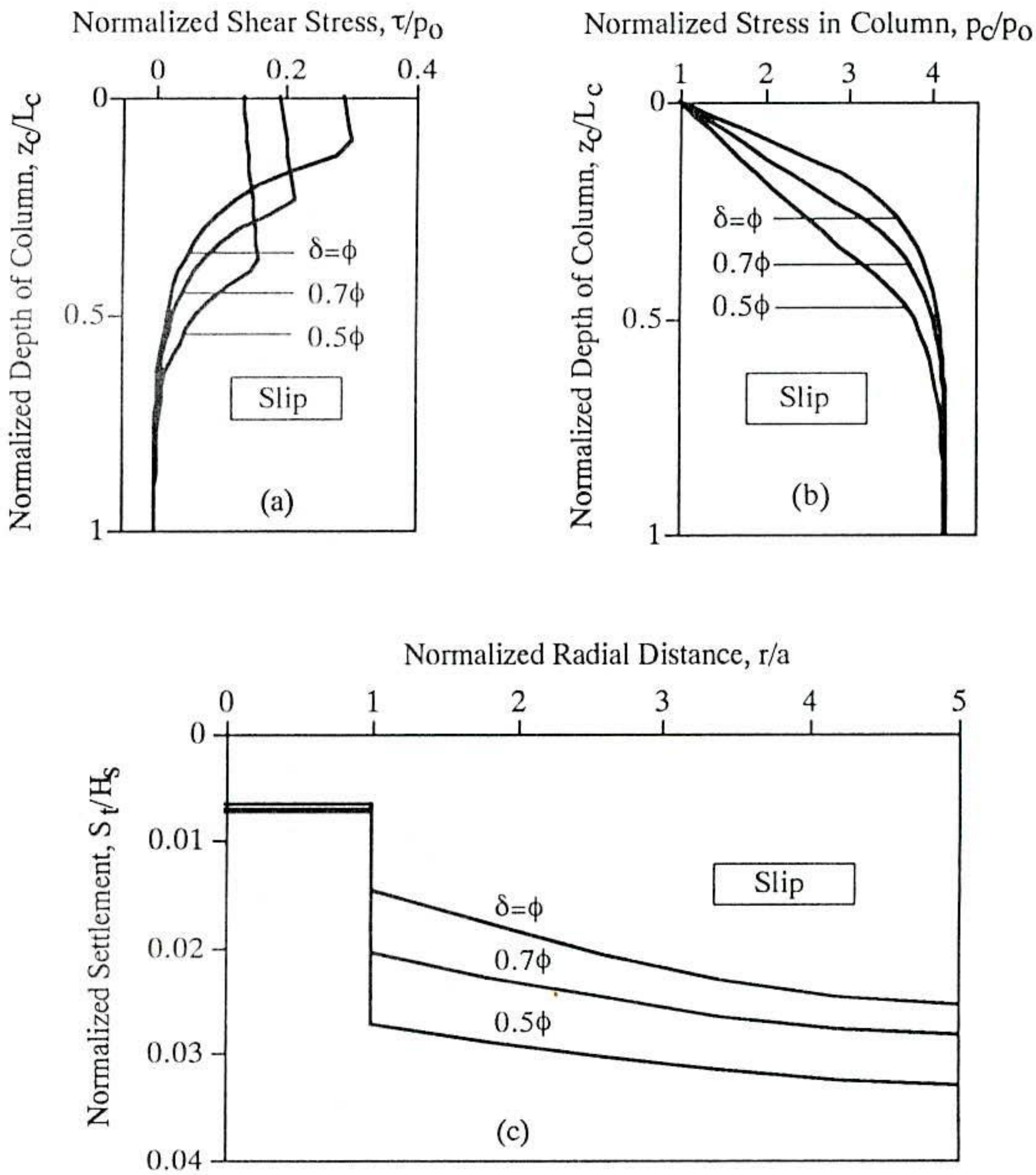


Figure 4.28 Influence of angle of friction between column and soil: (a) Distribution of shear stress at column-soil interface; (b) Variation of normal stress in column; and (c) Settlement profile of reinforced ground.

shown in Fig.4.28(a) for $\delta=0.5\phi$, 0.7ϕ and ϕ. The shear stres increases with the increasing value of δ. At the top of column i.e. at $z_c/L_c=0.0$, the value of $\tau/p_o=0.134$, 0.192 and 0.289

for the value of $\delta=0.5\phi$, 0.7ϕ and ϕ, respectively. The depth of slip zone increases with the decreasing value of δ. It is found as 0.10, 0.23 and $0.37L_c$ measured from the top of column for the value of $\delta=\phi$, 0.7ϕ and 0.5ϕ, respectively. The magnitude of shear stress differs at the upper portion of column but in the lower part it becomes almost same and tends to zero at bedrock level. Figure 4.28(b) shows that the value of normalized vertical stress in column, p_c/p_o, along the depth increases with the increasing value of δ. At a depth $z_c/L_c=0.25$, the value of $p_c/p_o=2.31$, 2.88 and 3.43 for the value of $\delta=0.5\phi$, 0.7ϕ and ϕ, respectively. As the limiting shear stress increases with δ, the amount of stress transfer from soil to column also increases. However, at the lower portion, the value of p_c/p_o becomes almost the same for any value of δ.

The settlement profile of the reinforced ground in slip condition for different values of δ is shown in Fig.4.28(c). The overall and the differential settlements increase with the decreasing value of δ. As the limiting shear stress at column-soil interface decreases, the amount of load transfer to the stiffer column gets reduced. As a result, soil settles more but column settles less with the decreasing value of δ. In the column region, $0\leq r/a\leq 1$, the values of normalized settlement of the reinforced ground, S_t/H_s, are 0.0066, 0.007 and 0.0074 for the value of $\delta=0.5\phi$, 0.7ϕ and ϕ, respectively. But at $r/a=1.0$, in soil region, theses values are 0.0274, 0.21 and 0.146, respectively. At $r/a=n=5$, the value of $S_t/H_s=0.0313$, 0.027 and 0.0245 for $\delta=0.5\phi$, 0.7ϕ and ϕ, respectively. From these findings, it can be seen that the differential settlement influenced significantly with the value of δ i.e. angle of friction between column and soil. Therefore, for the rational design of column-reinforced ground, it should be identified whether the slip take place at column-soil or soil-soil interface.

4.9 Conclusions

The application of the formulations and the numerical scheme, developed in the Chapter Three as a general approach for the solution of an important class of problems in the field of geotechnical engineering, is presented here in detail. These are used in solving the behaviour of soft ground reinforced by one of the most common practice of ground improvement, namely, columnar inclusions. The reinforced ground is covered by a layer of granular fill. This foundation system is idealized and the formulations and numerical scheme developed in Chapter Three, is used successfully without any modifications. The predictions are presented and discussed for end bearing and floating columns, installed in a group in both nonstratified and stratified soil layer systems. The analyses are performed for no slip and possible slip situations at column-soil interface. The compressibility of granular fill placed over the reinforced ground is also considered for the rational assessment of the contribution from overlaying granular fill. The results are presented to depict the effect of granular fill thickness and its modulus on the overall response of column reinforced ground. The distribution of interface shear stress, the variation of normal stresses in column and surrounding soil and the settlement profiles of the column-reinforced ground are presented. Evaluations are made for a wide range of values of various design parameters to illustrate their influences on the results.

From the predicted results, it is revealed that the proposed foundation model can be used successfully to evaluate the behaviour of soft ground reinforced by columnar inclusions. The results reveal that the settlement response of column reinforced ground is greatly influenced at the presence of granular fill over it. The response of reinforced ground ranges from flexible to rigid loading conditions depending on the magnitudes of thickness and deformation modulus of overlaying granular fill. The compacted granular fill over the column-reinforced ground is very effective in reducing both the overall and the differential settlements of the loaded composite ground. The compressibility of the granular fill has an appreciable influence on the settlement response of the composite ground as long as the modulus of the granular fill is less than approximately fifty times that of the soft ground. The stress in column increases with the increasing value of the stiffness of overlaying granular fill.

From the results of the present analysis, it appears that the length of slip may be somewhat smaller than previously thought. This observation appears to be in agreement with the test results reported by Kennan and Bozozuk (1985). The depth of slip decreases with the increase of degree of penetration of column. It is also seen that slip depth increases with the decreasing value of limiting shear stress at the column-soil interface. The result shows that the neutral depth is not influenced by the degree of penetration of column. But it depends on the no slip and slip situations at the column-soil interface. Slip case predicts higher value of neutral depth than its no slip counterpart. The result shows that the presence of a strong soil layer at the tip of the column would have a significant influence. This is, of course, as expected; the present analysis quantifies it. Another point worthy of mention is that a good portion of column sustains little or no interface shear stress at all for end bearing column. But for floating column, the whole length of column is subjected to shear stress either positive (i.e. downwards) or negative (i.e. upwards).

The stresses in column and the surrounding soil are unity at the surface after that it changes with depth and radial distance. For end bearing column, the stress in column increases with depth and attains a high value at the bottom. For floating column, it increases up to the depth of neutral plane beyond which decreases up to the bottom of column. In both cases, no slip situation gives higher value than that of possible slip. For the case of stratified soil systems, the stress in column decreases with the change of its location from upper soft soil layer to relatively stiffer lower soil layer. In case of end bearing column, the stress in soil decreases with depth and obtains minimum value at the base. But for floating case, it decreases up to a certain depth beyond which increases and tends to be unity at the base. The influence of possible slip at column-soil interface and the soil stratification are also evident in this case.

The settlement profile of the reinforced ground shows that the differential settlement is noticeable for the cases of uniform flexible loading acting over the entire reinforced area. Settlement profiles are different for the situations of no slip and possible slip at the column-soil interface. The overall and the differential settlements are more for slip analysis than that of no slip case. These values are also increased with the decreasing value of limiting shear stress at the column-soil interface. The influence of soil stratification is evident. But the reduction of settlement due to presence of stiffer lower soil layer indicates that the role of upper soil layer is more pronounced than that of lower soil layer. As the differential settlement does not reduced in case of floating column, end bearing column is more effective due to giving less overall settlement. Parametric study shows that the proposed approach can be used successfully to demonstrate the effects of the variations of various parameters in the working range, on the overall response of the soft ground reinforced by columnar inclusions. From the predicted results, for the variation of wide range of parameters, it is observed that the value of parameters such as (i) spacing of columns, (ii) length to diameter ratio of columns, (iii) degree of penetration of columns, (iv) relative stiffness of column and soil, and (v) angle of friction between column and soil, have a significant influence on the mobilization of shear stress, variation of stresses in column and soil and the settlement profiles of the treated ground.

CHAPTER FIVE

TIME-DEPENDENT RESPONSE

5.1 General

The time-dependent response of soft ground reinforced by columnar inclusions is presented in this chapter. The columnar inclusions such as stone columns/granular piles and sand compaction piles installed in soft clay deposits act as reinforcing elements and as drains. They act similar to sand drains to accelerate the rate of consolidation by decreasing the distance over which the pore water has to flow in the radial direction for primary consolidation to occur. The fact that the horizontal permeability of soil generally exceeds the vertical permeability adds to the effectiveness of the vertical drains. As a result of installation, columns can, in the absence of natural drainage layers within cohesive soils, significantly decrease the time required for primary consolidation.

The foundation system described in Chapter Four is considered for the present time-dependent analysis. The "Diffusion Theory" which is an extension of Terzaghi's one dimensional consolidation theory (Terzaghi 1925) is used to determine the dissipation of excess pore water pressure due to radial and vertical flow of water. The finite difference numerical technique is employed to solve the governing equations. The excess pore water pressure is evaluated at every nodal points for an elapsed time. The overall time-dependent response of the column-reinforced soft ground is then predicted by solving the governing equations developed column-reinforced soft ground is then predicted by solving the governing equations developed in Chapter Three in conjuction with the predicted uncoupled excess pore water pressures. The solution is also given for a stratified soil systems. The results are presented to illustrate the dissipatation of excess pore water pressure, the mobilization of shear stresses at the column-soil interface, the load sharing between the components of the system i.e. the column and the surrounding soil and the settlement profile of the reinforced ground with time. Evaluations are made varing the values of horizontal to vertical permeability ratio of soil and the spacing of columns. The predicted results are discussed in the light of field applications of the installation of columnar inclusions to improve the soft clay deposits and to accelerate the time rate of settlement.

5.2 Time-Dependent Response Function of the System

The consolidation process is a combination of two phenomenon: the permeability which controls the rate of flow, and the compressibility, which controls the evolution of the consolidation process (Leroueil 1988). For the prediction of the rate of settlement of column-reinforced soft ground, a solution is required to determine the vertical deformation of clay which is due to the expulsion of excess pore water by vertical and radial flow. The most rigorous theory of three dimensional consolidation is the Biot's theory of consolidation (Biot 1941) which combines the effects of diffusion and the elastic deformations resulting from the decrease of pore water pressure. Because of the complexity of this theory, few analytical solutions are available and numerical methods of solution are usually adopted (Sandhu & Wilson 1969, Christian & Boehmer 1970, Hwang et al. 1971 and Booker 1973). Numerical solution of Biot's consolidation theory involves more computational efforts than diffusion solution. The latter provides a satisfactory approximation to the values of degree of consolidation settlement (Balaam et al. 1977). Moreover, the difference between the predictions by the "Diffision Theory" and the Biot's three dimensional consolidation theory decreases with the increase of elapsed time. The "Diffusion Theory" is a pseudo-three dimensional consolidation theory. In this study, an uncoupled solution of consolidation of soft soil surrounding columnar inclusions, is presented to predict the time-dependent response of soft ground reinforced by columnar inclusions. The "Diffusion Theory" is used to predict the excess pore water pressure at an elapsed time without considering the variation of stress concentration in the column with time. The response of the reinforced ground is evaluated by using the formualtions developed in Chapter Three in conjunction with the excess pore water pressures determined by the uncoupled solution.

5.2.1 Governing equations

The expression for vertical stress σ_{zz} at time t>0, can be given by using Terzaghi's effective stress principle (Terzaghi 1943) as:

$$\sigma_{zz} = \sigma'_{zz} + u \tag{5.1}$$

where σ'_{zz} and u are the effective stress and the excess pore water pressure at time t>0, respectively. Using the above equation, the stress-strain equation, Eq.3.6, can be rewritten as

$$\frac{\partial w(r,z)}{\partial z} = \frac{1}{E(r,z)}\left[\sigma_{zz} - u\right] \tag{5.2}$$

in terms of degree of consolidation, one can write the above equation as

$$\sigma_{zz} = \frac{\partial w(r,z)}{\partial z}\frac{E_s(r,z)}{U(r,z)} \tag{5.3}$$

where $U(r,z)$ is the degree of consolidation of soil at a point (r,z) at time t>0, which can be expressed by the following relation

$$U(r,z) = 1 - \frac{u(r,z)}{u_o} \tag{5.4}$$

where $u(r,z)$ is excess pore water pressure of soil at a point (r,z) at any time $t>0$. u_o is the intial excess pore water pressure developed immediately after the application of load. With the aid of above equation, one can rewrite the Eq.(3.7) as

$$\frac{\partial w(r,z)}{\partial z}\frac{E_s(r,z)}{U(r,z)} = -\int_0^z \left[\frac{\partial}{\partial r}\left\{g(r,\xi)\frac{\partial w(r,\xi)}{\partial r}\right\} + \frac{1}{r}\left\{g(r,\xi)\frac{\partial w(r,\xi)}{\partial r}\right\}\right]dz + p_o \tag{5.5}$$

After simplification, one can express Eq.(5.5) in the following form

$$\frac{\partial w(r,z)}{\partial z}\frac{E_s(r,z)}{U(r,z)} + \int_0^z \left[G(\mathrm{r},\xi)\left\{\frac{\partial^2 w(r,\xi)}{\partial r^2} + \frac{1}{r}\frac{\partial w(r,\xi)}{\partial r}\right\} + \frac{\partial w(r,z)}{\partial r}\frac{\partial G(r,\xi)}{\partial r}\right] = p_o \tag{5.6}$$

The above equation can be expressed as

$$\frac{\partial w(r,z)}{\partial z} + \frac{U(r,z)}{E_s(r,z)}\int_0^z \left[G(\mathrm{r},\xi)\left\{\frac{\partial^2 w(r,\xi)}{\partial r^2} + \frac{1}{r}\frac{\partial w(r,\xi)}{\partial r}\right\} + G'(r,\xi)\frac{\partial w(r,z)}{\partial r}\right] = \frac{U(r,z)p_o}{E_s(r,z)} \tag{5.7}$$

where $G'(r,\xi)$ is the first order derivative of $G(r,\xi)$ with respect to r i.e. $G'(r,\xi) = \dfrac{\partial G(r,\xi)}{\partial r}$.

Eq.(5.7) is the desired equation to solve the time-dependent response of column-reinforced soft ground. To solve this equation, it is necessary to evaluate the value of $U(r,z)$ at time $t>0$.

The differential equation of two dimensional flow, applicable to vertical drain problems, can be written in terms of cylindrical coordinates as

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} + C_h \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) \tag{5.8}$$

where C_v is the coefficient of consolidation in the vertical direction for one dimensional strain conditions; C_h is the coefficient of consolidation in the radial direction also for one dimensional strain conditions; u is the hydrostatic excess pore water pressure; r and z are the radial and vertical coordinates, respectively and t is the elapsed time measured from the application of load. The derivation and analytical solution of this equation has been reviewed by Barron (1948). This is an extension of the Terzaghi's theory of one dimensional consolidation (Terzaghi 1925). This equation combines the one dimensional vertical flow solution as defined by Terzaghi (1943) and the radial flow solution as developed by Rendulic (1935). Carrillo (1942) has shown that the above equation can be solved as two separate parts and combined to give the complete solution. Therefore, the total flow expressed in Eq.(5.1) can be resolved into plane radial flow

$$\frac{\partial u_h}{\partial t} = C_h \left(\frac{\partial u_h}{\partial r^2} + \frac{1}{r} \frac{\partial u_h}{\partial r} \right) \tag{5.9}$$

and into linear vertical flow

$$\frac{\partial u_v}{\partial t} = C_v \frac{\partial^2 u_v}{\partial z^2} \tag{5.10}$$

where u_h is the excess pore water pressure at time t>0 due to radial flow only and u_v is the excess pore water pressure at time t>0 due to vertical flow only. By combining the Eqs.(5.9) and (5.10), the equation to obtain the degree of consolidation due to radial and vertical flow of water, can be expressed as

$$U(r,z) = 1 - \left(\frac{u_h \, u_v}{u_o} \right) \tag{5.11}$$

where $U(r,z)$ is the degree of consolidation at a point (r,z), at any time $t>0$. These equations must be solved in conjunction with the relevant boundary conditions which are stated in the following section.

5.2.2 Boundary conditions

The following initial and boundary conditions of the problem provide extra number of equations needed for a unique solutions.

(i) Initial condition: The initial excess pore water pressure, u_o is uniform throughout the soil mass for time, $t=0$ and it is equal to the load applied over the surface of reinforced ground i.e. $u_o=p_o$.

(ii) Boundary condition at the top of clay layer: Since the top of soft ground reinforced by columnar inclusions is generally covered by a drainage layer, therefore, the top surface of clay layer is considered as drainage free i.e. $u=0$ at $z_c=0$ for any time, $t>0$.

(iii) Boundary condition at the base: The lower horizontal boundary of the consolidated soil mass is impervious or, because of symmetry, no flow occurs across this boundary. Therefore, $\delta u/\delta z=0$ at $z_c=H_s$ for any time, $t>0$.

(iv) Boundary condition at the outside boundary of influence zone: The outside boundary of the influence zone is considered impervious because of symmetry of load and geometry. Thus $\delta u/\delta r=0$ at $r=d_e/2$ for any time, $t>0$.

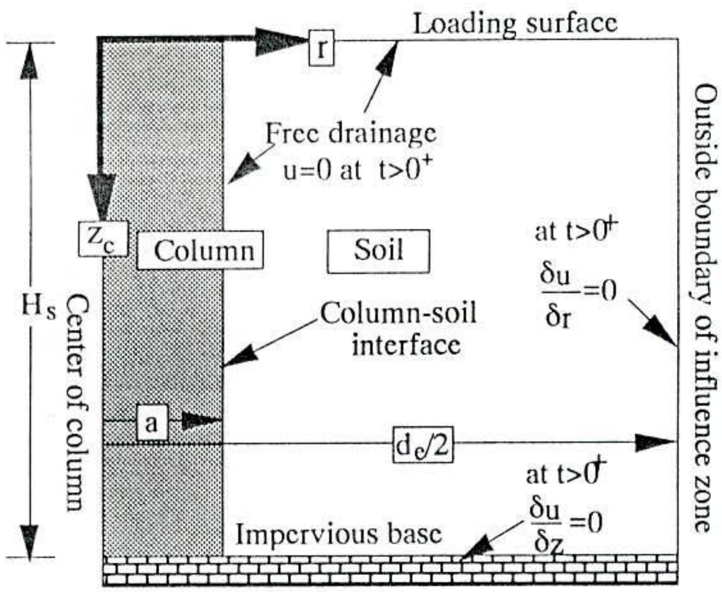


Figure 5.1 Boundary conditions of the problem used for consolidation analysis.

(v) Boundary condition at the column-soil interface: The permeability of the column is very high compared to the material which must be drained. Due to this high permeability, the excess pore water pressure will be dissipated immediately by the free flow of water through the drain. Hence, for practical purposes, the excess pore pressure is equals to zero at every point within the column and in particular along the radius of column. Therefore, $u=0$ at $r=a$ for any time, $t>0$.

5.2.3 Method of solution

The above equations can be solved analytically to obtain the value of degree of consolidation for the case of equal vertical strain and free strain. But a finite difference solution is found to be simpler and reasonably accurate specially when compared with the analytical solution for free strain case and for the stratified soil deposits. Finite difference solution have been given by Richart (1957). The general finite difference form of the governing differential equation, Eq.(5.8), can be written as

$$u^{t+\Delta t}(i,j) = u^t(i,j) + \frac{C_v \Delta t}{(\Delta z)^2}\left[u^t(i-1,j) - 2u^t(i,j) + u^t(i+1,j)\right] + \frac{C_h \Delta t}{(\Delta r)^2}\left[u^t(i,j-1) - 2u^t(i,j) + u^t(i,j+1)\right] + \frac{C_h \Delta t}{2r\Delta r}\left[u^t(i,j+1) - u^t(i,j-1)\right] \tag{5.12}$$

In finite difference form, the differential equation representing the boundary condition (iii) i.e. at the base, can be written as,

$$\frac{u^t(imax+1,j) - u^t(imax-1,j)}{2(\Delta z)} = 0 \qquad i.e. \quad u^t(imax+1,j) = u^t(imax-1,j) \tag{5.13}$$

The boundary conditions (iv) i.e. at the outside boundary of the zone of influence, can be expressed as

$$\frac{u^t(i,jmax+1) - u^t(i,jmax-1)}{2(\Delta r)} = 0 \qquad i.e. \quad u^t(i,jmax+1) = u^t(i,jmax-1) \tag{5.14}$$

where the superscripts t refer to the time level; Δt is the time interval and (i,j) is referred as a general node in the finite difference mesh. Since the predicted values of $U(i,j)$ will be used to solve the time-dependent response of reinforced ground, the same finite difference network, shown in Fig.3.6 is used for the predictions of $U(i,j)$. These equations, Eqs.5.12 to 5.14, can be solved with a high degree of accuracy to obtain the excess pore preşsure, u i.e. $u\ (i,j)$ and the degree of consolidation, $U(r,z)$ i.e. $U(i,j)$, at any time $t>0$ at a nodal point (i,j).

The consolidation process stated above is related to the nonstratified clay deposits. For the case of stratified soil i.e. a clay stratum, made up two or more horizontal layers, special care must be taken at the interface of the layers. At the interface between the two layers the conditions of equilibrium require that the velocity of flow leaving one layer must be equal to the velocity of flow entering the other (Richart 1957). Thus for two materials having coefficients of permeability of k_{v1} and k_{v2}, the following condition requires to be satisfied at the interface of two soil layers:

$$k_{v1}\frac{\delta u_v}{\delta z_1} = k_{v2}\frac{\delta u_v}{\delta z_2} \tag{5.15}$$

It is not always possible to develop a closed-form solution for consolidation in layered soil systems (Das 1983). There are several variables involved, such as different coefficients of permeability, the thicknesses of layers, and different values of coefficients of consolidation. In view of the above, numerical solutions provide a better approach. To calculate the excess pore water pressure at the interface of two layers (i.e. different values of C_v) of clayey soils, the first part of Eq.(5.12) will have to be modified to some extend. Based on the solutions given by Scott (1963), this modification can be expressed as:

$$\begin{aligned} u^{t+\Delta t}(i,j) = & \frac{1+\dfrac{k_{v1}}{k_{v2}}}{\dfrac{C_{v1}}{C_{v2}}+\dfrac{k_{v1}}{k_{v2}}}\frac{\Delta t}{(\Delta z)^2}\left[\frac{2}{1+\dfrac{k_{v2}}{k_{v1}}}u^t(i-1,j) - 2u^t(i,j) + \frac{2}{1+\dfrac{k_{v1}}{k_{v2}}}u^t(i+1,j)\right] + \\ & \frac{(C_{h1}+C_{h2})\Delta t}{2(\Delta r)^2}\left[u^t(i,j-1) - 2u^t(i,j) + u^t(i,j+1)\right] + \\ & \frac{(C_{h1}+C_{h2})\Delta t}{4r\Delta r}\left[u^t(i,j+1) - u^t(i,j-1)\right] + u^t(i,j) \end{aligned} \tag{5.16}$$

where *(i,j)* represents the node at the interface; the digit 1 and 2 used in the subscript of the parameters k_v, C_v and C_h represent the upper and the lower soil layers, respectively. The finite difference form of the Eq.(5.7) can be written as

$$w'(i+1,j) - w'(i,j) + \sum_{m=1}^{i} \alpha(m,j)\left[w'(m,j-1) - 2w'(m,j) + w'(m,j+1)\right] + \\ \sum_{m=1}^{i} \left[\beta(m,j) + \eta(m,j)\right]\left[w'(m,j+1) - w'(m,j-1)\right] = \frac{U'(i,j)p_o}{E_s(i,j)}\Delta z \tag{5.17}$$

where $\alpha(m,j) = \dfrac{U(i,j)G(m-1,j)(\Delta z)^2}{E_s(i,j)(\Delta r)^2}$, $\beta(m,j) = \dfrac{U(i,j)G(m-1,j)(\Delta z)^2}{E_s(i,j)2r(\Delta r)}$ and

$\eta(m,j) = \dfrac{U(i,j)G'(m-1,j)(\Delta z)^2}{E_s(i,j)(2\Delta r)}$. Eq.(5.17) is solved for any time $t>0$ to obtain the response of reinforced ground as described in Chapter Three for the solution of time-independent problem.

5.3 Dissipation of Pore Water Pressure

The dissipation of excess pore water pressure due to vertical and radial flow of water in the soft clay deposits reinforced by columnar inclusions is presented here. The results are presented in the following sections to illustrate the influences of the variation of horizontal to vertical permeability ratio of soil and the spacing of columns. The foundation system, considered in Chapter Four, is used here for the time-dependent analysis also. Although, the proposed time dependent model can handle the radial and vertical inhomogeneity of the material properties i.e. variations of E_s and G with z and r, in the following predictions these parameters are considered to vary with depth z, only. The results are presented in nondimensional form.

5.3.1 Horizontal to vertical permeability ratio of soil

The influence of the variation of horizontal to vertical permeability ratio of soil on the dissipation of excess pore water pressure and variation of degree of consolidation are presented in Figs.5.2 and 5.3, respectively. For these predictions the values of the parameters considered are L_c/d_c=10, d_e/d_c=5, p_o/E_{so}=0.10. To illustrate the influence of horizontal to vertical permeability ratio of soil, k_h/k_v, the value of C_h/C_v, the ratio of horizontal to vertical coefficient of consolidation of soil, is considered to vary from 1.0 to 10. The distribution of the normalized excess pore water pressure, u/p_o, with radial distance, r/a, at an elapsed time T_v=0.01, is presented in Fig.5.2. The elapsed time t is normalized here as $T_v=[(C_h/H_s^2)*t]$, where T_v is termed as time factor in the consolidation analysis. The value of u/p_o is evaluated

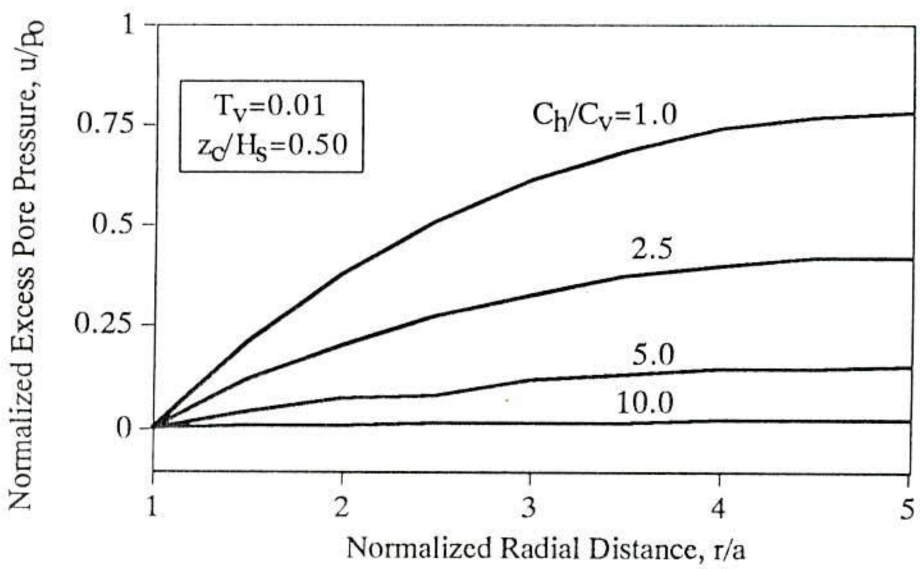


Figure 5.2 Influence of horizontal to vertical permeability ratio of soil on the dissipation of excess pore water pressure along the radial distance.

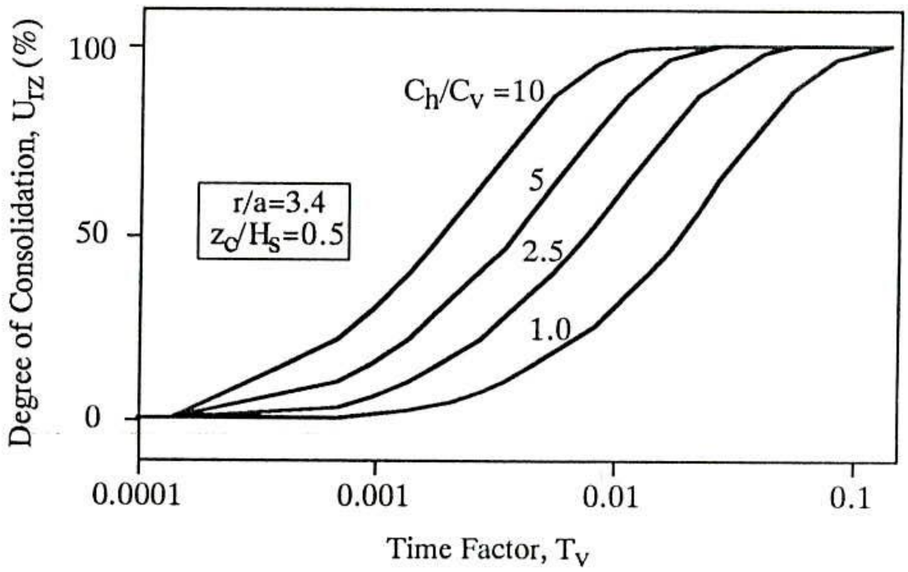


Figure 5.3 Influence of horizontal to vertical permeability ratio of soil on the distribution of degree of consolidation with time.

at a normalized depth z_c/H_s=0.50. The value of u/p_o increases with r/a, from zero at the column-soil interface i.e. at r/a =1, upto the outside boundary of the zone of influence i.e at $r/a=d_e/d_c=n$ and attains the maximum value at this point. For C_h/C_v=1.0, the value of u/p_o is found to vary from 0.00 at r/a=1.0 to 0.775 at r/a=5.0. The value of u/p_o at a particular radial distance r/a, decreases with the increase of C_h/C_v. From this figure, it is found that at r/a=3.4, the value of u/p_o decreases from 0.683 to 0.016 for the increase of C_h/C_v from 1.0 to 10. The variation of degree of consolidation $U(r,z)$, with time factor T_v, at a radial distance r/a=3.5, are presented in Fig.5.3 for the different values of C_h/C_v. The value of $U(r,z)$ increases with time and becomes 100% after a particular level of time has been elapsed. The time required for 100% consolidation decreases significantly with the increase of C_h/C_v. At an elapsed time i.e. T_v=0.01, the value of $U(r,z)$ increases from 0.325 to 0.981 for the increase of C_h/C_v from 1.0 to 10. These findings reveal that the value of C_h/C_v has a pronounced influence on the dissipation of excess pore water pressure and hence to decrease the time required for 100% degree of consolidation. The value of $U(r,z)$ depends significantly on the radial distance measured from the column-soil interface towards the outside boundary of the zone of influence.

5.3.2 Spacing of columns

The dissipation of excess pore water pressure is influenced significantly with the spacing of columns. The path for the radial flow of water increases with the increase of column spacing. The time rate of dissipation of excess pore water pressure depends on the length of this path which the water has to flow for dissipation. The distribution of excess pore water pressure and the variations of degree of consolidation for different spacings of columns are shown in Figs.5.4 and 5.5, respectively. The values of parameters are C_h/C_v=5, L_c/d_c=10 and $d_e/d_c(=n)$ varies from 2.5 to 10. In Fig.5.4, the distribution of normalized excess pore water

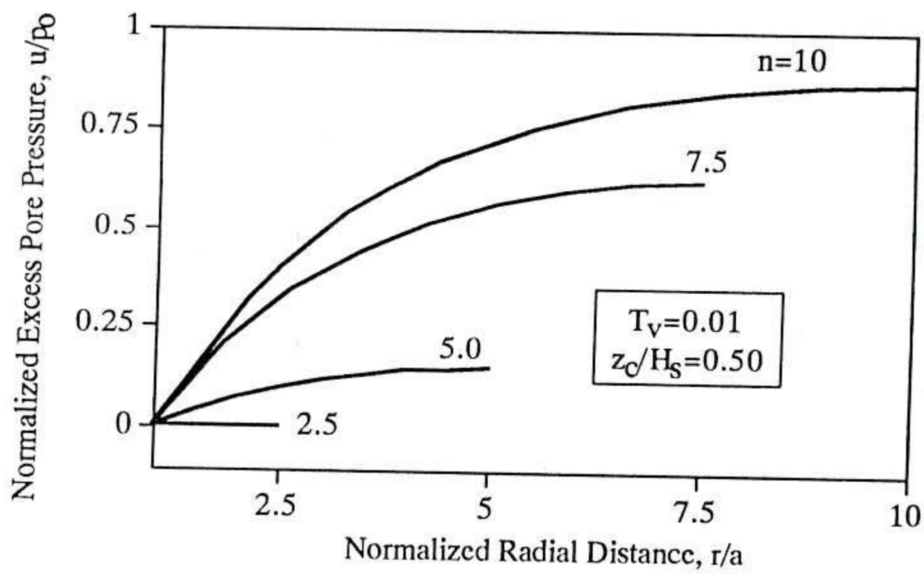


Figure 5.4 Influence of spacing of columns on the dissipation of excess pore water pressure along the radial distance.

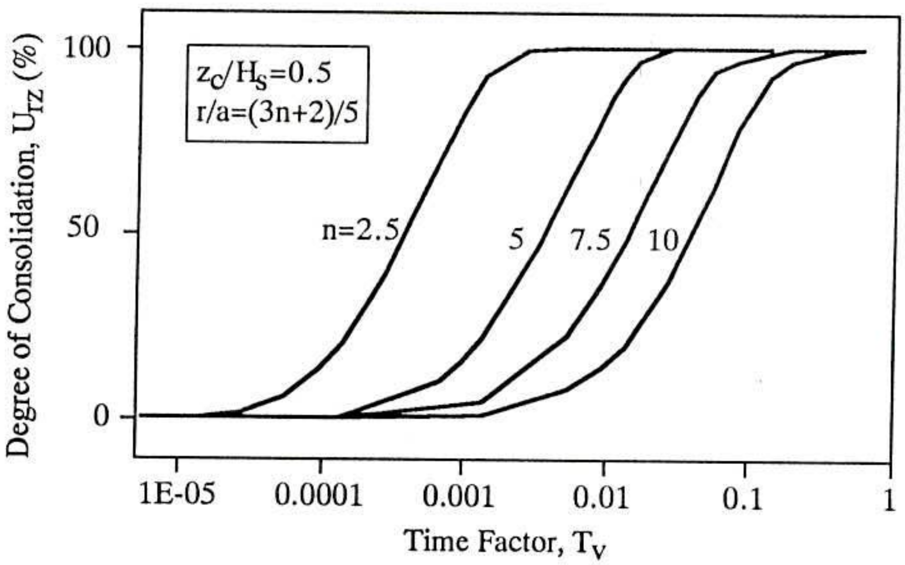


Figure 5.5 Influence of spacing of columns on the distribution of degree of consolidation with time.

pressure, u/p_o, with radial distance, r/a, is presented for the variation of n value increasing from 2.5 to 10. The results are presented here for the elapsed time factor T_v=0.01, measured at a normalized depth z_c/H_s=0.50. The value of u/p_o increases with r/a and decreases significantly with the decreasing value of n. The closer spacing leads to the faster dissipation the excess pore water pressure than that of for higher spacing. From this figure, it is found that after a certain elapsed time i.e. T_v=0.01, the value of u/p_o at $r/a=n$ varies from 0.00 to 0.87 for n value increasin from 2.5 to 10. The variation of degree of consolidation with normalized time for the various spacings of columns is shown in Fig.5.5. At a patricular time, the value of degree of consolidation decreases as the spacing of columns increases. The result shows that the time factor T_v, required for 50% consolidation at radial distance $r/a=(3n+2)/5$ varies from 0.0004 to 0.512 for n=2.5 to 10. The results presented in the foregoing four figures i.e. Figs.5.2 to 5.5 are well established for the case of vertical drains. The dissipation of pore water pressure and hence the degree of consolidation are varied significantly in radial direction.

These figures are presented here for the reason to emphasize the necessity for consideration of vertical and radial variation of degree of consolidation to predict the time-dependent response of column-reinforced ground. The average degree of consolidation does not represent the actual situation that prevails at a particular point of soil mass at an elapsed time. Therefore, it is necessary to use the value of $U(r,z)$, which varies in both vertical and radial directions, rather than the average value of degree of consolidation.

5.4 Settlement Response of Reinforced Ground with Time

The time-dependent response of soft ground reinforced by columnar inclusions, follows the response of consolidation process of soft ground due to the expulsion of excess pore water pressure in the radial and vertical directions. The mobilization of shear stress at the column-soil interface, the load sharing between the components of the system i.e. the column and the surrounding soil and the settlement profile of the improved ground with time are presented in the following sections. The influence of horizontal to vertical permeability ratio of soil and the spacing of columns are considered in the prediction of settlement profiles.

5.4.1 Mobilization of interface shear stress

The mobilization of shear stress at column-soil interface with time is shown in Fig.5.6 for a typical example of column-reinforced soft ground. Results are obtained for the values of parameters p_o/E_{so}=0.10, m_s/E_{so}=0.10, ν_s=0.40, L_c/d_c=10, d_e/d_c=5.0 and C_h/C_v=1.0. The mobilized shear stress at column-soil interface increases as the consolidation proceeds and reaches the maximum value at the end of primary consolidation. At the early stage of loading i.e. for T_v<0.001, the normalized column-soil interface shear stress, τ/p_o, changes its sign from positive (i.e. acting downwards) to negative (i.e. acting upwards) beyond a certain depth from the top of column. But after an elapsed time factor i.e.T_v>0.001, the value

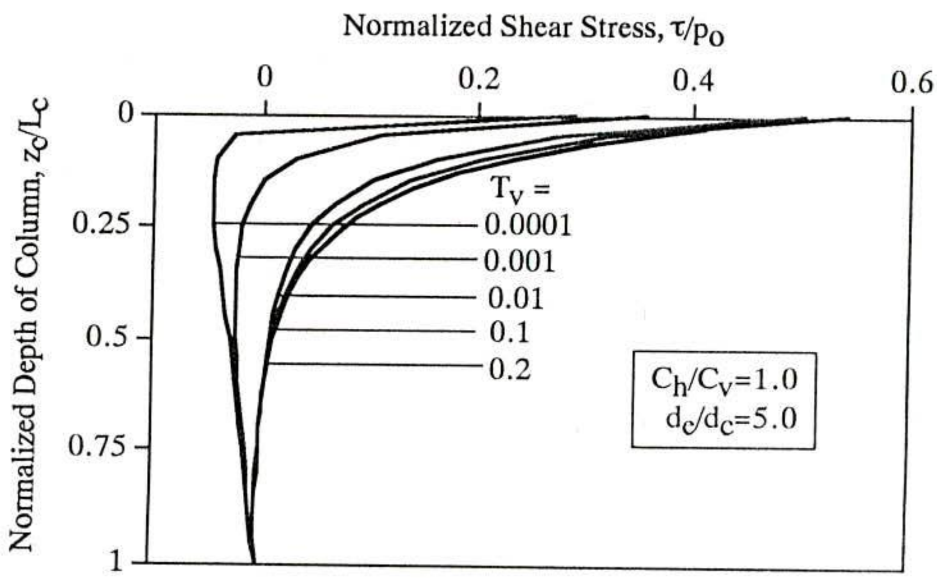


Figure 5.6 Mobilization of shear stress at column-soil interface along the depth of column with time.

of τ/p_o remains positive along the whole length of column. The value of τ/p_o at the top is found to be positive for any time. At a normalized depth z_c/L_c=0.10, the value of τ/p_o changes from -0.03 to 0.31 for T_v increasing from 0.0001 to 0.20. The variation of mobilized shear stress is more significant at smaller elapsed times i.e. T_v<0.01 than at later times i.e T_v>0.01.

5.4.2 Variation of stresses in column and soil

The varaition of normal stress in column, p_c, and surrounding soil, p_s, with time, is shown in Figs.5.7 to 5.9. The same example and the values of parameters as stated in the section 5.4.1 are considered here for the predictions. The variation of normalized vertical stress in the column, p_c/p_o, with normalized depth, z_c/L_c, is presented in Fig.5.7 for the various

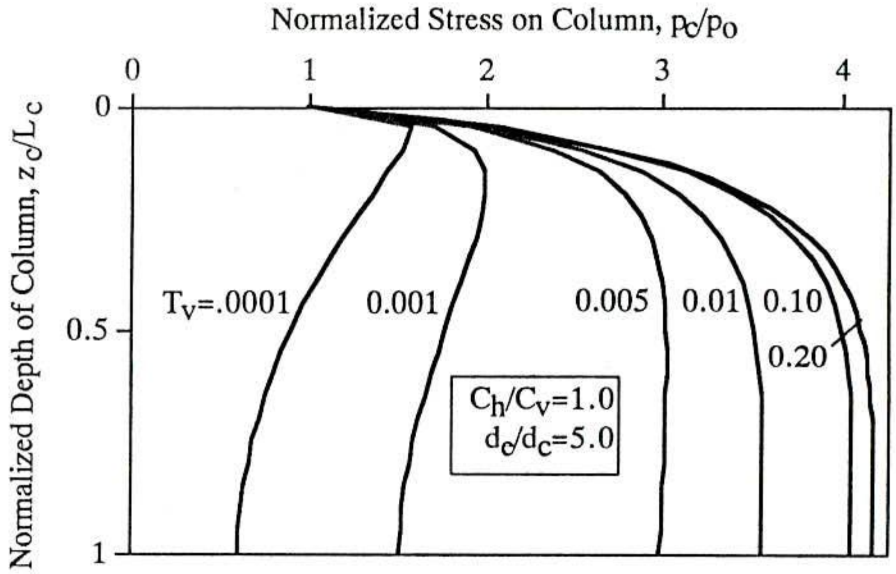


Figure 5.7 Variation of normal stress in column along the depth with time.

normalized elapsed times i.e. T_v=0.0001 to 0.20. At any depth of column, the value of p_c/p_o increases with time and reaches at the maximum value at the end of primary consolidation. Which is, of course, as expected. The value of p_c/p_o, predicted at a normalized depth, z_c/L_c=0.50, varies from 0.91 to 4.01 for T_v increasing from 0.0001 to 0.20. This figure also shows that at the initial stage of loading i.e. $T_v \leq 0.001$, the value of p_c/p_o increases from unity upto a certain depth beyond which it decreases gradually to the tip of column. At a time T_v=0.001, the value of p_c/p_o increases from 1.0 to 2.01 at a depth z_c/H_s=0.20 and after that it decreases to 1.53 at the bottom. After the certain elapsed time i.e. T_v>0.001, the value of p_c/p_o increases gradually from unity, with depth and becomes constant beyond a certain depth. The changes of stress in column with time is significant in the early stages of loading than at later times. This change of stress in column is expected as the interface shear stress changes its sign at the early stage of loading.

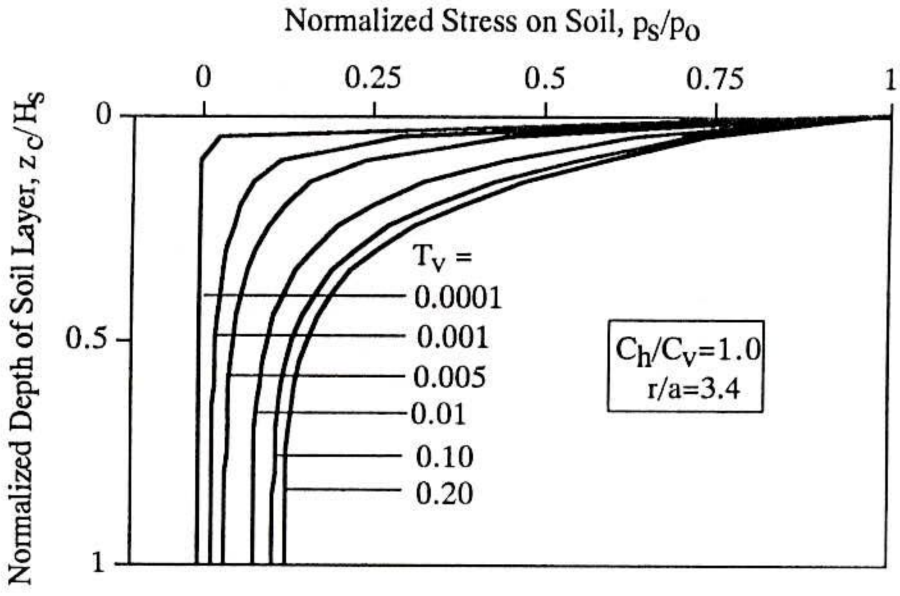


Figure 5.8 Variation of normal stress in soil along the depth with time.

The variation of normalized vertical stress in the surrounding soil, p_s/p_o, with the normalized depth of soil strata, z_c/H_s, is shown in Fig.5.8. The stress in soil is evaluated at a radial distance r/a=3.4. The value of p_s/p_o increases as the consolidation proceeds and is reached the maximum value at the end of primary consolidation. This is, of course, as expected. At any time, the value of p_s/p_o is unity at the surface and decreases gradually with depth. Beyond a certain depth p_s/p_o becomes constant. At a depth z_c/H_s=0.20, the value of p_s/p_o is found to increase from 0.0002 to 0.384 for the increasing value of time factor T_v, from 0.0001 to 0.20. The variation of normalized vertical stress in soil, p_s/p_o, with radial distance, r/a, at different times is shown in Fig.5.9. The value of p_s/p_o is predicted at a normalized depth of soil z_c/H_s=0.10. At lower times i.e. T_v<0.01, p_s/p_o increases gradually with r/a up to a certain radial distance beyond which it decreases and becomes almost constant at the outside

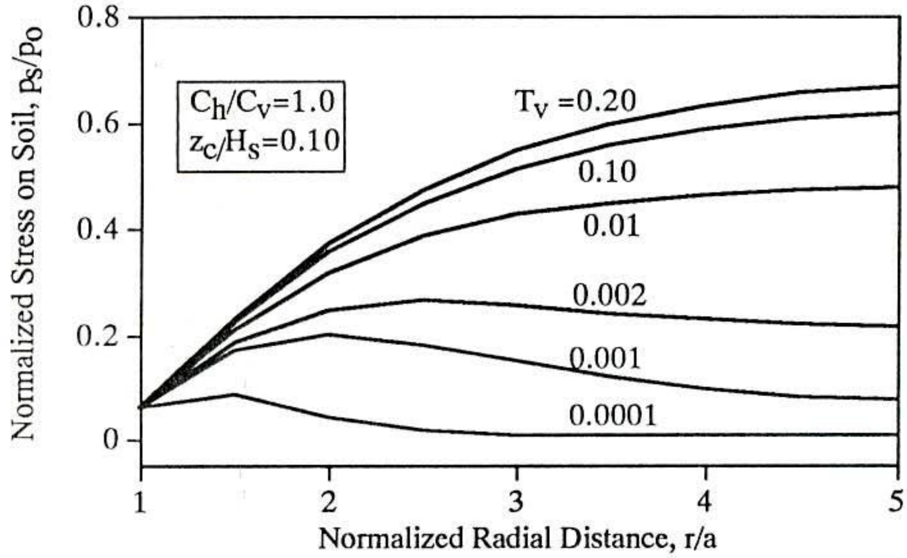


Figure 5.9 Variation of normal stress in soil along the radial distance with time.

boundary of the zone of influence. For T_v=0.001, the value of p_s/p_o increases from 0.057 to 0.196 at r/a=1.0 to 2.0 beyond which p_s/p_o decreases from 0.196 to 0.0724 at r/a=2.0 to 5.0. After an elapsed time i.e. $T_v \geq 0.001$, the value of p_s/p_o increases gradually with radial distance and becomes almost constant at the boundary of the zone of influence. At a time level T_v=0.1, the value of p_s/p_o increases from 0.057 to 0.61 with increasing radial distance, r/a, from 1.0 to 5.

5.4.3 Settlement profile with time

The settlement profile of soft ground reinforced by columnar inclusions at different times is shown in Fig.5.10 for the same example considered in sections 5.4.1 and 5.4.2. The settlement profile of the reinforced ground changes with elapsed time. At a certain radial distance, r/a, the value of S_t/H_s increases with the increase of time level, T_v. At an early stage

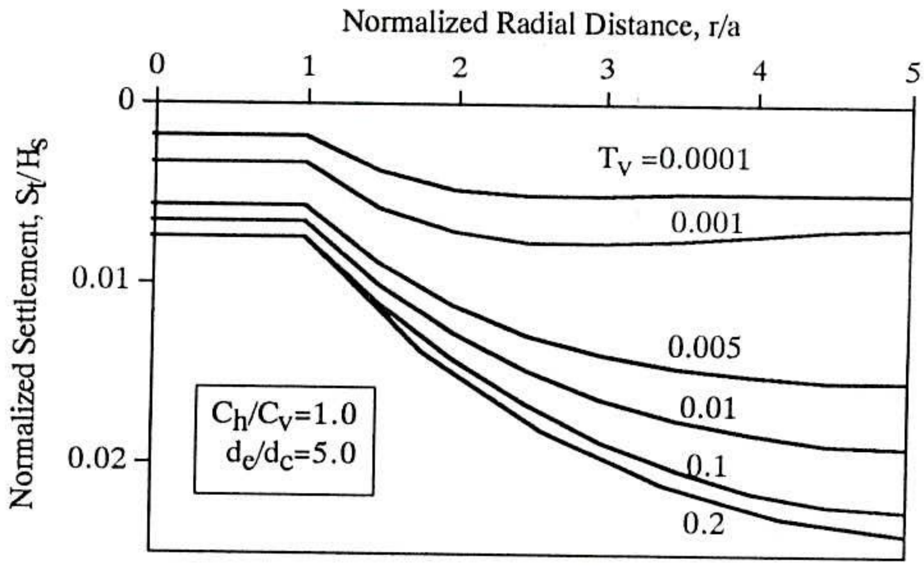


Figure 5.10 The course of settlement profile of treated ground with time.

of loading, the increase of settlement is more in the column region than in the soil region, which is, of course, to be expected because of the free draining characteristics of the column and the low permeability of the surrounding soil. The value of S_t/H_s increases from 0.0019 to 0.0035 at $0 \leq r/a \leq 1.0$ while it varies from 0.0051 to 0.007 at r/a=5.0 for the increase of T_v from 0.0001 to 0.001. But at later times, the increase of settlement is more in the soil region than in the column region. The value of S_t/H_s increases from 0.0058 to 0.0067 at $0 \leq r/a \leq 1.0$ and from 0.0155 to 0.0192 at r/a=5.0, for increasing T_v from 0.005 to 0.01. This figure also shows that the differential settlement of the column-reinforced ground increases with the increase of time. For T_v=0.0001, the maximum differential settlement is 0.0032 while it is 0.0163 at T_v=0.20.

The influence of horizontal to vertical permeability ratio of soil on the time rate of settlement of improved ground is shown in Fig.5.11. The results are predicted varying the

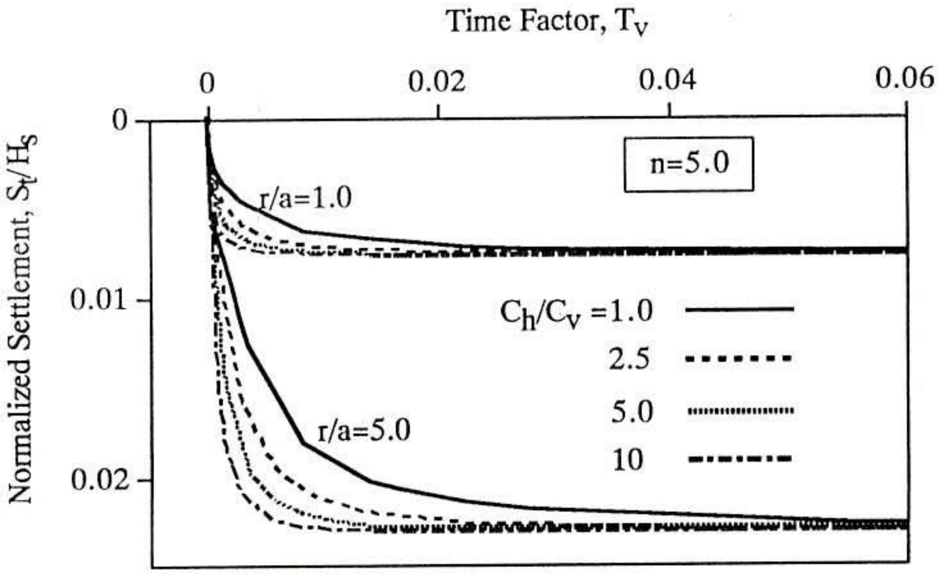


Figure 5.11 Influence of horizontal to vertical permeability ratio of soil on the course of settlement of treated ground with time.

value of C_h/C_v from 1.0 to 10. The normalized settlements of treated ground, S_t/H_s, at radial distances r/a=1.0 and 5.0, are plotted with time factor, T_v. This figure shows that the time rate of settlement increases significantly with the increase of C_h/C_v. For all the values of C_h/C_v, the values of S_t/H_s increase sharply with time and beyond a certain level of elapsed time they become almost constant. But this length of elapsed time at which S_t/H_s becomes constant, decreases with C_h/C_v. At an elapsed time T_v=0.01, the value of S_t/H_s varies from 0.0065 to 0.0076 at r/a=1.0 and 0.018 to 0.023 at r/a=5.0 for C_h/C_v increasing from 1.0 to 10. Figure 5.12 shows the effect of spacing of columns on the time rate of settlement of the soft ground reinforced by columnar inclusions. The column spacing i.e. $d_e/d_c(=n)$, varies from 2.5 to 10 and the value of C_h/C_v is 1.0. The normalized settlement, S_t/H_s, at a radial distance $r/a=(3n+2)/5$, is plotted with the time factor T_v. This figure shows that for all values of n, the value of S_t/H_s increases with the increase of time and becomes constant after a certain time which varies

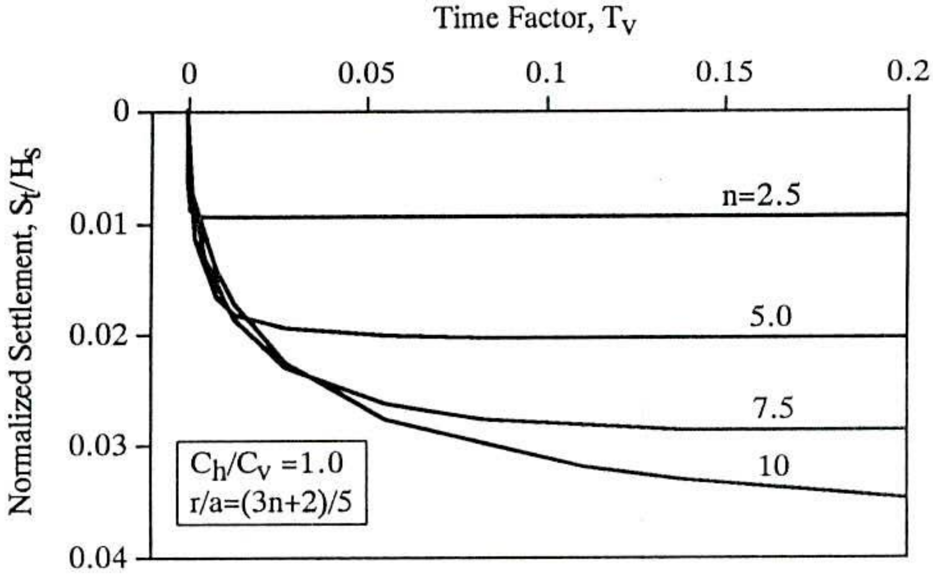


Figure 5.12 Influence of spacing of columns on the course of settlement of treated ground with time.

with the spacing of columns. The time rate of settelment is influenced significantly with the increase of column spacing. The value of S_t/H_s becomes constant at T_v=0.005 for n=2.5 while the value of S_t/H_s still increases at a time level T_v=0.20 for n=10. These findings reveal that a closer spacing of columns and the higher value of C_h/C_v increase the time rate of settlement of the soft ground reinforced by columnar inclusions.

5.5 Conclusions

A simple uncoupled consolidation model is proposed here to determine the time-dependent response of soft clay deposits reinforced by columnar inclusions. The stress concentration in the column with time is not considered. The consolidation of the surrounding soft soil due to radial and vertical expulsion of excess pore water pressure is evaluated based on the "Diffusion Theory" which is an extension of Terzaghi`s one dimensional consolidation theory. At any time, the value of degree of consolidation is determined at every nodal points and then the subsequent response of the reinforced ground is evaluated. This solution is not exact one but an approximate one which can be used as a first approximation. It is simple compared to Biot`s coupled consolidation theory. The radial inhomogeneity of the soil properties such as deformation modulus and shear modulus can be handled easily by the proposed model. It can also be used to handle the situation of stratified soil systems. Results are presented to illustrate the dissipation of excess pore water pressure and the variation of degree of consolidation at every nodal point. The influence of horizontal to vertical permeability ratio of soil and spacing of columns are also presented. Results are presented to show the dissipation of pore water pressure, load sharing between the column and the surrounding soil and time rate of settlement of improved ground.

CHAPTER SIX

VALIDATION OF THE PROPOSED MODEL
WITH
THEORETICAL AND EXPERIMENTAL RESULTS

6.1 General

Any newly developed model is subjected to validation by a standard procedure. The validation of the results predicted by the proposed foundation model with those of theoretical and experimental results, is presented in this chapter. The theoretical verification of the proposed model is performed by comparing the results with the existing approaches developed in the past for the analysis of the behaviour of soft ground reinforced by columnar inclusions. The existing methods include analytical and numerical solutions. In the numerical solutions, both finite difference and finite element methods are used. The results obtained from a standard finite element programme, CRISP, are also compared here with those obtained using the proposed model. The results obtained from laboratory tests on lime columns subjected to flexible loading are compared with the predictions obtained from the proposed model. The laboratory test results on sand columns subjected to flexible loading performed elsewhere and some field test results reported in the literature, are also compared with the proposed model.

This investigation shows that the proposed model can be treated as a better one than the existing methods since it can be used for any loading conditions ranging from flexible to rigid loading and can handle rather complicated situations. The result obtained from finite element analysis shows that there exists good agreement between the results obtained from the proposed model and those of by the finite element analysis. From the comparison of experimental results, it is also revealed that the proposed model can be used with a reasonable degree of accuracy in predicting the settlement behaviour of soft ground reinforced by columnar inclusions in the laboratory and as well as in the field.

6.2 Comparison with Existing Approaches

The settlement responses of the soft ground reinforced by columnar inclusions are compared with those evaluated using the existing methods of analysis. The results from the approaches proposed by Madhav and Van Impe (1994) and Alamgir et al. (1995) based on the "free strain" theory for the case of uniform flexible loading acting over the entire area of reinforced ground, are compared with those predicted by the proposed model. The predictions using the design approaches suggested by Priebe (1976), Aboshi et al. (1979) and Balaam & Booker (1981) based on the "equal strain" theory are also compared here with those of evaluated by the proposed foundation model for the case of rigid loading.

6.2.1 Approaches based on "free strain" theory

Madhav and Van Impe (1994) proposed an approach to predict the influence of granular bed placed over the entire area of soft ground reinforced by stone columns. Depending on the stiffness of overlaying granular bed, the settlement response of the reinforced ground ranges from flexible to rigid. For a particular example, the predicted settlement of the treated ground is compared with that obtained by the proposed model. For comparison two extreme cases, namely, flexible and rigid loading conditions, are considered. The columns are extended to bedrock. The values of the parameters are taken as d_e/d_c=2.5, E_r (modular ratio)=5.0 and

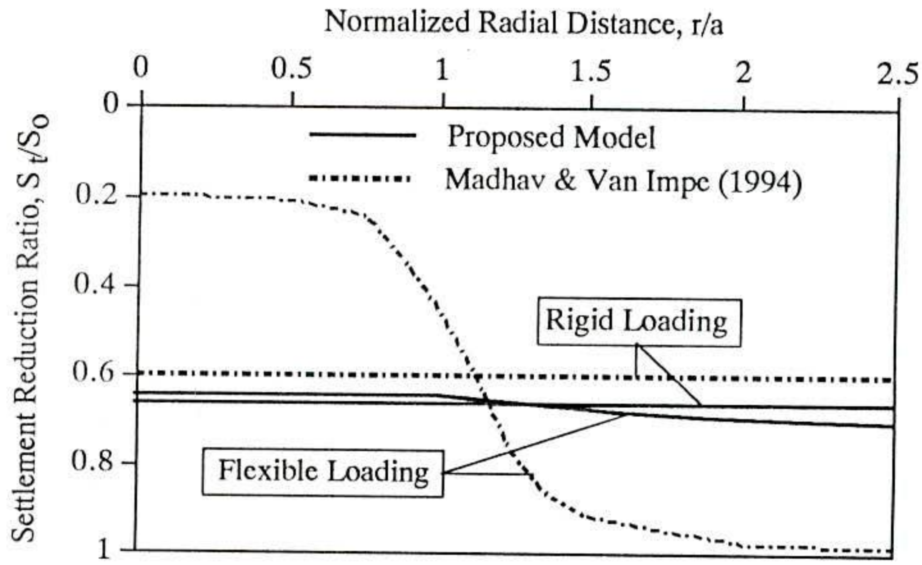


Figure 6.1 Comparison of the results predicted by the proposed model with those of by Madhav and Van Impe (1994).

ν_s=0.35. Figure 6.1 shows the comparison of settlement in the S_t/S_o versus *r/a* diagram, where, S_o and S_t are the settlements of the untreated and the treated ground, respectively. The predicted values differ significantly from each other specially for the case of flexible loading. For rigid loading they are very close to each other, however, the proposed model overpredicts by 10.12%. For flexible loading the proposed model underpredicts in the column region by 68.25% at *r/a*=0.0. But it overpredicts in the soil region by 39.55% at *r/a*=2.5. The proposed model considers the compatibility of displacements at every nodal point and the stress transfer between the column and the surrounding soil along the depth, while in Madhav and Van Impe (1994), the compatibility of displacement is satisfied only at the top and the stress transfer along the column-soil interface is not considered. These considerations lead to prevent the stress transfer to the column from the surrounding soft soil along the depth. Madhav and Van Impe (1994) also considered that the granular fill placed over the reinforced

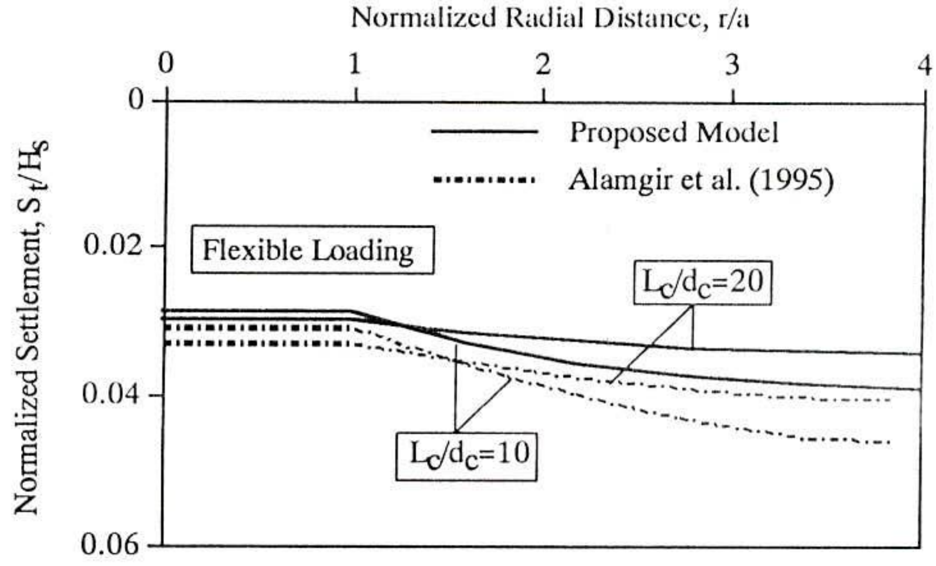


Figure 6.2 Settlement profiles of column-reinforced soft ground predicted by the proposed model and Alamgir et al. (1995).

ground is acted as an incompressible shear layer while in the proposed approach the compressibility of the overlaying granular fill is considered.

Figure 6.2 shows the comparison of settlement profiles of the column-reinforced soft ground as predicted by Alamgir et al. (1995) and by the proposed model. A typical soft ground reinforced by end bearing columnar inclusions subjected to surface loading over the entire area, is considered for prediction. The predictions are performed for two cases varying the length to diameter ratio of column, L_c/d_c. The values of parameters are taken as p_o/E_s=0.10, d_e/d_c=4.0, L_c/d_c=10 and 20, E_c/E_s=10 and ν_s=0.4. The predicted settlement profiles are very similar and their magnitudes are also very close to each other. Alamgir et al. (1995) overpredict the settlement over the entire area of reinforced ground for both cases i.e. L_c/d_c=10 and 20. The differences range from 6.64% at the column region i.e. $0 \leq r/a \leq 1.0$ to

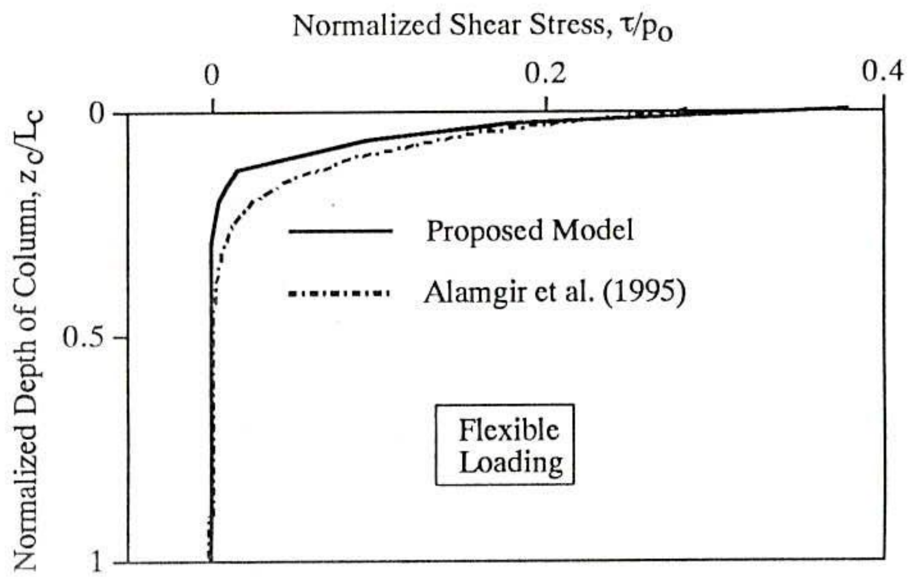


Figure 6.3 Distribution of shear stress at column-soil interface along the depth as predicted by the proposed model and Alamgir et al. (1995).

15.21% at the outside boundary of the zone of influence i.e. r/a=4.0. In Alamgir et al. (1995), the compatibility is satisfied between the displacement of column and the soil element by imposing a postulated displacement function. In this approach, the soil element near the outside boundary of the zone of influence i.e. at $r/a=n$, is considered only for displacement compatibility. While, in the proposed model compatibility of displacement is satisfied at every nodal point by using the fundamental equation of equilibrium that exists in any infinitesimal soil element. However, the stress transfer along the column-soil interface is considered in both the approaches. Fig.6.3 shows the distribution of shear stresses at column-soil interface along the depth. At the top of column i.e. at the normalized depth, z_c/H_s=0.0, the value of normalized shear stress, τ/p_o, predicted by Alamgir et al. (1995) is less than that of predicted by the proposed model. But, after that upto a certain depth, the proposed model underpredicts.

However, in the lower half of the column, they become almost equal and tend to zero at the base. Two-thirds of column remains almost shear stress free, a result predicted by both the approaches.

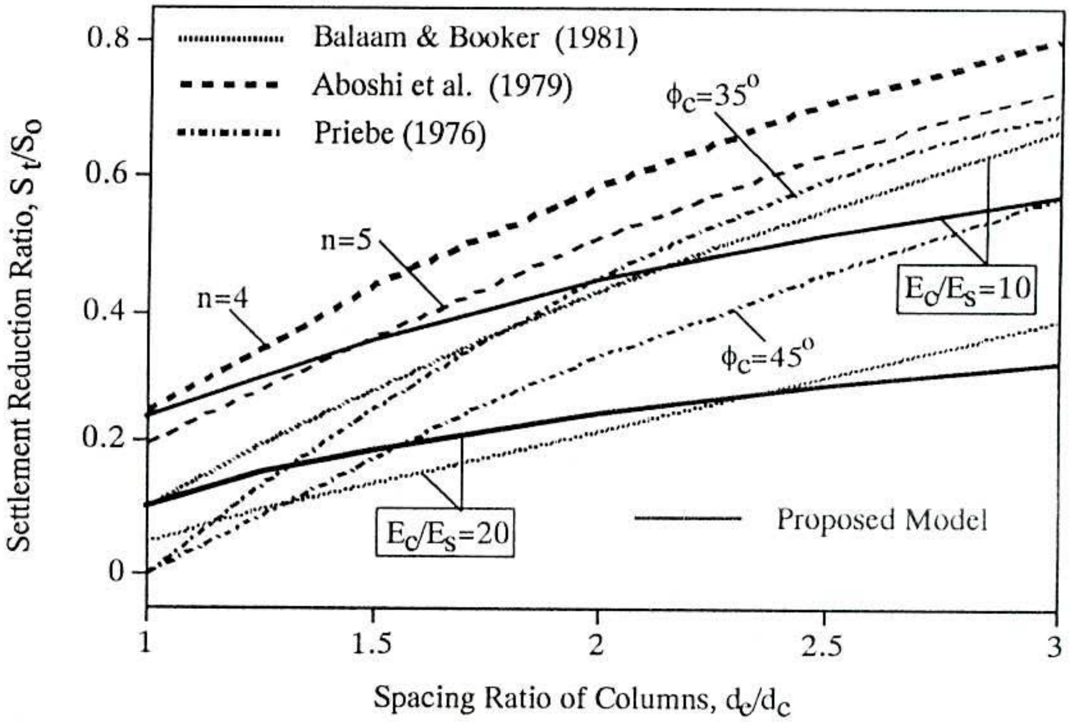


Figure 6.4 Comparison of the results predicted by the proposed model and the existing methods of analysis based on "equal strain" theory.

6.2.2 Approaches based on "equal strain" theory

The practicing engineers still use the approaches based on "equal strain" theory to predict the behaviour of soft ground reinforced by columnar inclusions. In the "equal strain" theory, it is assumed that the horizontal section of the ground remains horizontal even after the application of load. The results obtained by such existing approaches are compared here with those predicted by the proposed model for the case of rigid loading. The approaches suggested by Priebe (1976), Aboshi et al. (1979) and Balaam and Booker (1981) have been and are being used widely and hence they are taken into account for comparison. In the proposed model, to account for rigid loading condition, a granular fill having considerably high stiffness, E_f/E_s=100, H_f/H_s=0.333 and ν_f=0.30, is considered to exist over the entire reinforced ground. Figure 6.4 shows that the predicted settlements of the reinforced ground as presented in the S_t/S_o versus d_e/d_c plotting. The approaches use different parameters to characterize the stiffness of column. Priebe (1976) uses angle of internal friction of granular materials, ϕ_c, Aboshi et al. (1979) uses stress concentration ratio n_c, while Balaam et al. (1981) and the proposed model use the relative stiffness of column and soil i.e. modular ratio, E_c/E_s. For comparison, the average values of these parameters are considered. Figure 6.4 shows that he proposed model underpredicts the settlement reduction ratio, S_t/S_o, compared to that predicted by Aboshi et al. (1979) for any spacing ratio. It overpredicts settlement reduction ratio compared to that of Priebe (1976) for very closer spacing but underpredicts for higher spacing. It can be seen from this figure that the relationship between the predictions of Balaam and Booker (1981) and the proposed method are closer. Although the proposed model overpredicts somewhat for closer spacing i.e. $1 \le d_e/d_c \le 2.25$, while underpredicting for higher spacing i.e. $2.25 \le d_e/d_c \le 3.0$.

6.3 Verification by Finite Element Analysis

The settlement responses of some representative cases of soft ground reinforced by columnar inclusions obtained by the proposed method are compared with those predicted by the finite element analysis. It is considered that the column and the surrounding soil are bonded together i.e. there is no slip at the column-soil interface. The load is applied through flexible and as well as rigid loading conditions. End bearing and floating column situations are also considered for the verification. For the finite element analysis, the program CRISP (Britto & Gunn 1987) is used. A typical finite element mesh covering the solution region is shown in

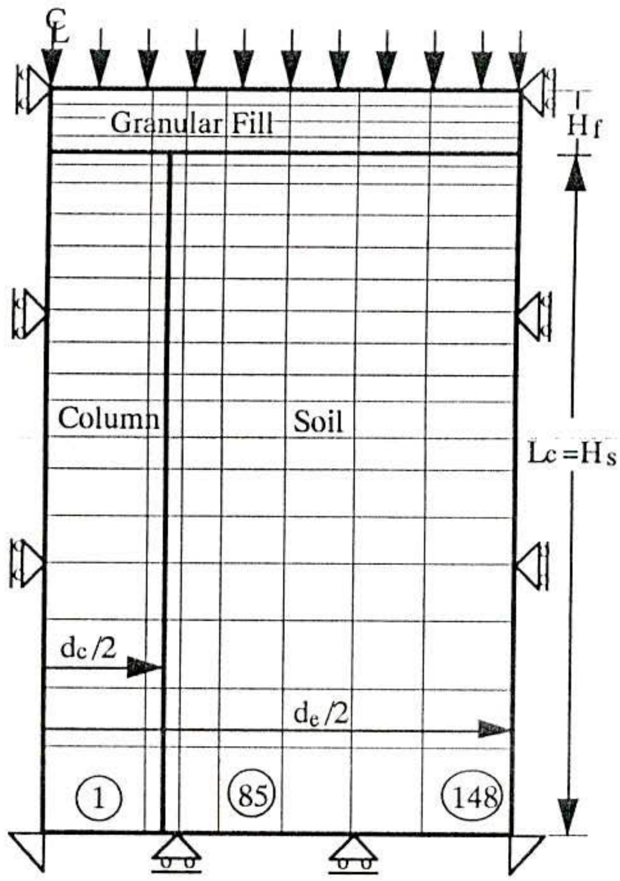


Figure 6.5 Typical finite element mesh used for the analysis.

Fig.6.5. Because of symmetry only one half of the problem requires modelling and this is done using 168 eight-nodded linear-strain quadrilateral elements. Each element has nine integration points at which the stresses and the strains are calculated. By putting small elements near the column-soil interface and the loading boundaries, the sharp changes in stresses and strains are accurately modelled. The load is applied as uniform pressure at the top. The boundary conditions for the mesh are the same as considered in the proposed method, that is, the outer boundary is restrained in horizontal direction and is assumed to be smooth i.e. free to move in the vertical direction. The base of the layer assumed to be smooth but rigid and hence is restrained in the vertical direction but free to move in a horizontal direction. To model for

flexible loading condition, no granular fill over the reinforced ground is considered while for rigid loading condition a granular fill having considerably higher stiffness i.e. E_f/E_s=100, H_f/H_s=0.333 and ν_f=0.30, is considered to exist over the entire reinforced ground. The predictions for the case of time-independent and time-dependent problems compared and presented in the following sections.

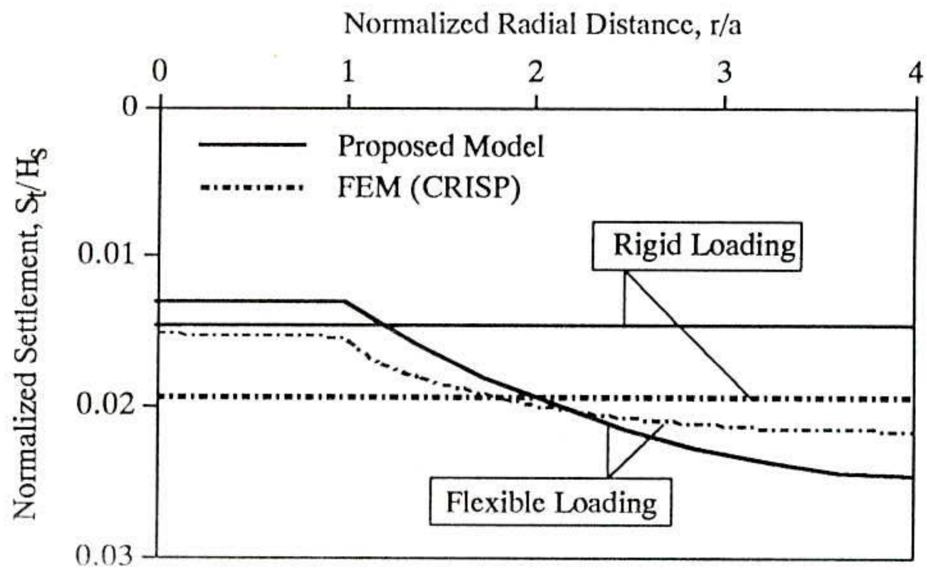


Figure 6.6 Comparison of settlement profiles predicted by the proposed method and the finite element analysis for end bearing column-reinforced soft ground.

6.3.1 Time-independent response of reinforced ground

The settlement profile of the composite ground, the distribution of shear stresses along the column-soil interface and the variation of stress concentration in column with depth are shown in Figs.6.6, 6.7(a) and 6.7(b), respectively. The problem selected is that of a uniformly loaded soft ground reinforced by columnar inclusions in group having the values of p_o/E_s=0.10, d_e/d_c=4.0, L_c/d_c=10, L_c/H_s=1.0, E_c/E_s=25, ν_c=0.20, ν_s=0.40, ν_f=0.30, H_f/H_s=0.333 and E_f/E_s=100. The settlement of the composite ground along the radial distance is shown in Fig.6.6. For flexible loading, the proposed model underpredicts the settlement compared to the finite element method from the center of column up to the radial distance r/a=2.25, while beyond that it overpredicts up to the outside boundary of the zone of influence i.e. at r/a=4. The differences in the magnitudes of the settlements are 16.31% over the column and 11.85% at the boundary of the zone of influence. This figure reveals that the predicted deformation patterns of the reinforced ground obtained by the proposed model and the finite element analysis are quite similar. From this figure, it can also be seen that the proposed model underpredicts the settlement of the reinforced ground by about 24.50% in case of rigid loading.

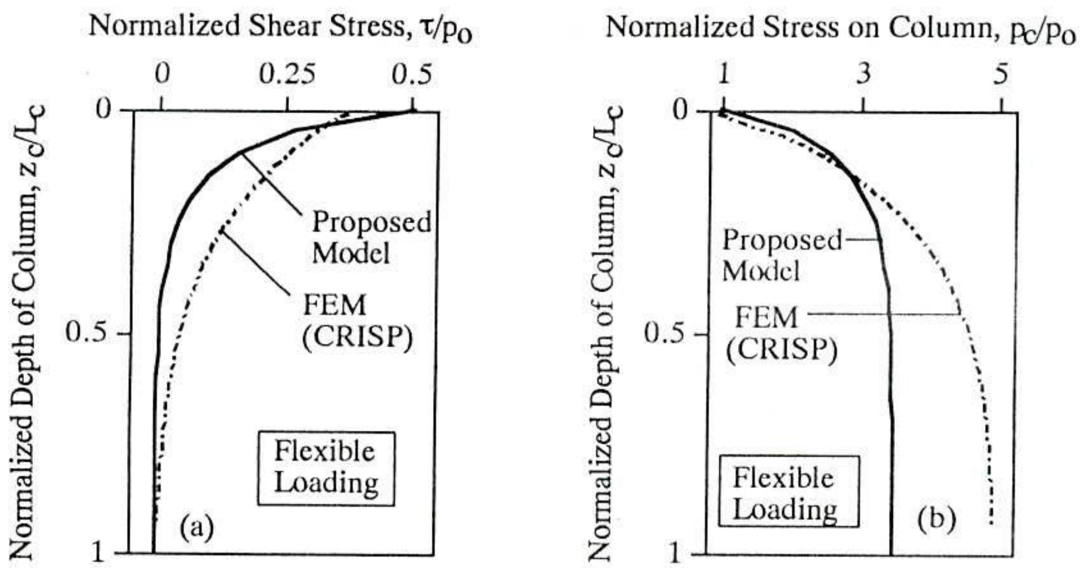


Figure 6.7 Comparison of results predicted by the proposed model and finite element analysis for end bearing column-reinforced soft ground: (a) Distribution of shear stress at column-soil interface; and (b) Variation of normal stress in column along the depth.

The distribution of shear stress along the depth of column is shown in Fig.6.7(a). The proposed model overpredicts at the top but beyond a certain depth it underpredicts up to the bottom of column. At the normalized depths of column, z_c/L_c=0.0 and 0.20, the predicted values of normalized shear stress, τ/p_o, are 0.51 and 0.07, respectively, by the proposed method and 0.38 and 0.15, respectively by the finite element analysis. The variation of stress concentration in the column along the depth is presented in Fig.6.7(b). In both predictions, the stress in column increases rapidly with depth and beyond a certain depth it becomes almost constant. Near the top region of column i.e. $0 \le z_c/L_c \le 0.20$, the predictions obtained by both the approaches are agreed well with each other. But at the lower region of column, the proposed model underpredicts when compared to the results of the finite element analysis.

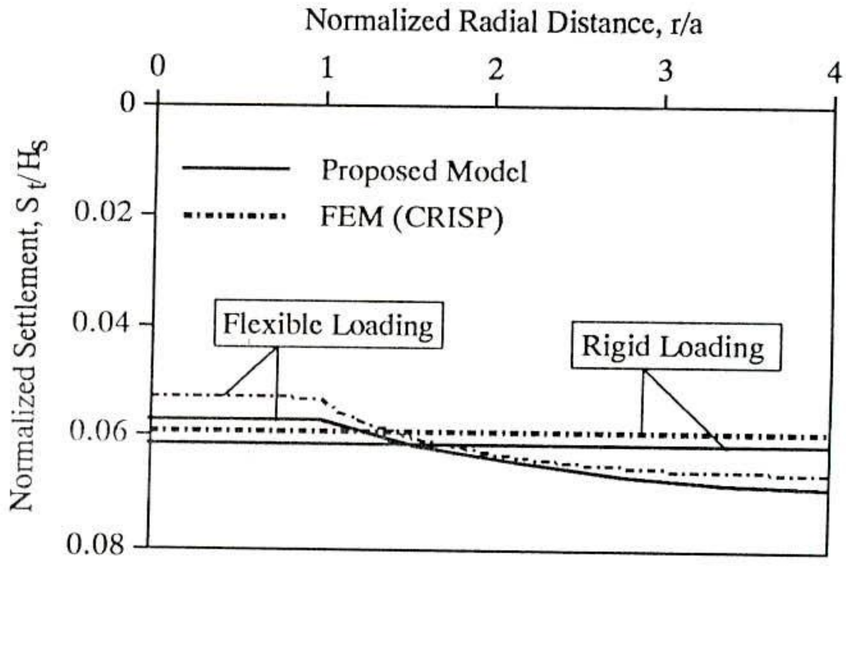


Figure 6.8 Comparison of settlement profiles predicted by the proposed model and the finite element analysis for floating column-reinforced soft ground.

Similar comparison as stated above, is made in Figs.6.8 and 6.9 for the case of soft ground reinforced by floating columnar inclusions. The columns penetrated up to the mid depth of soil strata. The values of the parameters are taken as p_o/E_s=0.10, d_e/d_c=4.0, L_c/d_c=5.0, L_c/H_s=0.50, E_c/E_s=25, ν_c=0.20, ν_s=0.30, ν_f=0.30, H_f/H_s=0.333 and E_f/E_s =100. Figure 6.8 shows that there exists good agreement between the predictions obtained by the proposed approach and the finite element method. However, the proposed model overpredicts by the amounts of 8.62% at column region i.e. $0 \leq r/a \leq 1$ and 3.61% at the outside boundary of the zone of influence i.e. r/a=4.0, for flexible loading compared to the results from the finite element method. In case of rigid loading, the difference of S_t/H_s is obtained as 3.88% over the entire area by both the approaches. The distribution of normalized shear stress, τ/p_o, along the depth of column is shown in Fig.6.9(a). The proposed model underpredicts both the positive

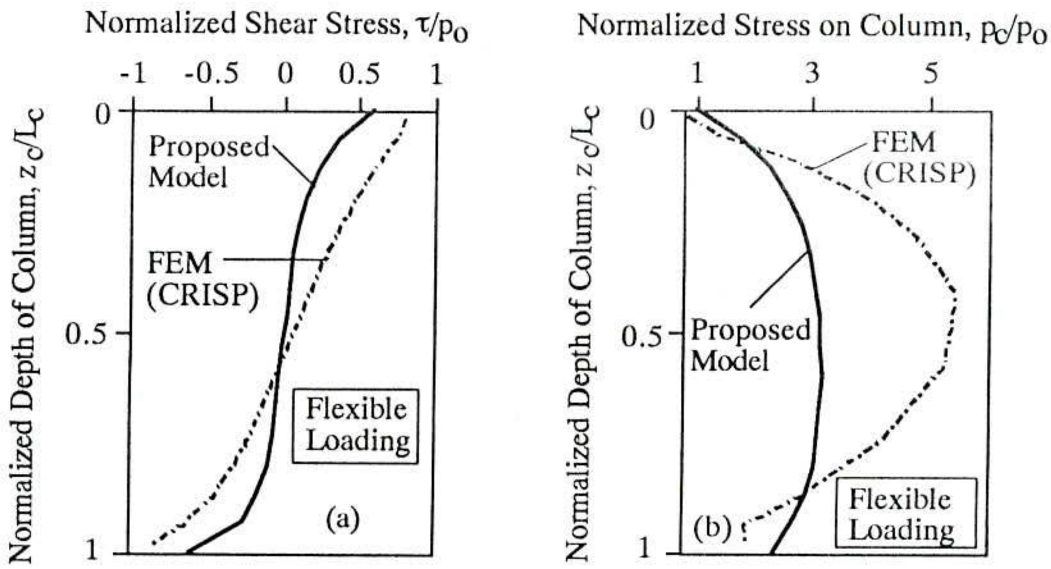


Figure 6.9 Comparison of results predicted by the proposed model and finite element analysis for floating column reinforced soft ground: (a) Distribution of shear stress at column-soil interface; and (b) Variation of normal stress in column along the depth.

(i.e.acting downward) and negative (i.e. acting upward) shear stresses moblized along the column-soil interface. But, it is interesting to note that the neutral plane is found almost at the same depth, which is around $0.6L_c$, for both the predictions. Fig.6.9(b) represents the variation of normal stress in column along the depth. The trends of the changes of stress in column are similar as predicted by the proposed model and the finite element analysis. But the proposed model underpredicts the normalized stress in column significantly in the region of $0.10 \le z_c/L_c \le 0.85$.

6.3.2 Time-dependent response of reinforced ground

The time-dependent response of column-improved ground obtained from the proposed method and the finite element analysis are compared here. The problem selected is that of a uniformly loaded soft ground reinforced by end bearing columnar inclusions having the value p_o/E_s=0.10, d_e/d_c=4.0, L_c/d_c=10, L_c/H_s=1.0, E_c/E_s=25, ν_c=0.20 and ν_s=0.45. The consolidation of soft ground is evaluated considering the expulsion of excess pore water pressure due to vertical and radial flow. The anisotropic flow parameter C_h/C_v is 5.0. The boundary conditions for consolidation analysis is that the top surface and the interface of column-soil are drainage free i.e. at these locations, the excess pore water pressures are zero at any time. The results are presented in Figs.6.10 and 6.11, respectively to show the comparisons on the distribution of excess pore water pressure and the course of settlement with time. Figure 6.10 shows the distribution of normalized excess pore water pressure, u/p_o, with normalized radial distance, r/a, for the time factor T_v=0.00045, 0.0013 and 0.018. The pattern of distribution of excess pore water pressure is similar for both cases as predicted by the proposed model and the finite element analysis. This figure also shows that there exists good agreement between the magnitude of the results. However, the proposed model underpredicts comparison with the finite element analysis. The course of settlement of reinforced ground with time is shown in Fig.6.11 for radial distances $0 \le r/a \le 1.0$ i.e. at column region and at 4.0 i.e.

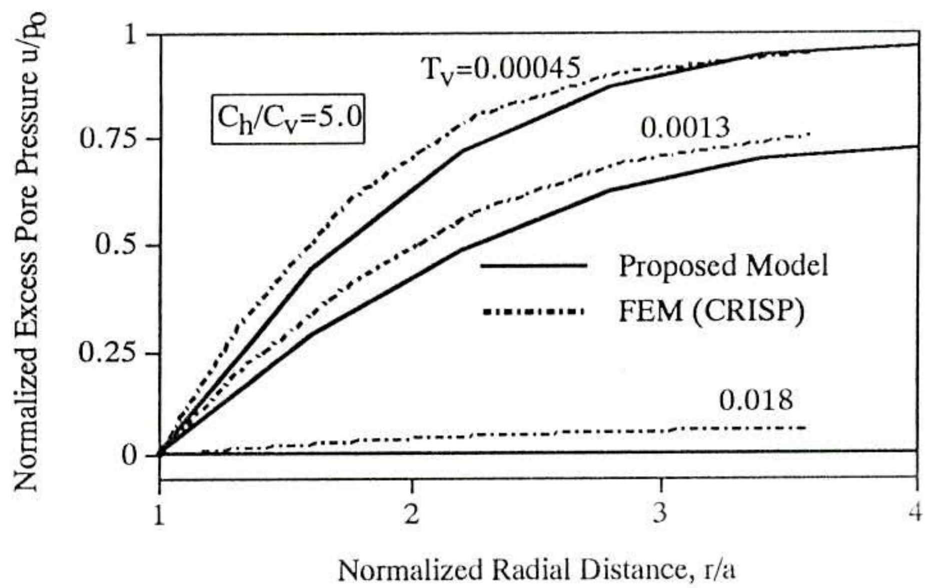


Figure 6.10 Comparison of distribution of excess pore water pressure along the radial distance with time predicted by the proposed model and the finite element analysis.

somewhat in outside boundary of the zone of influence. The pattern of changes of settlement with time is quite similar as predicted by the proposed model and the finite element analysis. The time rate of settlement as predicted by the proposed model is more than that of finite element method. In the proposed model, the excess pore water pressure is evaluated by the "Diffusion Theory" which is an uncoupled formulation. Moreover, the stress concentration in the column is not considered while evaluating the excess pore water pressure. But the finite element method, CRISP, uses Biot's three dimensional consolidation theory which is a coupled consolidation solution. As the difference of the time rate of settlement obtained from the two predictions is not large specially for larger elapsed time, theses findings reveal that the proposed model can be used with a reasonable degree of accuracy to determine the time-dependent response of column-reinforced ground.

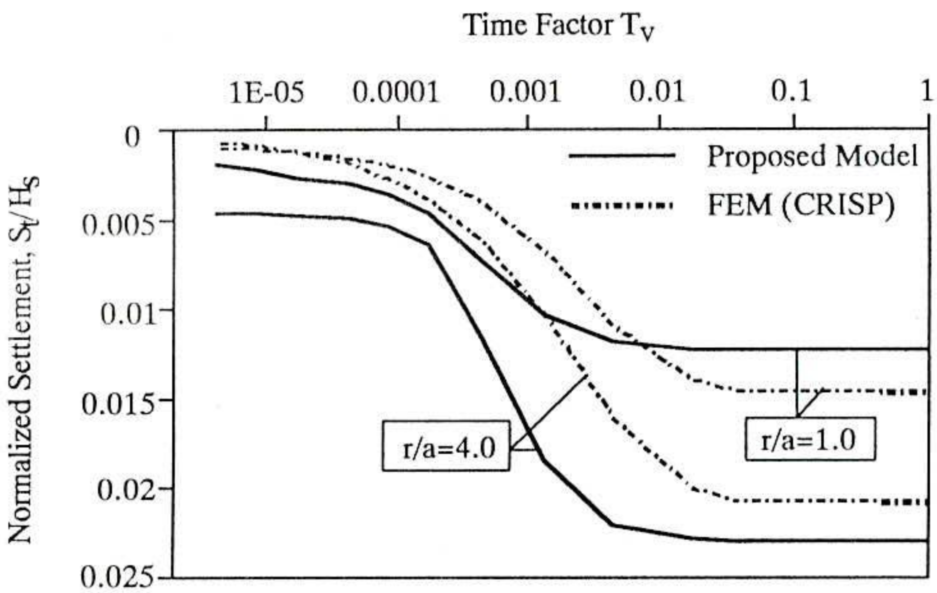


Figure 6.11 The course of settlement of column-reinforced soft ground with time predicted by the proposed model and the finite element analysis.

6.4 Comparisons with Experimental Results

The applicability of the proposed model to predict the laboratory and field test results of soft ground reinforced by columnar inclusions is verified here. The performance of the proposed model is examined for both rigid and flexible loading conditions. The results obtained from a small scale laboratory test scheme conducted by Nishida (1994) for flexible loading condition is compared with those of the predicted by the proposed model. In this test, lime columns were installed in group to reinforce soft ground having various degrees of penetration of columns. The test results on sand columns subjected to uniform surcharge presented by Leung and Tan (1993) and some field test results reported in the literature are also compared.

6.4.1 Laboratory test results

A small scale laboratory test was conducted by Nishida (1994) to examine the settlement response of soft ground reinforced by a group of lime columns. Both the end bearing and floating columns, varying the degree of penetrations, are considered. The base clay was obtained from 3.0m to 4.0m depth from a site in Kawasoe town of Saga City, Saga, Japan. The physical properties of the clay are shown in Table 6.1.

Table 6.1 Physical Properties of Used Ariake Clay.

Natural water content , w_n (%)	115~125
Specific gravity, G_s	2.623
Liquid limit, w_L (%)	99.20
Plastic limit, w_P (%)	39.60
Plasticity index, I_P	59.60

The experiments were conducted in a circular plastic mold of 0.30m diameter and 0.30m high. The Ariake clay (Table 6.1) was thoroughly remoulded at a water content of about 120%. The clay slurry was poured taking care not to trap air bubbles in slurry. Pouring of slurry was carried out in three layers and the preliminary consolidation was conducted for each layer under the vertical pressure of about 2.5kPa for a certain time. After the completion of pouring slurry and preliminary consolidation, the final consolidation pressure of 10kPa was applied for 3 to 4 days, untill the end of primary consolidation. After that pressure was removed and the columns were installed. A casing of diameter 0.05m, was driven in clay media till the desired depth and the equivalent clay cylinder as well as the casing were removed gradually. The lime columns were then driven. For a particular test, seven columns having the

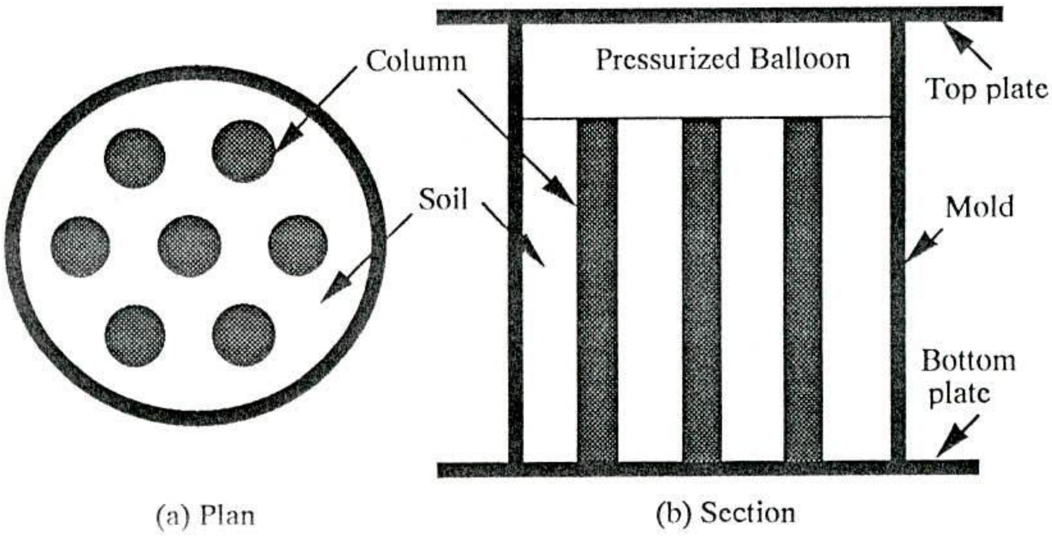


Figure 6.12 Typical arrangement of column used for laboratory test.

same diameter and length, were installed giving equal spacing. A typical arrangement of columns during test is shown in Fig.6.12. The cylindrical lime columns were made by using 10% of lime by the weight of clay. The columns were kept around one month under water for curing. Some specimens were made also for testing the compressive strength of column. The columns have the dimater 0.05m and lengths 0.25, 0.20, 0.15 and 0.10m. After the installation of columns, the same pressure as used for the consolidation of clay slurry i.e. 10kPa, was applied again and kept it for 2~3 days. The incremental uniform pressure was then applied and each load increment was kept untill the end of primary consolidation. The applied pressure was increased from 10kPa to 120kPa at an increment of 10kPa. The uniform pressure was applied through the presurized rubber balloons applied from a compressure. For each increment of pressure, the settlements were measured on the top of column and on soil, in between two columns. The stresses were also measured on the top and bottom of column and soil. From this test, the stresses in column and soil are presented in the Tables 6.2 and 6.3, measured at the bottom and top, respectively. And the settlement of the improved ground are given in Tables 6.4 and 6.5, measured on the top of column and soil, respectively.

Table 6.2 Normal Stresses Measured at the Bottom of Column and Soil.

Pressure (kPa) / L_c/H_s	Ratio of normal stresses at the bottom of column and soil											
	10	20	30	40	50	60	70	80	90	100	110	120
0.40	1.57	1.83	2.15	2.56	2.59	3.02	3.83	4.02	4.49	4.61	4.93	5.28
0.60	1.85	2.12	2.62	4.57	5.53	5.88	6.32	7.10	8.04	8.91	9.17	9.13
0.80	2.62	4.21	6.81	10.16	18.42	33.04	55.9	63.34	96.52	-	-	-
1.00	20.3	42.7	136.3	154.1	167.5	177.4	196.1	289.8	326.7	363.3	400.3	-

Table 6.3 Normal Stresses Measured at the Top of Column and Soil.

Pressure (kPa) / L_c/H_s	Ratio of normal stress at the top of column and soil											
	10	20	30	40	50	60	70	80	90	100	110	120
0.40	-	1.63	1.71	1.56	1.53	-	-	1.89	1.78	1.79	1.81	1.83
1.00	-	1.24	1.43	1.42	1.34	1.34	1.35	1.36	1.37	1.37	1.38	1.36

Table 6.4 Settlement Measured at the Top of Column.

Pressure (kPa) / L_c/H_s	Measured Normalized Settlement (Settlement / Depth of soil layer)											
	10	20	30	40	50	60	70	80	90	100	110	120
0.40	0.017	0.028	0.045	0.065	0.072	0.091	0.10	0.105	0.114	0.119	0.124	0.129
0.60	0.005	0.011	0.021	0.038	0.049	0.057	0.065	0.072	0.083	0.088	0.092	0.096
0.80	0.006	0.013	0.017	0.022	0.028	0.030	0.033	0.036	0.038	0.040	-	-
1.00	-	0.002	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	0.009	-

Table 6.5 Settlement Measured on Soil in Between Two Columns.

L_c/H_s \ Pressure (kPa)	Measured Normalized Settlement (Settlement / Depth of soil layer)											
	10	20	30	40	50	60	70	80	90	100	110	120
0.40	0.014	0.027	0.046	0.069	0.077	0.10	0.11	0.116	0.126	0.132	0.139	0.145
0.60	0.005	0.015	0.030	0.049	0.061	0.070	0.080	0.088	0.102	0.102	0.113	0.118
0.80	0.006	0.016	0.023	0.031	0.040	0.045	0.049	0.054	0.059	0.064	-	-
1.00	-	0.005	0.009	0.014	0.016	0.018	0.021	0.024	0.027	0.028	0.029	-

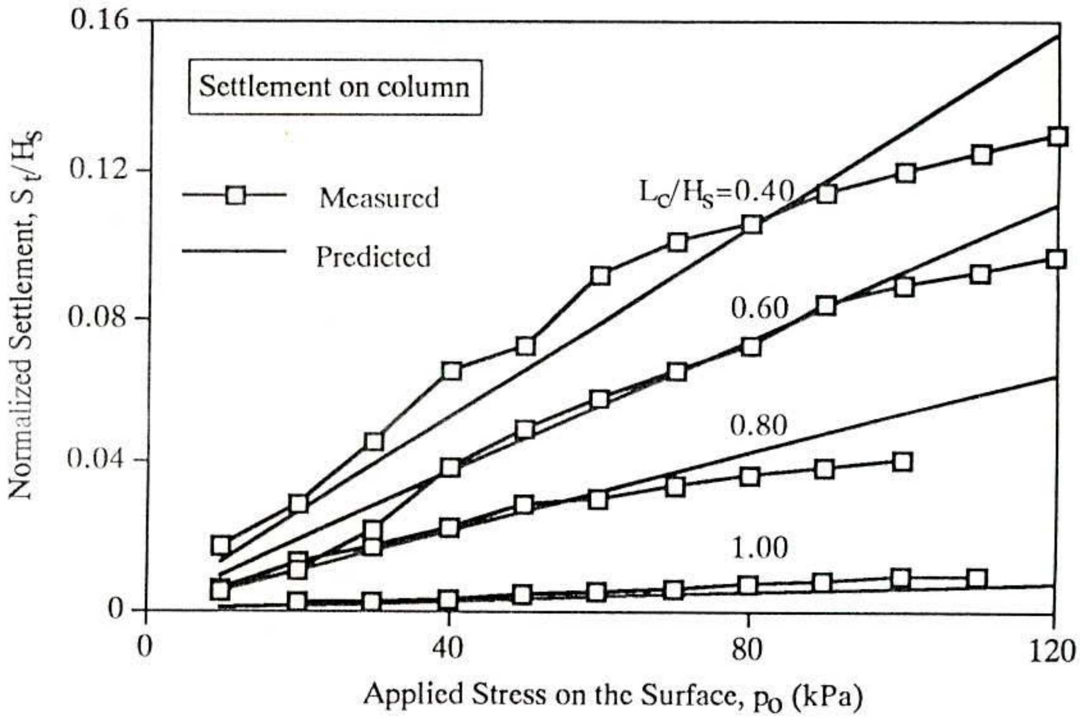


Figure 6.13 Measured and predicted values of settlements of soft ground reinforced by lime columns in group in the laboratory.

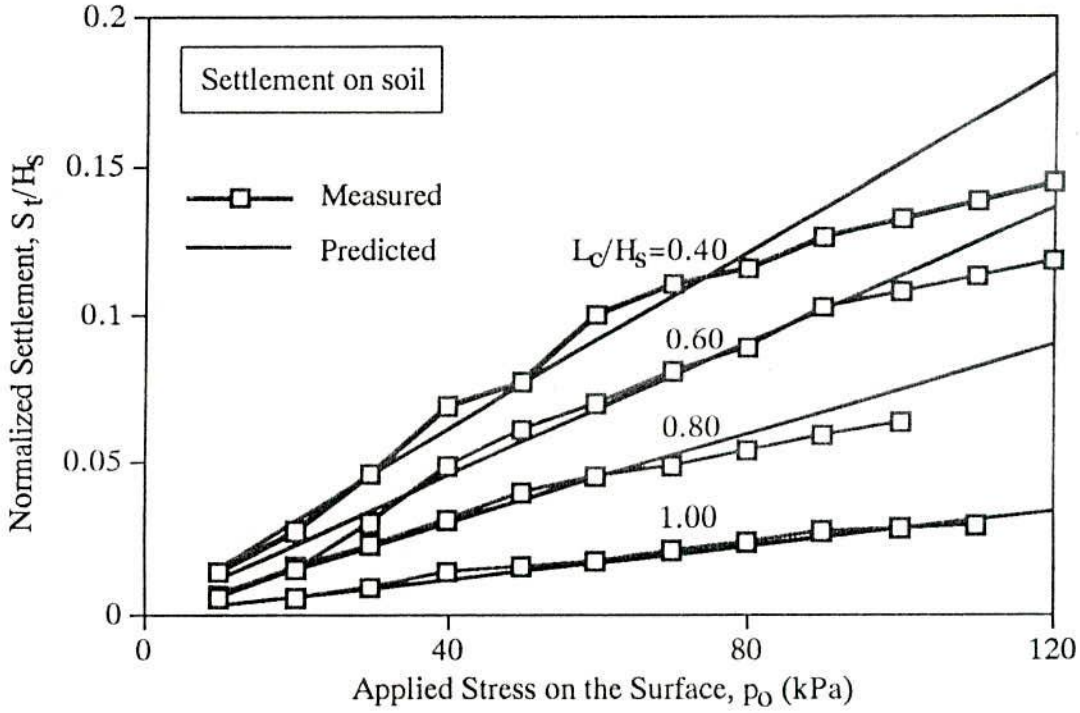


Figure 6.14 Measured and predicted values of settlements of soft ground reinforced by lime columns in group in the laboratory.

For comparison, the measured settlement response of this reinforced ground are predicted by the proposed model. The mechanical properties of the soil and the columns are estimated as E_s=500, E_c/E_s=80 and ν_s=0.40 based on routine laboratory test results. The diameter of the zone of influence for each column is estimated as 0.113m. The measured and the predicted values are presented in Figs.6.13 and 6.14 for the settlements on the top of column and on the soil respectively. The predictions are good in determining the settlement at the top of column and at the top of soil upto the stress level of 75kPa. A higher applied stress, the proposed model overpredicts the settlements compared to those from the tests. This may be due to the changes of column and soil stiffnesses at higher stress levels during the course of tests which are not considered in the predictions. However, the differences between the predicted and the measured values for stresses over 75kPa, remain within tolerable limit.

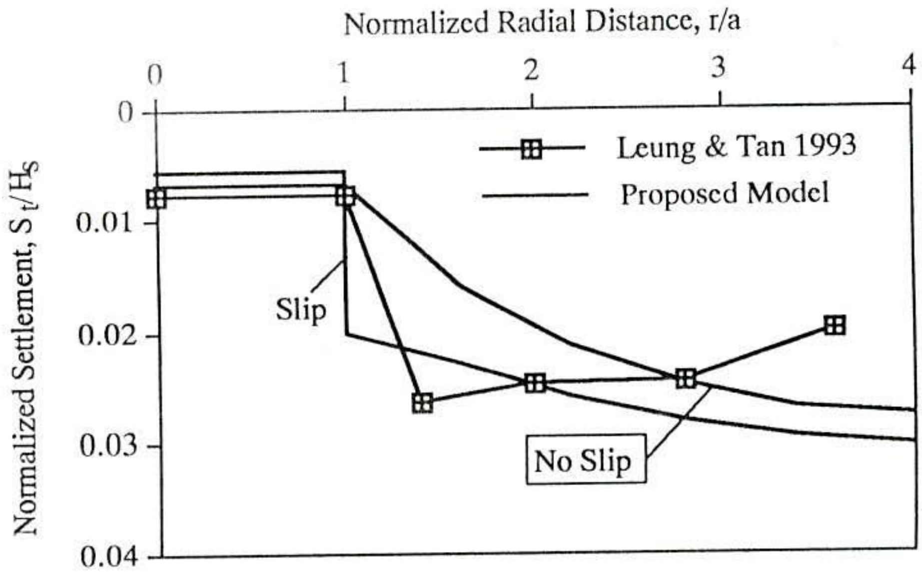


Figure 6.15 Prediction versus measurement of settlement profile of soft ground reinforced by sand column in the laboratory.

The tests conducted by Leung & Tan (1993) on sand columns through a unit cell concept and subjected to uniform surcharge i.e. flexible loading condition, are compared with the predictions by the proposed model. The test was performed on a circular steel tank having 1.0m diameter and 1.0m height and filled up with soft marine clay up to 0.5m from the bottom. The marine clay used had a composition of 15% sand, 40% silt and 45% clay. The clay was thoroughly remoulded at a water content of 90% using a larger mixer. The liquid limit and the plasticity index of the clay were 93% and 65%, respectively. The sand column having diameter of 0.25m is installed to full depth of clay at the center of the mold. The compacted sand used has a composition of 15% fine, 75% medium and 10% coarse and was compacted to a unit weight of 18 kN/m^3. The load is applied over the treated ground as a surcharge by dry sand of 0.5m height having unit 15 kN/m^3, which gave an equivalent stress of 7.50kPa. The required design parameters of column and soil are estimated based on the given information. The estimated values used for the predictions are p_o/E_s=0.0375, d_e/d_c =4.0, E_c/E_s=12, ν_s=0.40, ϕ=14° and $\delta=\phi$. The measured settlement on the surface along the radial distance at the end of primary consolidation is considered here for the comparison by the proposed model. The variation of normalized settlement, S_t/H_s, of the improved ground, with normalized radial distance, r/a, is presented in Fig.6.15. This figure shows that there exists a reasonable agreement between the measurements and the predictions except near the boundary of the mold. Near the surface of the mold the proposed model overpredicts the settlement compared to that from the experiment. It may be noted that the settlement increases from the center of column towards the outer surface of mold, but the same is not found in the reported test results. The investigators already pointed out that the effect of friction along the inner surface of the mold might reduce the magnitude of settlement near it. The predictions are made considering no slip and slip situations at the interface. It is observed the predictions considering slip at column-soil interface are closer to the measured values than no slip consideration.

6.4.2 Field test results

A series of test data available in literature are compared with the proposed model. As all the relevant data are not well documented, the average values of the parameters are considered for the predictions. For the predictions of settlement of reinforced ground by the proposed model the values of the parameters are taken as p_o/E_s=0.10, E_c/E_s=5 to 30 and ν_s=0.40. It is considered that the reinforced ground is covered by a layer of well compacted granular fill having reasonably high value of stiffness. This considerations ensures uniform settlement over the entire composite ground. The results are presented in Fig.6.16 in the form of settlement reduction ratio versus spacing ratio of columns. The data points are plotted for the measured values reported in the literature while the predictions by the proposed model are indicated as full lines for the variation of E_c/E_s from 5 to 30. The variation of settlement

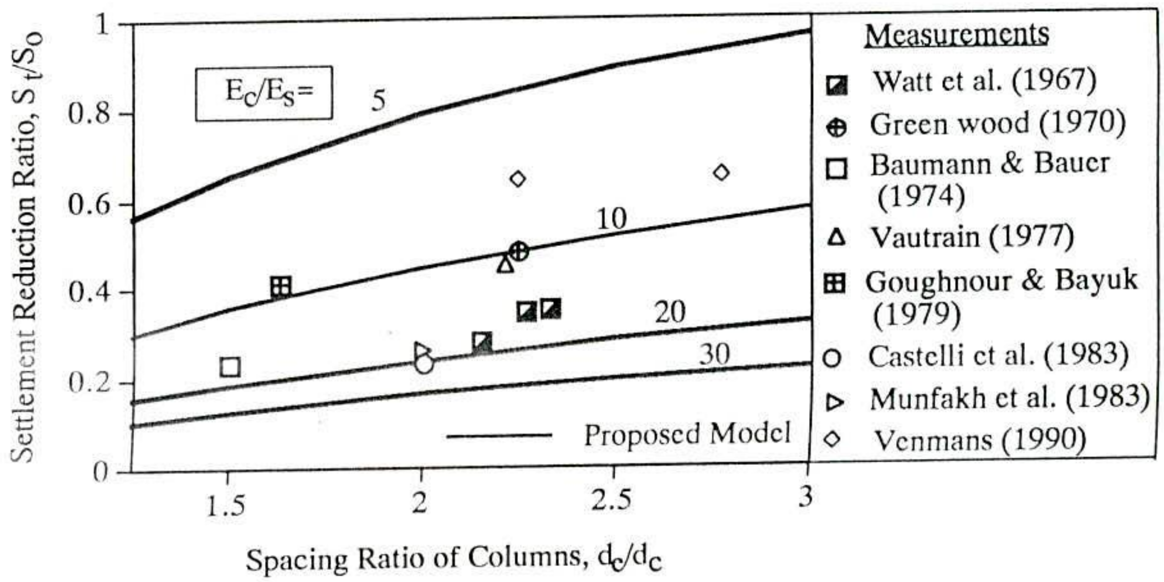


Figure 6.16 Validation of the proposed model for predicting the settlement reduction ratio.

reduction ratio with spacing for the test data is considerable. These variations may be due to the differences in site conditions and methods of granular column installation. While the results are not conclusive, since not enough well documented data are available from the field, these figures support the validity of the proposed model in the field scale.

6.5 Conclusions

The proposed foundation model has been compared with the existing approaches and verified by the finite element analysis as well as the experimental results from laboratory and field. It is revealed that the predictions by the existing methods are reasonably close to the proposed model in case of rigid loading. The methods based on the "equal strain" theory such as Priebe (1976), Aboshi et al. (1979) and Balaam and Booker (1981) give conservative results compared to the proposed model. The results obtained by the proposed model are closer to Balaam and Booker's (1981) results than those of the others. The predictions by the proposed method in case of flexible loading differ significantly from those predicted by Madhav & Van Impe (1994) but the predictions are close to each other for rigid loading. A reasonably good agreement is found while comparing the results obtained by the proposed model and Alamgir et al. (1995) for flexible loading. The predictions by the proposed method account for stress transfer along the column-soil interface and consider the displacement compatibility at every nodal points. In both flexible and rigid loading cases, the proposed model offers solutions accounting for the overlaying granular fill, end bearing and floating columns, slip and no slip situations and also for time-dependent analysis.

The comparison of results obtained from the finite element analysis and those from the proposed model, indicates that the proposed model can be used with a reasonable degree of accuracy to predict the settlement behaviour of end bearing and floating columns subjected to flexible and rigid loading. The predicted settlement profiles are found to be very similar to each other although they differ in magnitudes. The differences can be considered to be within tolerable limits. The prediction of interface shear stress by the proposed model compares well with the finite element analysis for both end bearing and floating columns. However, a significant differences in the variations of stresses in column are noticed specially for the case of floating column. The time-dependent predictions show that the results obtained from the proposed model differ with those obtained from the finite element analysis. These differences are expected as the finite element method uses Biot's three dimensional consolidation theory while the proposed model is based on the diffusion theory. But the differences are not so much, which indicates the applicability of the proposed model to estimate the time-dependent response of the reinforced ground.

The predictions obtained from the proposed model show good agreement with the results from both laboratory and field tests. From comparison, it is revealed that the laboratory test results for the cases of relatively rigid column i.e. lime column and deformable column i.e. sand column, can be predicted by the proposed method with a reasonable degree of accuracy.

The group effects can also be modelled reasonably by the proposed model since it compares well for predicting both the column in group and also in a unit cell. Comparison with field test results shows that the column-reinforced ground having different column stiffnesses and spacings can also be modelled by the proposed method.

CHAPTER SEVEN

SUMMARY AND CONCLUSIONS

The development of modern foundation practices, namely, ground improvement techniques, to overcome the limitations of the conventional foundation systems, has been proved to be viable both technically and economically for the improvement of marginal sites. Amongst the various ground improvement techniques for improving soft ground conditions, columnar inclusions are considered as one of the most versatile and cost effective method as the other methods such as preloading, dredging, dynamic compaction, thermal stabilization, ground freezing and soil replacement techniques can no longer be used due to environmental restrictions and post construction maintenance expenses. The columnar inclusions can be of the form such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., which are stiffer and stronger than the surrounding soil. This ground improvement technique has been and is being used in many difficult foundation sites throughout the world to increase bearing capacity, reduce settlement, increase the rate of consolidation, improve embankment stability and resistance to liquefaction.

Predictions obtained from the existing analytical and numerical methods and the analysis of the results from a large number of field and laboratory tests conducted till very recent times, reveal that there are various aspects that need to be considered for a rational design. These can be identified as the exact role of granular fill placed over the reinforced ground, the slip along the column-soil interface, the stress transfer between the column and the surrounding soil and the time-dependent behaviour resulting from the consolidation of soft ground. It is realized that still there is a strong need to develop a unified theoretical model which the practicing engineers can use to design such foundation types with a high degree of accuracy.

A theoretical foundation model, simple in concept and computations but versatile in application, is developed in the present study to analyze the overall settlement response of the soft ground reinforced by columnar inclusions. The reinforced ground is covered by a layer of granular fill and subjected to uniform loading over the entire area. The different types of columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., are categorized in a single foundation type from the standpoint of foundation analysis. The composite ground consists of stiffer and stronger columns and the surrounding soft soil. The backbone of the present analysis was developed by Poorooshasb and Bozozuk (1967) who, in turn, used a concept proposed by Hill (1963). It is a straight forward approach as it advocates the use of simple kinematically admissible displacement field and then attempts to obtain the overall equilibrium of the system. Based on this approach an axi-symmetric analysis is proposed in this study. In the present analysis, it is considered that the radial displacement component is considerably small and hence can be neglected. The rationality of this assumption is almost evident for the type of the problems considered in this analysis. The present approach can handle versatile aspects which often encountered for the case of soft ground improvement by columnar inclusions. The proposed model incorporates nonlinearity of the material properties, the interaction as well as stress transfer between the column and the surrounding soil along the depth, the flexibility of the granular fill, the possible slip along the column-soil interface and time-dependent behaviour resulting from the consolidation of the surrounding soft soil. An uncoupled solution is adopted to solve time-dependent response of the reinforced ground. In this solution, the "Diffusion Theory" which is an extension of Terzaghi's one dimensional consolidation theory is used to determine the degree of consolidation due to radial and vertical expulsion of pore water. The stress concentration due to the presence of stiffer column is not considered while computing the excess pore water pressure and the degree of consolidation at any time. The proposed model can handle the rather complicated situation such as certain types of material inhomogeneity (i.e. radial inhomogeneity), soil stratification, different column geometry (cylindrical and tapered as well) and end bearing as well as floating column conditions. The proposed model is compared with existing approaches, and verified by the finite element analysis. The results are also compared with those from laboratory and field tests. To illustrate the influences of various parameters on the predictions, parametric studies with a wide range of parameters are also carried out. The results are evaluated by developing a simple computer programme in Quick Basic, based on a numerical scheme developed for the solution of the proposed model. A personal computer is used to run the programme and which takes only few minutes to obtain the response. The results are presented in nondimensional form. The following conclusions can be drawn based on the predictions by the proposed model and from theoretical and experimental verifications.

(i) The foundation model developed in the present study is simple both in concept and computations but versatile in application and can handle rather complicated situations. It can be used for the predictions of column-reinforced ground as well as other similar geotechnical engineering problems with a high degree of accuracy considering all possible phenomena that may occur simultaneously at any loading stage, where most of the existing design approaches are not applicable.

(ii) Minimum number of design parameters are required for the predictions by the proposed method. They can be obtained easily from the conventional laboratory tests. The computation of numerical scheme needs relatively less time and can be implemented by a personal computer.

(iii) The parametric study shows that the proposed model predicts the behaviour over a wide range of parameters. It is observed that the effects of overlaying granular fill, spacing and length to diameter ratio of columns, relative stiffness of column and soil, degree of penetration of columns, are significant on the distribution of load sharing between the components of the system and the settlement of the treated ground but the Poisson's ratio of soil has little influence on them.

(iv) The compacted granular fill placed over the soft ground reinforced by columnar inclusions is very effective in reducing both the overall and the differential settlements. The compressibility of the granular fill has an appreciable influence on the settlement response of the treated ground as long as the modulus of the granular fill is less than approximately fifty times that of the soft ground. The predictions indicate that low thickness but well compacted granular fill is desirable in order to obtain the better performance.

(v) From the predictions considering possible slip at the column-soil interface, it appears that the length of the slip zone may be somewhat smaller than previously thought by other researchers. It is also found that the depth of slip zone increases with the decrease of degree of penetration of column.

(vi) The predicted result for the case of floating columns, shows that the neutral depth is not influenced by the degree of penetration of columns but it is different for slip and no slip situations. The presence of a strong soil layer at the tip of the column would have a significant influence on it as reflected from the predictions.

(vii) End bearing columns are found to be more effective than its floating counterpart in reducing the settlement of the improved ground.

(viii) The comparison of the results of time-dependent analysis by the proposed model and the finite element analysis, indicates that the "Diffusion Theory" can be used to consider the consolidation of the soft ground due to radial and vertical flow of water, as the differences between the predictions are remained within the tolerable limit despite the use of Biot's three dimensional consolidation theory for the finite element analysis by CRISP.

(ix) Comparisons of results between the proposed model and the existing methods, indicate that the existing methods may be used for the prediction in case of rigid granular fill placed over the entire area of column-reinforced soft ground. But their application for the case of flexible loading is not valid. The proposed model offers better solution for any loading situation ranging from flexible to rigid.

(x) While comparing laboratory and field test results with the predictions by the proposed model, it is observed that agreement between the two is reasonable. These findings reveal that the proposed model can be used with a reasonable degree of accuracy to predict laboratory as well as field test results.

In a nutshell, this dissertation can be concluded as: a theoretical model is proposed to solve an important class of getechnical engineering problems e.g. settlement response of soft ground reinforced by columnar inclusions. This foundation model is simple in concept but versatile in applications, takes minimum computational efforts and can handle rather complicated situation such as certain types of inhomogeneity, possible slip along the column-soil interface, soil stratification, end bearing and floating columns, flexible to rigid loading conditions and also time-dependent behaviour resulting from the consolidation of surrounding soil.

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APPENDIX I

DERIVATION OF EQUILIBRIUM EQUATION

In order to specify completely the state of stress at a point in a continuous medium, it is essential to specify the components of the stress at a given point, acting on three mutually orthogonal planes passing through the point. These arbitrary planes are usually taken perpendicular to the co-ordinate directions of some orthogonal co-ordinate system. For instance, this co-ordinate systems can be either Cartesian system (x,y,z) or Cylindrical co-ordinate system (r,θ,z). The stresses acting on a differential soil element are shown in Fig.I.1 in the cylindrical co-ordinate system.

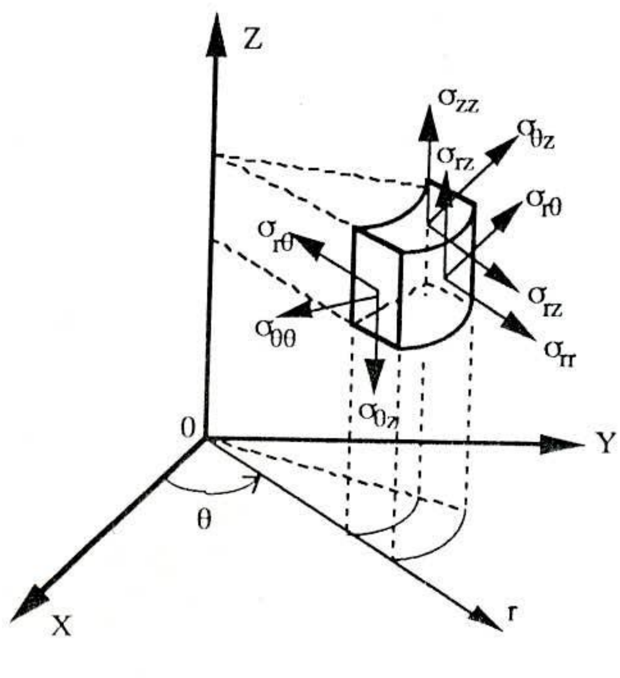

Figure I.1 The stress componets acting on a infintesimal element.

The stress components acting on this soil element can be identified as the following terms.

$$\begin{matrix} \sigma_{rr} & \sigma_{r\theta} & \sigma_{rz} \\ \sigma_{\theta r} & \sigma_{\theta\theta} & \sigma_{\theta z} \\ \sigma_{zr} & \sigma_{z\theta} & \sigma_{zz} \end{matrix} \tag{I.1}$$

Let the edges of the soil element are dr, rdθ and dz. If the soil element is in equilibrium and at rest (inertia forces are assumed to be absent) and a normal stress σ_{zz} at on of its face, at the opposite face there will be a stress of $\sigma_{zz}+\Delta\sigma_{zz}$ of opposite sign. The intensity of variation of a function with that of a variable is the derivative of the function, with respect to the argument. Therefore, in the present case, the increment of stress σ_{zz} by a unit length is $[\partial\sigma_{zz}/\partial_z]$ and hence the increment through the length dz is

$$\Delta\sigma_{zz} = \frac{\partial\sigma_{zz}}{\partial z}dz \tag{I.2}$$

Thus the normal stress in z direction at the opposite face is

$$\sigma'_{zz} = \sigma_{zz} + \frac{\partial\sigma_{zz}}{\partial z}dz \tag{I.3}$$

Similarly the stress on the opposite faces, can be given as

$$\begin{matrix} \left[\sigma_{rr} + \frac{\partial\sigma_{rr}}{\partial r}dr, \quad \sigma_{r\theta} + \frac{\partial\sigma_{r\theta}}{\partial r}dr, \quad \sigma_{rz} + \frac{\partial\sigma_{zr}}{\partial r}dr,\right] \\ \left[\sigma_{\theta\theta} + \frac{\partial\sigma_{\theta\theta}}{\partial\theta}rd\theta, \quad \sigma_{\theta r} + \frac{\partial\sigma_{\theta r}}{\partial\theta}rd\theta, \quad \sigma_{rz} + \frac{\partial\sigma_{\theta z}}{\partial\theta}rd\theta,\right] \\ \left[\sigma_{zz} + \frac{\partial\sigma_{zz}}{\partial z}dz, \quad \sigma_{zr} + \frac{\partial\sigma_{zr}}{\partial z}dz, \quad \sigma_{z\theta} + \frac{\partial\sigma_{z\theta}}{\partial z}dz,\right] \end{matrix} \tag{I.4}$$

Since the soil element is assumed to be in equilibrium, by resolving the forces exerted on the infinitesimal element in the z direction, the following equation can be written.

$$\left[\sigma_{zz}+\frac{\partial\sigma_{zz}}{\partial z}dz\right]rd\theta dr-\sigma_{zz}rd\theta dr+\left[\sigma_{rz}+\frac{\partial\sigma_{rz}}{\partial\theta}\right]drdz-\sigma_{rz}drdz+$$
$$\left[\sigma_{rz}+\frac{\partial\sigma_{rz}}{\partial r}dr\right]rd\theta dz-\sigma_{rz}rd\theta dz+p_z rd\theta drdz=0 \qquad \text{(I.5)}$$

where p_z is the body force in z-direction. By simplifying Eq.(I.5), one gets the following equation

$$\frac{\partial\sigma_{rz}}{\partial r}+\frac{\partial\sigma_{zz}}{\partial z}+\frac{1}{r}\frac{\partial\sigma_{\theta z}}{\partial\theta}+\frac{\sigma_{rz}}{r}+p_z=0 \qquad \text{(I.6)}$$

when the stress system is symmetric about z axis, $\partial/\partial\theta$ terms become zero and $\sigma_{r\theta}=0$. For this special case, in the absence of body forces, Eq.(I.6) reduces to

$$\frac{\partial\sigma_{zz}}{\partial z}+\frac{\partial\sigma_{rz}}{\partial r}+\frac{\sigma_{rz}}{r}=0 \qquad \text{(I.7)}$$

which is the required equilibrium equation.

APPENDIX II

DETERMINATION OF DEFORMATION MODULUS

Typical e-logp curve on undisturbed and remoulded clay in a odeometer test is shown in Fig.II.1.

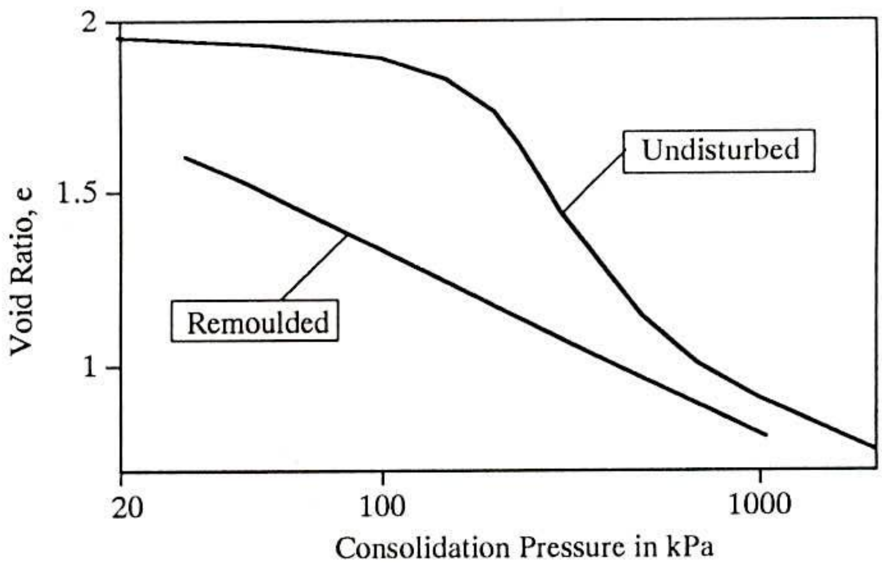

Figure II.1 Typical odeometer test results.

Consider the element at a depth z at which position the effective vertical stress is obviously $\sigma'_v=\gamma' z$. Thus assuming an e-log(p) in the form;

$$e = e_0 - \lambda\, ln(\sigma'_v) \tag{II.1}$$

where $\lambda = C_c \, log\underline{e}$; where $\underline{e}$ is the base of natural logarithm, the initial void ratio of the element (i.e. void ratio before application of the load) is evaluated as;

$$e_i = e_0 - \lambda ln(\gamma' z) \tag{II.2}$$

After the consolidation by the surcharge p_o the void ratio changes to its final value as;

$$e_f = e_0 - \lambda \, ln(\gamma' z + p_o) \tag{II.3}$$

Thus the soil experience a strain of magnitude ε as;

$$\varepsilon = \frac{\lambda ln\left[\dfrac{\gamma' z + p_o}{\gamma' z}\right]}{\left[1 + e_i\right]} = \frac{\lambda ln\left[\dfrac{\gamma' z + p_o}{\gamma' z}\right]}{\left[1 + e_0 - \lambda ln(\gamma' z)\right]} \tag{II.4}$$

The function $E(z)$ is now determined as ;

$$E(z) = \frac{p_o}{\varepsilon} = \frac{p_o\left[1 + e_0 - \lambda \, ln(\gamma' z)\right]}{\gamma \, ln\left[\dfrac{\gamma' z + p_o}{\gamma' z}\right]} \tag{II.5}$$

PUBLICATIONS: JOURNALS AND CONFERENCES

(During April, 1993 ~ March, 1996)

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   M. Alamgir, N. Miura, H.B. Poorooshasb and M.R. Madhav
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2. Negative Skin Friction on Rigid and Deformable Piles
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