# Improving Granular Column Capacity by Geogrid Reinforcement

M. R. Madhav
*Indian Institute of Technology, Kanpur, India*

M. Alamgir & N. Miura
*Saga University, Japan*

ABSTRACT: Granular columns can be reinforced by geogrids or related products to get further improvements in their performance. This paper presents a method for estimating the contributions of reinforcement to the ultimate capacity and stiffness of a reinforced granular column. The reinforcement is provided in the form of layers at a given spacing in the top region of granular column material to prevent bulging failure. The restraint offered to the granular material by the reinforcement, prevents the lateral deformations i.e. bulging and thus increases the ultimate capacity and the stiffness of the reinforced granular column. The present analysis shows that the ultimate capacity and stiffness of reinforced column increase with the number of reinforcement layers. The predictions based on the proposed approach agree well with the small scale in-situ test results presented by Madhav (1982).

## 1 INTRODUCTION

Installation of granular columns (usually referred to as Stone columns or Granular piles) has become a common practice for ground improvement throughout the world to increase the bearing capacity, reduce settlement, improve slope stability and resistance to liquefaction in soft clays or loose granular deposits. Granular columns in end bearing conditions are cost effective and can be installed rapidly using vibro-replacement, compozer, rammed stone column techniques and even by heavy tamping (Van Impe & Madhav, 1992). Bergado et al. (1991) presents a recent state of art summary of work on granular columns - installation, design and analysis. When the granular column reinforced ground is being loaded, the columns deform laterally into the surrounding soft soil strata while distributing the stresses at the upper portion of the soil layer. For most granular columns whose lengths are greater than the critical length, it is recognized that the bulging failure governs the load carrying capacity whether they bear on a stiff layer or penetrate partially into the medium stiff soil (Madhav & Miura, 1994). Experiments on granular columns reveal that bulging occurs near the top at a depth equal to approximately one to one and half diameters of the column. The lateral deformations decrease with depth and tend to be negligible beyond a depth greater than two to three diameters of the column (Hughes & Withers, 1974). This type of failure restricts the ultimate capacity of granular column to a lower value irrespective of the stiffness of the constituent materials. In use, the granular column relies on the lateral support provided by the surrounding soil mass. If part or all of the lateral support is lost during any period of the life of structure, the loaded granular column may collapse and the structure suffer distress. Hence, modification as well as improvement of the conventional granular column becomes imperative.

The performance of the conventional granular column can be improved upto a certain extent by strengthening the column at the top region to prevent bulging or below that depth to get additional stiffness. In Europe, for some applications, cement has been added to form a rigid granular column (Barksdale & Bachus, 1983). Rao and Bhandari (1977) suggest skirting the granular column while Madhav (1982) proposes a concrete plug or reinforcement of the granular column. Installation of granular columns in soft soils especially those with undrained cohesion less than 10 kPa, is very difficult as the granular material is lost into the surrounding soil. Reinforcement of the granular column particularly in its top two to three diameter, would minimise this loss as the reinforcement prevents the lateral flow of the granular material. Alamgir (1989) suggests jacketing the granular column by wrapping a geosynthetic liner around the whole surface area of column. Ayadat and

Hanna (1991) suggest enveloping the granular material in a membrane in order to strengthen the granular column. All of these alternatives lead to a significant improvement of column capacity over that of its conventional counterpart and ensure its better performance even in extreme ground conditions.

In this paper, a method for the estimation of ultimate capacity and stiffness of geogrid reinforced granular column is proposed. The reinforcement (geogrid or a related product) is provided in the form of layers at a given spacing in the top region of the granular column where bulging is likely to occur. The reinforcement can be placed easily within the granular material by any of the existing installation techniques and the drainage function of the column unhindered. The improvement of the column capacity is achieved through the interaction between the granular material and the reinforcement. The effect of the interaction shear stresses is to generate tensile stresses in the reinforcement and to restrain laterally the granular column material through these boundary shear stresses. 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 increases with an increase of lateral confinement as well as due to reinforcing effect. The stiffness of the plain or reinforced granular column is evaluated using an approach proposed by Duncan and Chang (1970).

From the results evaluated by the proposed method of analysis, it is found that the ultimate capacity and the stiffness of reinforced column increases with the number of reinforcement layers, the angle of shearing resistance of column material and the coefficient of frictional resistance of reinforcement. The improvement also depends on the spacing of reinforcement layers and the depth of location of bulging failure. Comparing in–situ test results on the reinforced granular column performed elsewhere, it is found that the proposed method predicts the ultimate capacity and stiffness of the reinforced column with a reasonable degree of accuracy.

## 2 ANALYSIS

A single, isolated reinforced granular column (Fig.1) of diameter, $d_p$(=2a), and length, H, surrounded by a soft soil media resting on a smooth rigid base and loaded through a rough rigid footing is considered here for the analysis. The reinforcement is provided in layers within the dense granular material all over the area of the column at a spacing, s. The top reinforcement layer is placed at a depth, $u_o$, from the top of the reinforced granular column. The main features of the reinforced granular column and the stresses acting on it are shown in Figs.1(a),(b) and (c). The method of estimation of ultimate capacity, reinforcement–granular material interaction characteristics and the stiffness of the granular column are discussed in the following sections.

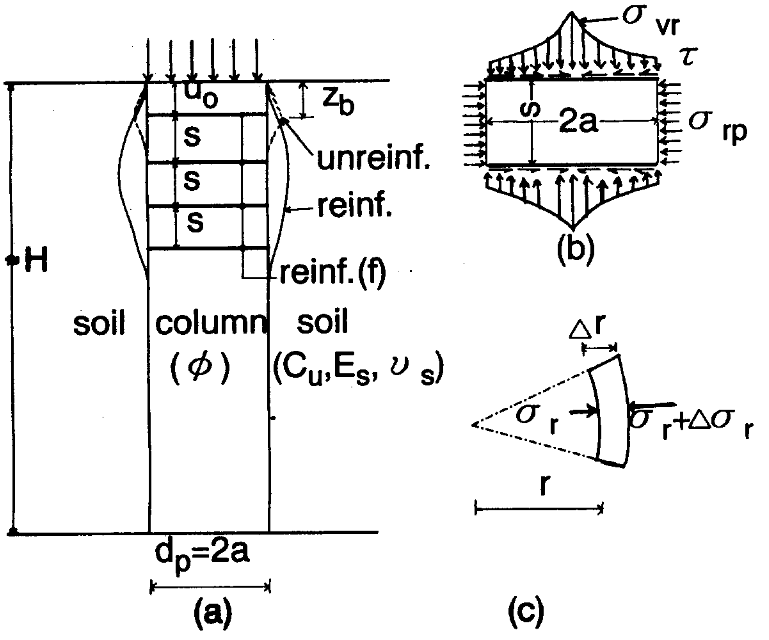

Fig.1 Definition sketch

### 2.1 Ultimate capacity of granular column

For most single, isolated granular columns whose lengths are greater than a critical length, it is recognized that bulging failure is the controlling mechanism whether they are floating, bear on a stiff layer or penetrate partially into the medium stiff soil. Most of the analytical solutions assume that a triaxial state of stress exists in the granular column and that both the column and the surrounding soil are at failure. In this analysis it is considered, that the lateral confining stress which supports the granular column equals the ultimate passive resistance which the surrounding soils mobilize as the column bulges outward against it. The passive resistance developed by the surrounding soil as a first approximation can be better modeled as an infinitely long cylinder which expands about the axis of symmetry until the ultimate passive resistance is developed. The approach of Hughes and Withers (1974) can be used and the following equation is obtained considering $C_u$ and $E_s$ to increase linearly with depth.

$$\sigma_{rp}(z)=\sigma_{ro}(z)+C_u(z)[1+\ln\frac{E_s(z)}{2C_u(z)(1+\nu_s)}] \qquad (1)$$

where, $\sigma_{rp}(z)$= ultimate lateral stress at a depth z; $\sigma_{ro}(z)$= initial in–situ lateral stress at a depth z; $E_s(z)= E_o+K_e z=$

elastic modulus of soil at a depth z; $E_o$ = elastic modulus of soil at the top of granular column; $k_e$ = constant signifying the increase of elastic modulus of soil with depth; $C_u(z) = C_{uH}(z/H)$ = undrained shear strength of soil at a depth z; $C_{uH}$ = undrained shear strength at depth H; $\upsilon_s$ = Poisson`s ratio of soil.

Since bulging failure governs the ultimate capacity of the column, $\sigma_{rp}$, is calculated taking the value of z as $z_b$, where $z_b$ is the average depth at which bulging is likely to occur for the unreinforced granular column. In this analysis it is assumed that the granular column is in a state of limiting equilibrium. Therefore, the ultimate vertical stresses, $q_{up}$, which the column can bear is

$$q_{ur} = \sigma_{rp} k_p \tag{2}$$

where, $k_p$ = the coefficient of passive earth pressure = $(1+\sin\phi)/(1-\sin\phi)$; $\phi$ = angle of internal friction of granular material.

## 2.2 Interaction between granular material and reinforcement

The reinforcement is provided in layers within the dense granular material (Fig.1a). When the reinforced granular column is being loaded, the column deforms laterally into the surrounding soil strata while transmitting the stresses at the upper portion of the soil layer. Due to the difference in their moduli of deformation, shear stresses are mobilized at the interfaces between the granular material and the reinforcement (Madhav, 1992). The mobilized shear stresses depend on: frictional properties of reinforcement–granular material interface (frictional coefficient = f); mechanical properties of granular material (angle of internal friction = $\phi$); spacing of reinforcement and the vertical and radial stresses acting on the column. These interface shear stresses provide extra confinement which prevents the lateral deformation and thus the bulging failure of granular column. Since the center of column gets more confinement than the outer parts, the maximum confinement effect is offered by the reinforcement at the center. The additional confinement stress decreases with radial distance and becomes equal to $\sigma_{rp}$ at the edge of the reinforcement. The granular column material confined within the two reinforcement layers and the surrounding soft soils are shown in Fig.1b. The applied vertical stress, the mobilized radial stress and the shear stresses developed at the interface of granular materials and reinforcement are also shown in this figure. The stresses acting on a differential element of granular materials at a radial distance, r, from the center of column between the two reinforcement layers are shown in Fig.1c. From the equilibrium of horizontal forces acting on the differential element, the following equation is obtained.

$$\sigma_{rr} r\Delta\theta - (\sigma_{rr} + \Delta\sigma_{rr})(r+\Delta r)(\Delta\theta) = 2\tau(r\Delta\theta)\frac{\Delta r}{s} \tag{3}$$

where, $\sigma_{rr}$ = lateral stress on the reinforced granular column at a radial distance r; $\Delta\sigma_{rr}$ = differential increment of radial stress; $\tau$ = shear stress at the interface of granular material and reinforcement; s=spacing of reinforcement layers and $\Delta r$ and $\Delta\theta$ are the differential radial distance and angle respectively.

Since full passive resistance is mobilized, the vertical stress ($\sigma_{vr}$) on reinforced granular column is expressed as, $\sigma_{vr} = \sigma_{rr} N_\phi$, where $N_\phi = \tan^2(45^\circ + \phi/2)$. The mobilized shear stresses at the granular materials–reinforcement interface depend on the vertical stress ($\sigma_{vr}$), frictional coefficient (f) and the angle of internal friction ($\phi$) and is expressed as the $\tau = \mu\sigma_{vr}$, where $\mu = f\tan\phi$. Using these relationships and by integrating Eq.3, the following equation is derived after proper simplification

$$\sigma_{vr} + \sigma_{vr}\ln(r) + 2\mu N_\phi \sigma_{vr}\left(\frac{r}{s}\right) = C_1 \tag{4}$$

where $C_1$ is a constant. The boundary conditions are: at the outer boundary of the granular column i.e. at r=a, $\sigma_{rr} = \sigma_{rp}$ and $\sigma_{vr} = \sigma_{vp}$, where $\sigma_{rp}$ and $\sigma_{vp}$ are the radial and vertical stresses for plain granular column. Using this boundary condition and solving Eq.4 for the constant $C_1$, the following equation is obtained to predict the variation of mobilized vertical stresses with the radial distance acting on the column area.

$$\sigma_{vr} = \frac{2\mu N_\phi(a/s) + \ln(a) + 1}{2\mu N_\phi(r/s) + \ln(r) + 1}\sigma_{vp} \tag{5}$$

Eq.5 is used for evaluate the confinement effect on reinforced granular column between two layers of reinforcement. For a single layer of reinforcement the following equation is obtained by replacing s by $u_o$ in Eq.4 to predict the confinement effect on reinforced granular column.

$$\sigma_{vr} = \frac{\mu N_\phi(a/u_o) + \ln(a) + 1}{\mu N_\phi(r/u_o) + \ln(r) + 1}\sigma_{vp} \tag{6}$$

Once the distribution of vertical stresses due to the confinement effect of reinforcement is known, the ultimate capacity of reinforced granular column is obtained by integrating the vertical stresses acting on it, as

$$q_{ur}=\frac{1}{A_c}\int_0^{2\pi}\int_0^{a}\sigma_{vr}\,dr\,dA \tag{7}$$

where, $A_c$ is the plan area of granular column and $q_{ur}$ is the ultimate capacity of reinforced granular column.

### 2.3 Stiffness of granular column

The stiffness of the granular column varies with the state of stress developed within the column. The reinforced granular column is stiffer than its plain counterpart due to the greater confinement obtained from the mobilization of shear stresses along the interface of granular materials and reinforcement. As the number of reinforcement layers increases, the lateral confinement as well as the vertical stresses on column increase leading to an increase of the stiffness of the reinforced granular column. The stiffness of the plain and reinforced granular columns is evaluated using the approximate method developed by Duncan and Chang (1970) based on the hyperbolic representation of nonlinear behavior.

$$k_r=k\sigma_\theta^n\left[1-\frac{(1-\sin\phi)(\sigma_v-\sigma_r)R_f}{2(c\cos\phi+\sigma_r\sin\phi)}\right] \tag{8}$$

where, $k_r$ = stress dependent stiffness parameter of the granular column; k and n = constants defining the initial stiffness; c and $\phi$ = cohesion and angle of internal friction of granular material; $R_f$=failure ratio; $\sigma_\theta=\sigma_v+2\sigma_r$. In the absence of specific test data, the following constants are used for soft clays: k = 67.62; n = 1.14; $R_f$=0.86, where $k_r$ and $\sigma_\theta$ are in kPa.

## 3 RESULTS AND DISCUSSIONS

Predictions made for a typical reinforced granular column are compared with a series of in-situ test results in order to evaluate the ability of the method to model the actual behavior of reinforced granular column effectively. Results for a limited range of parameters and the improved response of a reinforced granular column over its plain counterpart are presented in Figs. 2–7.

The variation of normalized vertical stresses ($\sigma_{vr}/\sigma_{vp}$) with normalized radial distance (r/a) of reinforced granular column are shown in Fig.2. The results are given for spacing ratio ($s/d_p$) of 1.0, $\phi$ equal to 40° to 50° and f equal to 0.8 to 1.2. The confinement effect offered by the reinforcement is maximum at the center of the column. The normalized vertical stresses decrease with radial distance and become unity at r/a=1. This figure also reveals that the vertical stresses on reinforced granular column increase with the increase of $\phi$ and f.

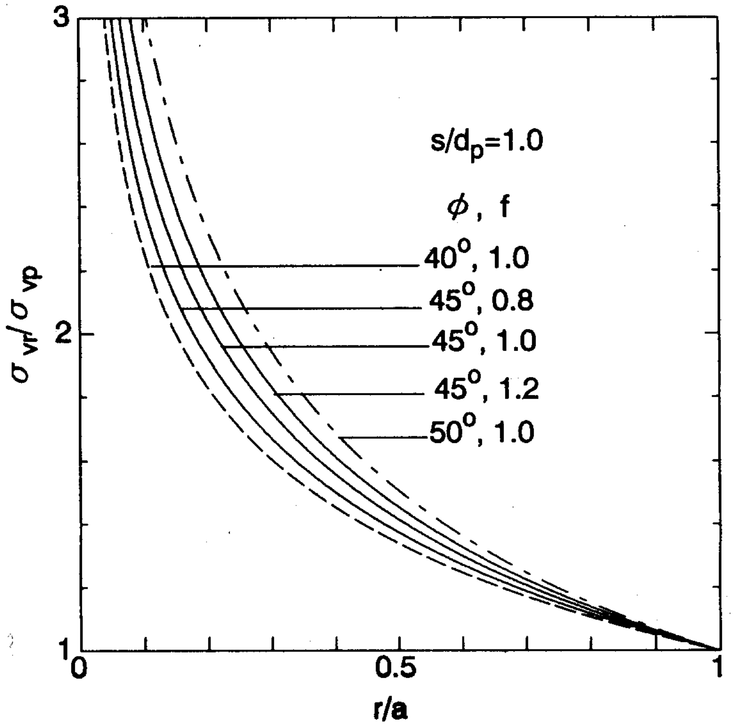

**Fig.2 Variation of vertical stresses with radial distance**

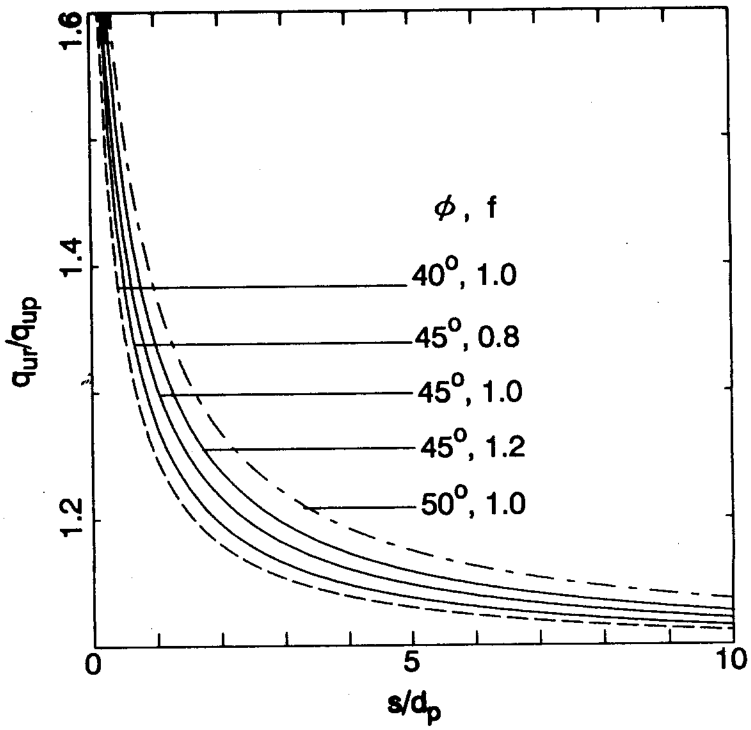

**Fig.3 Normalized vertical stresses on granular column with the spacing ratio of reinforcement layers**

The variation of normalized ultimate capacity of reinforced granular column ($q_{ur}/q_{up}$) with the spacing of reinforcement layers for different $\phi$ and f is shown in Fig.3. The values of $\phi$ and f vary from 40° to 50° and 0.8 to 1.2 respectively. The improvement in reinforced granular

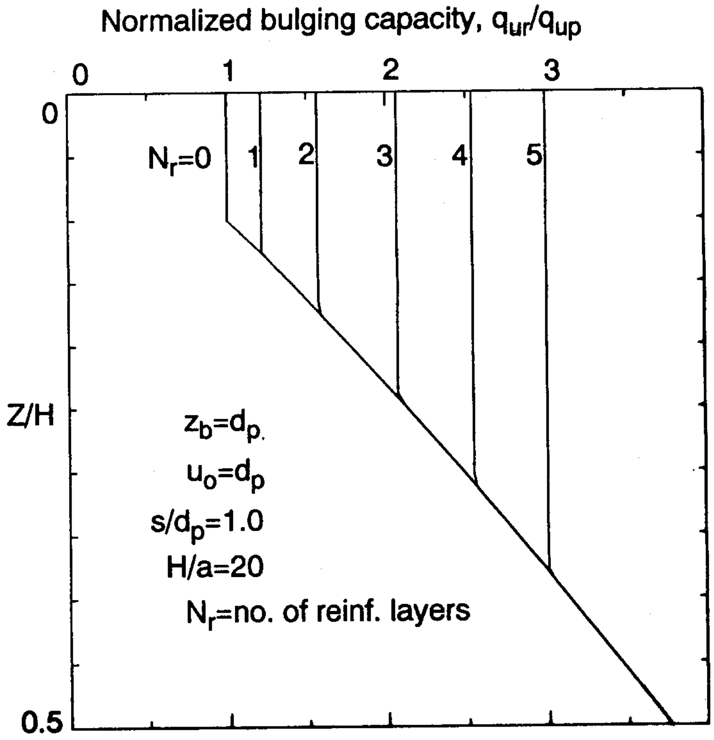

Fig.4 Variation of bulging capacity of granular column with depth and the number of reinforcement layers

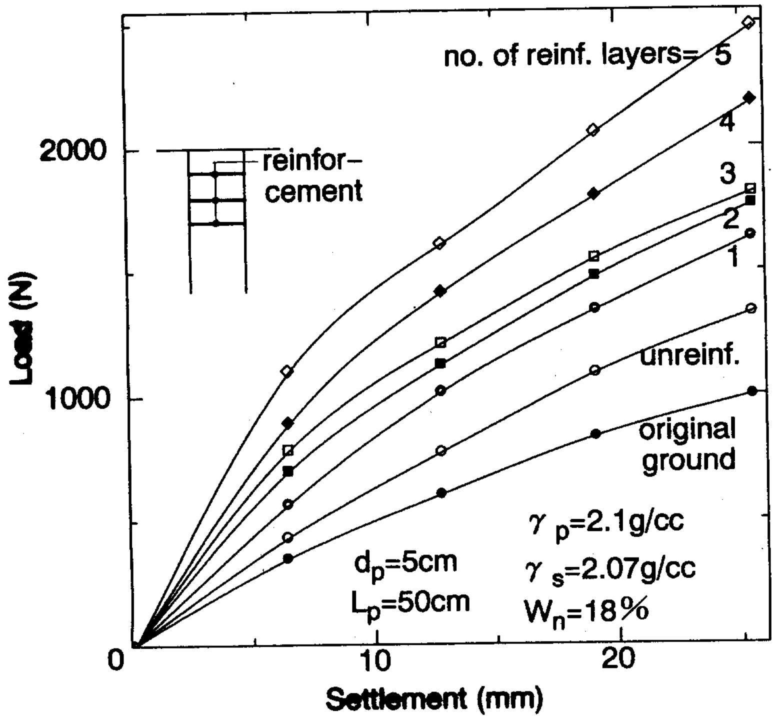

Fig.5 Small scale in–situ test results of unreinforced and reinforced granular column (after Madhav, 1982).

column capacity is high for closer spacing ( $s/d_p<1$) but the capacity decreases sharply with the increase of spacing ratio. This figure also reveals that the effectiveness of reinforcement on granular column capacity increases with increase of $\phi$ and f. Figs.2 and 3 indicate that the reinforced granular column has relatively high capacity for closer spacing and higher values of $\phi$ and f.

The variations of maximum bulging capacity of reinforced granular column with depth and the number of reinforcement layers ($N_r$) are depicted in Fig.4. The capacity of the reinforced granular column is governed by the capacity of the reinforced part or the least value of the unreinforced portion. The limiting value is at a depth where the two capacities equal each other. These variations are evaluated for $z_b=d_p$, $u_o=d_p$, $H/a=20$, $s/d_p=1$ and $N_r=1$ to 5. It is found that the bulging capacity of the granular column increases with the increase in number of reinforcement layers as the depth at which bulging failure is possible to occur, increases.

The applicability of the present analysis is verified by comparing the evaluated results with available in–situ test results on reinforced granular column. In this regard test results of Madhav (1982) on reinforced granular column are used for the comparison of theoretical and test results. These in–situ test results are replotted and shown in Fig.5. The figure shows that the ultimate capacity and stiffness of granular column increase with the increase in number of reinforcement layers.

The predictions by the present analysis and test results stated above are shown in Figs 6 and 7. In Fig.6, the variations of ultimate capacity of reinforced granular column normalized with respect to a plain granular column with $N_r$ from small scale in–situ tests are compared with the predicted values. By the proposed method, the predictions are made for $z_b = d_p$, $1.5d_p$ and $2d_p$. The first layer of reinforcement are placed at a depth, $u_o=d_p$, $1.5d_p$ and $2d_p$ respectively. The spacing ratio for the reinforcement layer is considered as $s/d_p=1.0$. This figure

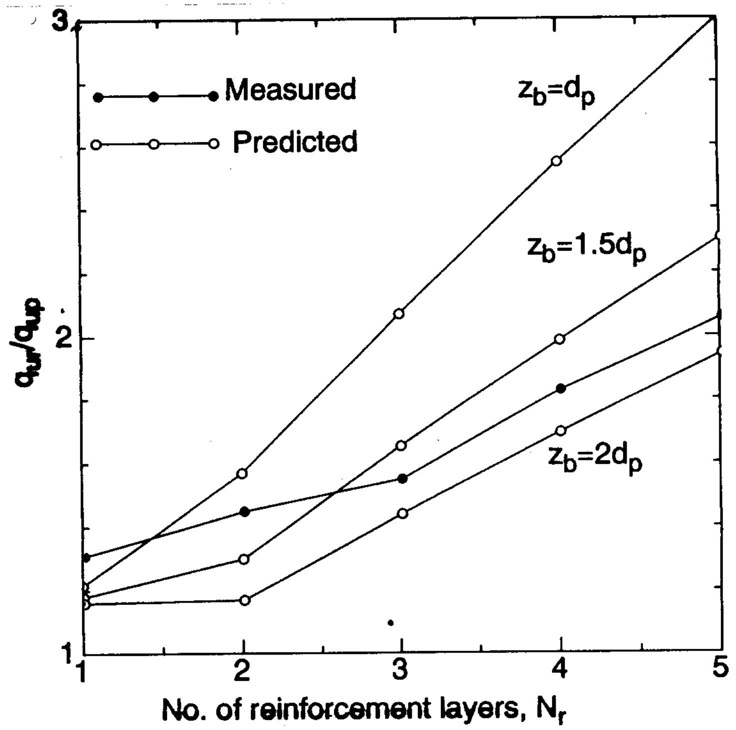

Fig.6 Comparison between predicted and measured ultimate capacity of reinforced granular column

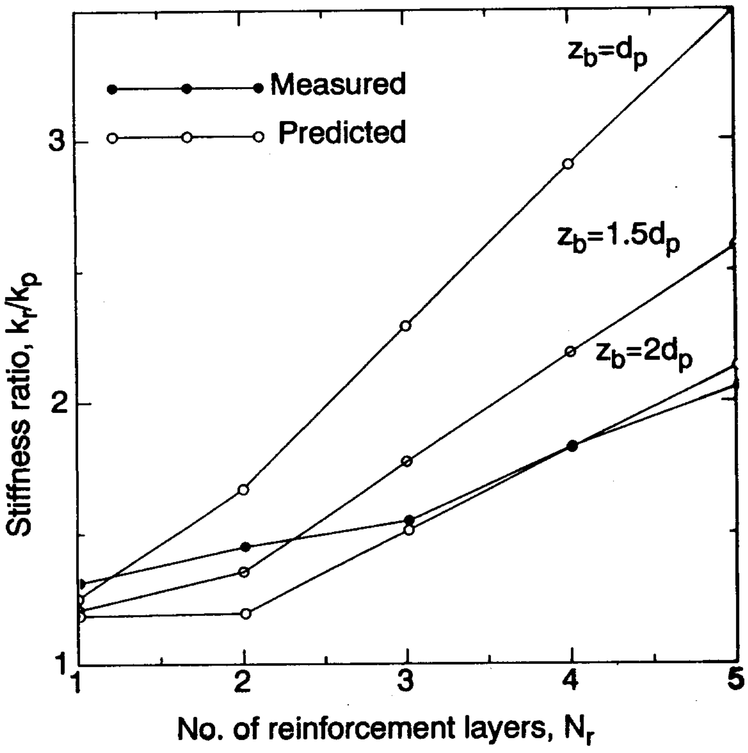

Fig.7 Comparison between the predicted and measured stiffness of reinforced granular column

shows that the predictions underestimate the granular column capacity ratio by about 10% compared to the values from tests for $N_r$ equal to 1.0. At higher value of $N_r(\geq 2)$, the test results are straddled by the predicted curves for $z_b=1.5d_p$ and $2d_p$ indicating that failure possibly is occurring at $z_b=(1.75$ to $1.80)d_p$.

In Fig.7, the predicted variations of stiffness ratio of reinforced to plain granular columns with $N_r$ are compared with those of test results. The trends in the improvement of stiffness from predicted and test results are the same as noted in Fig.6 for the improvement of ultimate capacity of granular column. The predictions match better if the bulging failure for unreinforced granular column is considered to at a depth, $z_b=2d_p$ in case of $N_r$ = 3 to 5. The results presented in Figs.6 and 7 indicate that the most important parameter 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.

## 4 CONCLUSIONS

Reinforcement of granular columns by geosynthetics or related products can lead to further improvement of their performance. A method to predict the ultimate capacity and the stiffness of geogrid reinforced granular column is presented. The tendency for bulging of the granular column is restrained by the confinement effect of geogrid reinforcement as it mobilizes tensile resistance. Both the ultimate capacity and the stiffness of the granular column increase with increasing number of reinforcement layers and higher values of column material and geogrid interface frictional resistance and with closer spacing. The predicted results compared well with those from small scale in-situ tests.

## REFERENCES

Alamgir, M. (1989) *Analysis and design of plain and jacketed stone columns in clays,* M.Sc.Engg. thesis, Department of Civil Engineering, BUET, Dhaka, Bangladesh.

Ayadat, T and Hanna, T.H. (1991) Performance of vibro columns in collapsible soils, *Proc. of IGC–91, geotechnical analysis practice and performancees,* Surat, India, 383–386.

Barksdale, R.D. and Bachus, R.C. (1983) *Design and construction of stone columns, Volume I,* U.S.A. Department of Transportation, Federal Highway Administration Report No. FHWA/RD–83/026.

Bergado, D.T., Alfaro, M.C. and Chai, J.C. (1991) The granular pile: its present state and future prospects for improvement of soft Bangkok clay, *Geotechnical Engineering,* J. of SEAGS, 22:143–173.

Duncan, J.M. and Chang, C.Y. (1970) Nonlinear analysis of stress and strain in soils, *J. of Geot. Engrg.* ASCE, 96:1625–1653.

Hughes, J.M.O. and Withers, N.J. (1974) Reinforcing of soft cohesive soils with stone columns, *Ground Engineering,* 7:42–49.

Madhav, M.R. (1982) Recent development in the use and analysis of granular piles, *Proc. of Symposium on Recent Development in Ground Improvement Techniques,* Bangkok,117–129.

Madhav, M.R. (1992) Elastic deformations of reinforced soil under triaxial stress conditions, *Indian Geotechnical Journal,* 22:110–122.

Madhav, M.R. and Miura, N. (1994) Soil improvement, panel report on stone columns, *Proc. of the XIII ICSMFE,* New Delhi, India, 5:163–164.

Rao, B.G. and Bhandari, R.K. (1977) Reinforcing of non–cohesive soil by granular piles, *Proc. of 6th ARC on SMFE,* Singapore, 1:175–178.

Van Impe, W.F. and Madhav, M.R. (1992) Analysis and settlement of dilating stone column reinforced soil, *Osterreichische Ing. und Arch. –Zeitschrift,*137:114–121.