Time Dependent Behavior of Geosynthetic Reinforced Granular Fill-Soft Soil System Subjected to Strip Loading

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Introduction

In the early development of soil reinforcement technology mainly steel was used as reinforcing material but now geosynthetics have made obsolete almost all other soil reinforcing materials used earlier. There are many situations where the weak soil has to be improved before any construction on it. A compacted granular fill with one or two layers of reinforcement is often placed on soft soil to transfer the load on weak soils. Use of granular fill containing reinforcements effectively reduces the settlement and increases the bearing capacity of the soft foundation soil. Such geosynthetic reinforced granular fills placed on soft soil are often used as foundation for unpaved roads, shallow footings, low embankments, oil drilling platforms, heavy industrial equipment, etc. (Pinto, 20003). Several theoretical and experimental studies have been done in the area of reinforced soil-foundation interaction. Some of the analytical models are due to Madhav and Poorooshasb (1988, 1989), Ghosh (1991), Shukla and Chandra (1994a, 1994b, and 1994c), Yin (1997). A large number of model tests have also been conducted to bring out the effects of various parameters on the load carrying capacity and settlement characteristics of the reinforced systems using geosynthetics.

In this work, the model proposed by Yin (1997) is extended to incorporate the effect of time dependent behavior of soft clay in the response of the model. In this model, some of the input parameters such as coefficient

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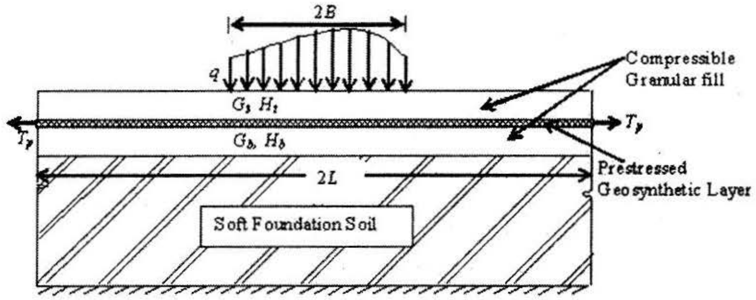

FIGURE 1 : Definition Sketch of Geosynthetic Reinforced Granular Fill-Soft Soil System

of friction between reinforcement and granular fill and lateral stress ratio are eliminated, and at the same time, the consolidation of the soft soil and the stiffness of geosynthetic reinforcement are taken into consideration.

Definition and Formulation of the Problem

A geosynthetic-reinforced granular fill on soft soil is shown in Fig.1. A geosynthetic membrane has been used as reinforcement in the granular fill. A mechanical model as shown in Fig.2 idealizes above-mentioned foundation system.

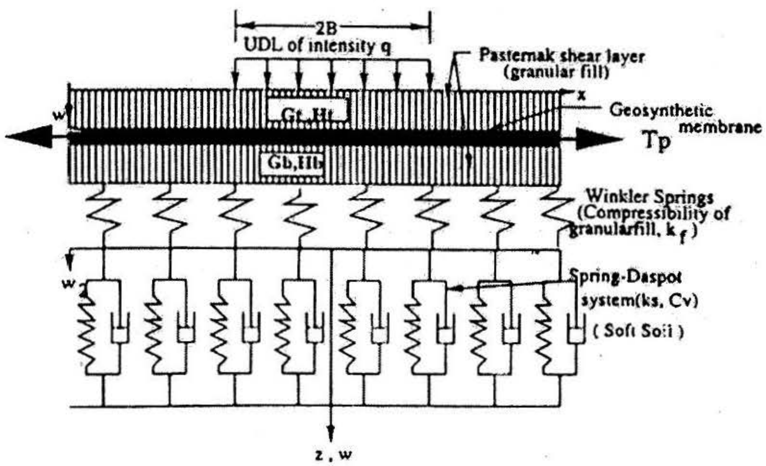

FIGURE 2 : Definition Sketch of Idealized Foundation Model for Geosynthetic Reinforced Granular Fill-Soft Soil System

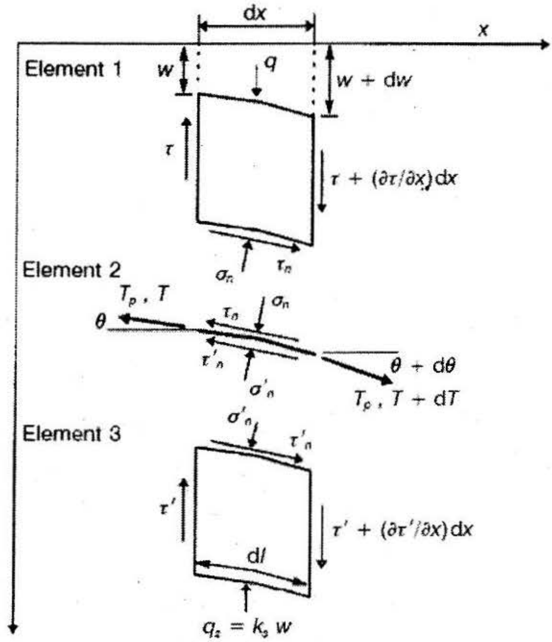

FIGURE 3 : Free Body Diagram of Three Elements from a Vertical Segment of Very Small Width (dx)

Following assumptions are made in this study:

1. Geosynthetic reinforcement is linearly elastic, rough enough to prevent slippage at the soil interface and has no shear resistance, and thickness of reinforcement is neglected.

2. Spring constant has a constant value irrespective of depth and time.

3. The strip loading is simplified as a uniform pressure loading of width 2B.

Considering the equilibrium of three elements i.e. geosynthetic element, top and bottom shear layer elements as shown in Fig.3, one can get the governing differential equation of the proposed model as,

$$\frac{d^2T}{dx^2} = \sin\theta\cos\theta\frac{d^2w}{dx^2}\frac{dT}{dx}+\frac{1}{\cos\theta}\left[\frac{G_t}{H_t}+\frac{G_b}{H_b}\right]\left[\sqrt{\left(\frac{T}{E_g}+1\right)^2-\left(\frac{dw}{dx}\right)^2}-1\right] \quad \ldots (1)$$

$$\frac{d^2w}{dx^2}=\frac{\left(\frac{\alpha}{1+\alpha}\right)\left\{k_s+u_o(1-U)\right\}-\frac{dT}{dx}\sin\theta-q}{\left[\left(T+T_p\right)\cos^3\theta+H_tG_t+H_bG_b\right]} \tag{2}$$

where
- q = footing pressure,
- dx = projected length of element in x-direction.
- θ = slope of the geosynthetic membrane,
- H_t = thickness of top granular layer,
- H_b = thickness of the lower granular layer,
- G_t = average shear modulus of top granular fill layer,
- G_b = average shear modulus of bottom granular layer,
- w = vertical displacement,
- T = mobilized tension in geosynthetic,
- U = degree of consolidation at the time t,
- E_g = tension modulus of geosynthetic reinforcement in kN/m,
- T_p = prestress in the geosynthetic layer,
- u_o = initial pore water pressure,
- k_f = spring constant per unit area for spring attached to the bottom of the Pasternak shear layer,
- α = (k_f/k_s) modular ratio or spring constant ratio.

Using the non-dimensional parameters as, $X = x/B$, $W = w/B$, $G_t^* = G_t/k_sB$, $G_b^* = G_b/k_sB$, $H_t^* = H_t/B$, $H_b^* = H_b/B$, $q^* = q/k_sB$, $T_p^* = T_p/k_sB^2$, $T^* = T^*/k_sB^2$, $\alpha = k_f/k_s$, the Eqns.1 and 2 can be written in non-dimensional form as,

$$\frac{d^2T^*}{dX^2}=\sin\theta\cos\theta\frac{d^2W}{dX^2}\frac{dT^*}{dX}+\frac{1}{\cos\theta}\left[\frac{G_t^*}{H_t^*}+\frac{G_b^*}{H_b^*}\right]\left[\sqrt{\left(\frac{T^*}{E_g^*}+1\right)^2-\left(\frac{dW}{dX}\right)^2}-1\right] \tag{3}$$

$$\frac{d^2W}{dX^2}=\frac{\left(\frac{\alpha}{1+\alpha}\right)\left\{W+u_o^*(1-U)\right\}-\frac{dT^*}{dX}\sin\theta-q^*}{\left[\left(T^*+T_p^*\right)\cos^3\theta+H_t^*G_t^*+H_b^*G_b^*\right]} \tag{4}$$

Equations 3 and 4 can be written in finite difference form for an interior node i, as,

$$T_i^* = \frac{1}{2}\left[T_{i+1}^* + T_{i-1}^* - (\Delta X)^2 \left\{ \begin{array}{l} 0.5\sin 2\theta \left(\dfrac{T_{i-1}^* - T_{i+1}^*}{\Delta X}\right)\left(\dfrac{W_{i-1} - 2W_i + W_{i+1}}{(\Delta X)^2}\right) \\ + \dfrac{1}{\cos\theta}\left[\dfrac{G_t^*}{H_t^*} + \dfrac{G_b^*}{H_b^*}\right]\left[\left\{\sqrt{\left(\dfrac{T^*}{E_g^*} + 1\right)^2 - \left(\dfrac{W_{i-1} - W_{i+1}}{2\Delta X}\right)^2}\right\} - 1\right] \end{array} \right\}\right] \quad \ldots (5)$$

$$W_i = 0.5\left[W_{i+1} + W_{i-1} - \frac{\left\{ \begin{array}{c} b_i\left(W_i + u_{o_i}^*(1-U)\right)\left(\dfrac{T_{i-1}^* - T_{i+1}^*}{2\Delta X}\right)\sin\theta \\ -\left(\dfrac{q_{i+1}^* + q_{i-1}^*}{2}\right) \end{array} \right\}}{\left(T_i^* - T_p^*\right)\cos^3\theta + G_t^* H_t^* + G_b^* H_b^*}\right] \quad (6)$$

Boundary Conditions

The solution of the Eqns.(5) and (6) is obtained for a non-dimensional uniform load intensity (q^*) acting over a width 2B. Due to symmetry of load, the slope, dW/dX will be zero at the center of loaded region. The rate of increase in mobilized tension with distance from the center, dT/dX is taken as zero, i.e.

$$\left.\begin{array}{l} dW/dX = 0 \\ dT/dX = 0 \end{array}\right. \qquad \text{for } X = 0$$

$$u_o^* = q^* \qquad \text{for } X \le 1$$

$$u_o^* = 0 \qquad \text{for } X > 1$$

At the edge of the reinforced zone the slope of settlement-distance profile is considered as zero, as observed in most of the practical cases, whether the membrane is free or fixed. The mobilized tensile force at the edge of reinforcement is considered as zero.

$$dW/dX = 0 \qquad \text{for } X = 2$$

$$T_{X=2} = 0$$

The solution has been obtained with convergence criterion

$$\left|\frac{W_i^k - W_i^{k-1}}{W_i^k}\right| \leq \varepsilon$$

for all i where k and k−1 are present and previous iterations respectively and ε is the specified tolerance which is generally taken as 0.0001.

Results and Discussions

The governing differential equations (Eqns.5 and 6) are solved using above mentioned boundary conditions using an iterative finite difference scheme. Various parametric studies are done for the following range of parameters:

Load intensity, $q^* = 0.1$ to 1.0

Shear Modulus, $G_t^*, G_b^* = 0.1$ to 1.0

Spring constant ratio, $\alpha = 5$ to infinity

Pretension, $T^* = 0.1$ to 1.0

Degree of consolidation, U = 0 to 100%

Tension modulus of geosynthetic membrane, $E_g^* = 1$ to 200

The settlement predictions obtained from the present mechanical foundation model are compared with the results obtained by Shukla and Chandra (1994c), Ghosh (1991) and Madhav and Poorooshasb (1988). Figure 4 shows the settlement profiles obtained from present model and from other existing models for different values of non-dimensionalized load intensities. The settlement obtained by the present model is less than the settlement predicted by Ghosh(1991) and Shukla and Chandra (1994c) in the regions 0.4 to 1.3 and 0.6 to 1.1 (normalized distances from center of loading) respectively. The settlement that is predicted by the present model throughout the length of the reinforced zone is more than the settlement predicted by Madhav and Poorooshasb (1988). It can be also noticed that the difference between the settlement predictions of present model and that predicted by

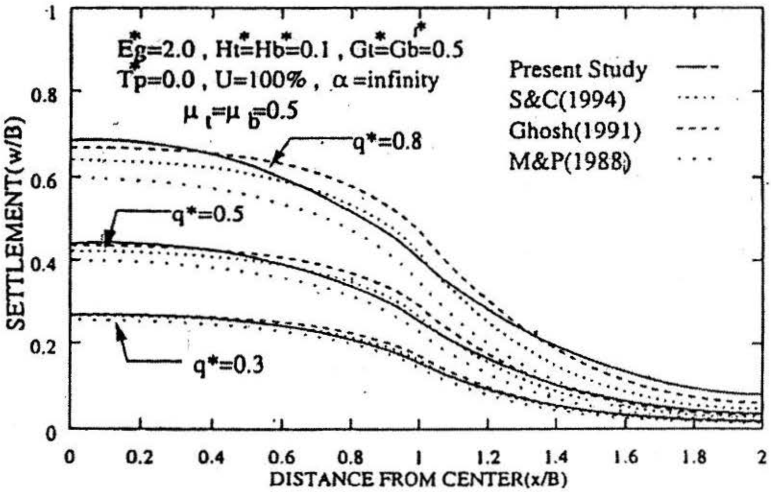

FIGURE 4 : Comparison of Present Model with those of Shukla and Chandra (1994c), Ghosh (1991), Madhav and Pooroorshasb (M&P) (1988) for Different Values of Nondimensional Load Intensities (Q^*)

other models increases at higher load intensity, but difference is very small at the lower load intensities. For example, the settlement obtained from the present model at the center of loading is more by 0.86% for $q^* = 0.3$ and by 7.06% for $q^* = 0.8$ with respect to the settlement predicted by Shukla and Chandra (1994c). The settlement at the center of loading obtained in the present study is higher by 4.42% and 14.05% for $q^* = 0.3$ and $q^* = 0.8$ respectively, as compared to the results of Madhav and Poorooshasb (1988). The corresponding values are less by 1.01% for $q^* = 0.3$ and more by 2.5% for $q^* = 0.8$ compared with the values of settlement reported by Ghosh (1991).

Figure 5 shows the typical settlement profiles at various stages of consolidation of the saturated soft soil for a particular value of modular ratio or spring constant ratio (α = 10) with no pre-tension in the geosynthetic reinforcement. It is observed that as degree of consolidation increases settlements both at the center of loaded region as well as edge of geosynthetic reinforcement increase.

Figure 6 gives the insight into the mobilized tension at various stages of consolidation of geosynthetic reinforced granular fill-soft saturated soil. At the initial stage of consolidation (20%), maximum mobilized tension occurs at the center of loading. As consolidation process proceeds, mobilized tension in the geosynthetic reinforcement decreases at the center of loading, and it increases at the edge of loaded region. At 50% consolidation condition the

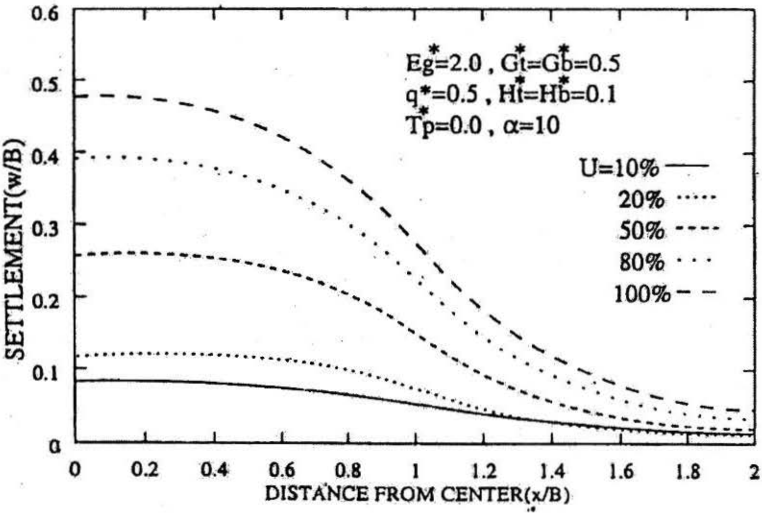

FIGURE 5 : Settlement Profiles for Various Values of Degree of Consolidation (U)

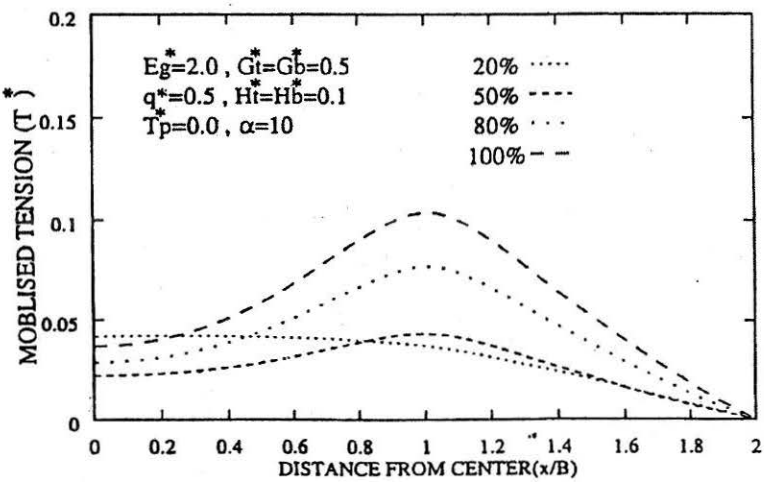

FIGURE 6 : Mobilized Tension Profiles for Various Values of Degree of Consolidation (U)

mobilized tension in geosynthetic becomes almost equal at edge of loading and center of loading and after this as consolidation proceeds further the mobilization of tension at the center of loaded region increases again. Increase in mobilized tension is 31.80% and 77.95% at the center of loading

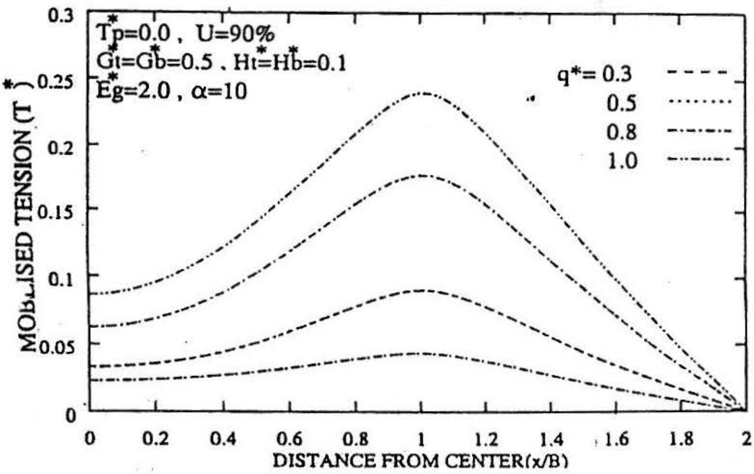

FIGURE 7 : Mobilized Tension Profiles for Various Values of Load Intensities (Q*)

and edge of loading respectively for 50% to 80% consolidation respectively and the corresponding increase in values are 27.97% and 35.06% for 80% to 100% consolidation.

Figure 7 shows typical results at 90% consolidation, for mobilized tension in the geosynthetic reinforcement for different values of load. The figure shows that as load intensity increases mobilized tension increases through out the length but the increase in mobilized tension is larger at the edge of the loading. As load increases from 0.3 to 0.5, 0.8 and 1.0, increase in mobilized tension at the center of loading are 45.45%, 181.82% and 290.91% respectively. Corresponding increase of mobilized tension at the edge of loading are 105.01%, 305.24%, and 446.7% respectively. This shows that the increase in mobilized tension is higher at higher load intensity for the same load increment at the center of loading as well as at the edge of loading. This increase is more rapid at the edge of loading compared to the center of loading.

Figure 8 shows, at 90% consolidation, the effect of variation of tension modulus of geosynthetic reinforcement on the settlement response of the geosynthetic reinforced granular fill-soft soil system considering other parameters of the model constant. As the normalized tension modulus increases the settlement in the loaded region decreases. This decrease in settlement is highest at the center of loading and zero at the edge of loading. Beyond the loaded region, settlements increase with increase in normalized tension modulus. At the edge of loading, the settlement does not undergo

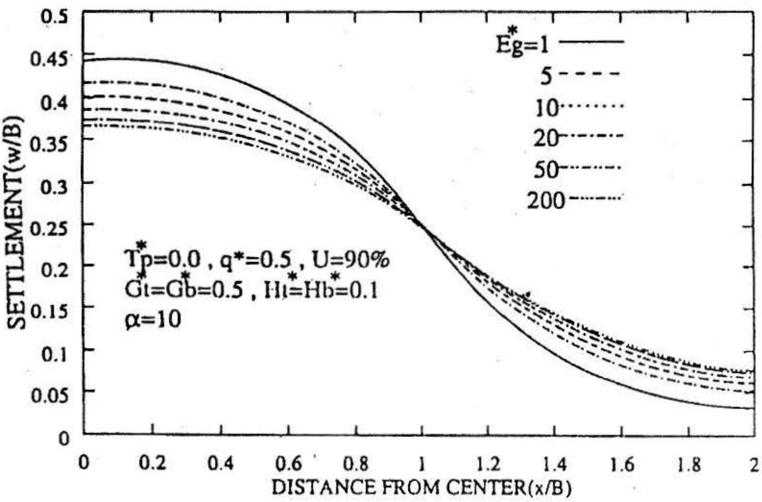

FIGURE 8 : Settlement Profiles for Various Values of Tension Modulus (E_g^*)

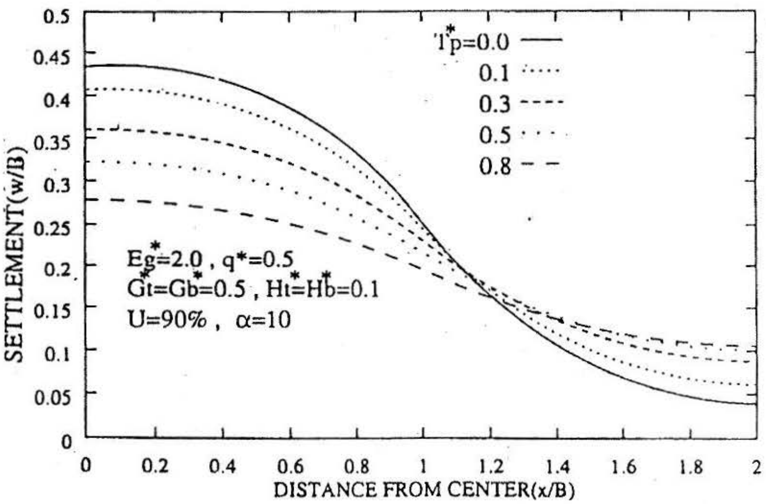

FIGURE 9 : Settlement Profiles for Various Values of Pretension in Geosynthetic Membrane (T_p^*)

any variation with change in the value of tension modulus. The improvement in the differential settlement of the system is significant up to normalized tension modulus of 50; but beyond this value, the increase of tension modulus has small effect. Decrease in settlement at the center of loaded region is

5.68% for increase of normalized tension modulus from 1 to 5 but it is only 1.86% for increase of modulus from 50 to 200.

Figure 9 shows, at 90% consolidation, the effect of pretension in the geosynthetic reinforcement on the settlement behavior of the system. As pretension in geosynthetic reinforcement is increased settlement at the center of loading decreases and settlement of the edge of reinforcement increases. This results in reduction in the differential settlement. When normalized pretension is increased from zero to 0.1, 0.3, 0.5 and 0.8, the corresponding reductions of settlement at the center of loading are 6.17%, 17.04%, 25.73% and 35.94%. For the same increase of pretension, the corresponding increase of settlement at the edge of geosynthetic reinforcement are 54.52%, 121.11%, 153.27% and 163.82% respectively. From the figure it may be seen that close to normalized distance of 1.2 from center of loading, there is almost no effect of pre-tensioning. Pretensioning of geosynthetic reinforcement may prove to be very effective ground improvement technique where very small differential settlement is desired.

Figure 10 shows, at 90% consolidation, the effect of variation of thickness of granular fill shear layers on the settlement response of the fill geosynthetic reinforced granular fill soft soil system without any pre-tension. In reducing the settlement at the center of loading it is as effective as pre-tension. When normalized thickness increases from 0.05 to 0.1, 0.2, 0.3, 0.4 and 0.5 reduction of settlements at the center of loading are 3.32%, 9.58%,

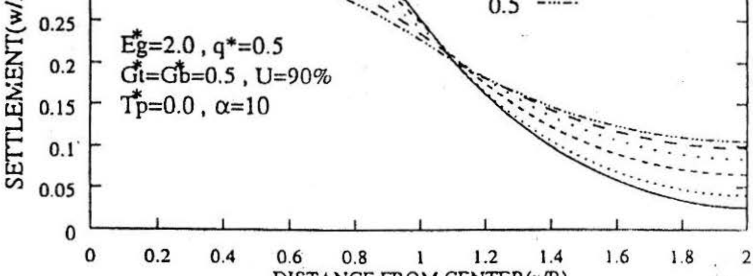

FIGURE 10 : Settlement Profiles for Various Values of Granular Fill Thickness (H_t^* or H_b^*)

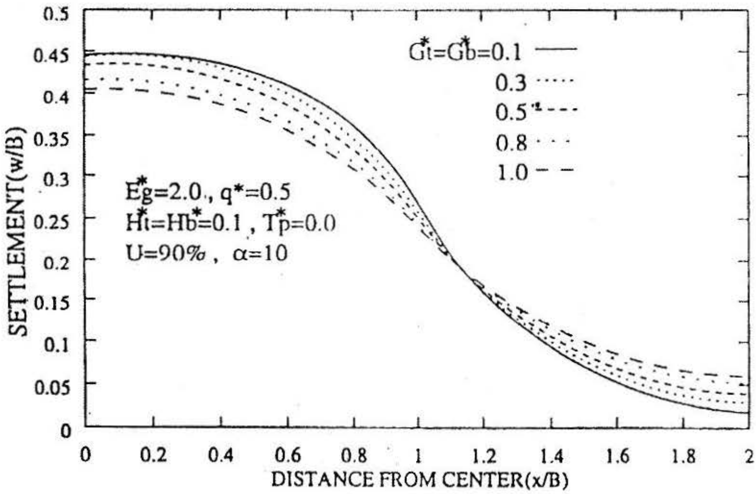

FIGURE 11 : Settlement Profiles for Various Values of Shear Modulus of Granular Fill (G_t^* or G_b^*)

15.46%, 20.66% and 25.34% respectively. Corresponding values of increase of settlements at the edge of geosynthetic reinforcement are 60%, 162%, 238%, and 320%. These show that thickness of granular fill is more effective in reducing the settlement as compared to pre-tension.

Figure 11 shows, at 90% consolidation, the settlement profile for various values of normalized shear modulus of granular fill and without pre-tension in geosynthetic reinforced granular fill soft soil system. It is observed that as the value of normalized shear modulus increases from 0.1 to 0.3, 0.5, 0.8, and 1.0, settlement at the center of loading decreases by 0.39%, 2.59%, 6.55% and 9.11% respectively. For increase of normalized shear modulus from 0.3 to 0.5, decrease in settlement is 2.2% while for increase of normalized shear modulus from 0.8 to 1.0; it is 2.56% at the center of loading. Thus, there is a slight increase in the rate of reduction of settlement for normalized shear modulus values above 0.3. The edge of reinforcement experiences increase in settlement against the trend at the center of loading. The region, between normalized distances 1.0 and 1.2 from center of loading, has almost no effect on the settlement response due to variation of normalized shear modulus.

Figure 12 shows, at 90% consolidation, typical settlement profiles of a geosynthetic reinforced granular fill soft soil system with no prestress in the geosynthetic reinforcement, bringing out the effect of the relative compressibility of the granular fill and the soft soil (spring constant ratio or

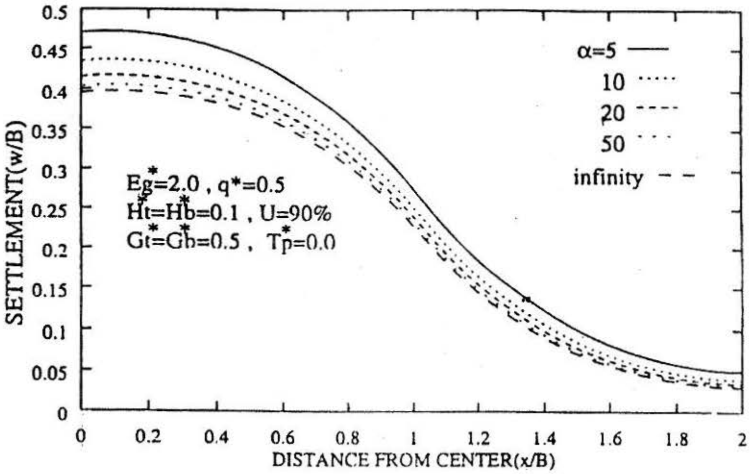

FIGURE 12 : Settlement Profiles6 for Various Values of Modular Ratio (α)

modular ratio, a) on the settlement behavior. The settlement profile plotted for spring constant ratio, $\alpha = \infty$ (infinity) corresponds to the granular fill being taken as incompressible. It is observed that the settlement at any location decreases as the spring constant ratio is increased, i.e. relative compressibility of granular fill is decreased. For example, as a increases from 5 to 50, the settlement at the center of loaded region decreases by 14.34% whereas the decrease in settlement is 1.65% for increase of a from 50 to ∞. Thus it can be observed that when the granular fill is 50 times or more stiff than the soft soil, the compressibility of the granular fill may be ignored.

Conclusions

The proposed foundation model for geosynthetic reinforced granular fill soft soil system is well suited to evaluate the settlement and mobilized tension response during any stage of the consolidation of the soft soil. The results after 100% consolidation compare reasonably well with the results of existing models. The detailed parametric studies are presented typically for 90% consolidation to observe the effects of each parameter on the settlement and mobilized tension responses of the proposed model for geosynthetic reinforced granular fill soft soil system during consolidation of the soft soil. The following conclusions are generally true for various degrees of consolidation.

1. Settlement and mobilized tension both increase with increase in the intensity of loading.

2. Maximum tension is mobilized at the edge of loading for the constant value considered for other parameters.

3. Tension modulus of geosynthetic has significant effect on the reduction of total and differential settlements as long as its value is less than 50.

4. The settlement is highly influenced by the consolidation, and pattern of mobilized tension in geosynthetic reinforcement is a function of degree of consolidation.

5. Pretension in the geosynthetic reinforcement and the thickness of granular fill are found to be very effective in reducing the total and differential settlements.

6. Increase in shear modulus of granular fill also causes reduction in total and differential settlement.

7. Compressibility of the granular fill has an appreciable influence on the settlement as long as its stiffness is less than approximately 50 times that of the soft soil.

Notations

B = Half of the width of loaded region (m)

dx = Projected element length in x direction (m)

E_g = Tension modulus of geosynthetic layer (kN/m)

E_g^* = Non-dimensional tension modulus of geosynthetic layer

G_b = Shear modulus of granular fill below the reinforcement (kN/m^2)

G_b^* = Non-dimensional shear parameter of granular fill below the reinforcement

G_t = Shear modulus of granular fill above the reinforcement (kN/m^2)

G_t^* = Non-dimensional shear parameter of granular fill above the reinforcement

H_b = Thickness of the granular fill below the reinforcement (m)

H_b^* = Non-dimensional thickness of the granular fill below the reinforcement

H_t	=	Thickness of the granular fill above the reinforcement (m)
H_t^*	=	Non-dimensional thickness of the granular fill above the reinforcement
i	=	Subscript referring to a nodal point
k_f	=	Modulus of subgrade reaction for granular fill (kN/m^3)
k_s	=	Modulus of subgrade reaction for soft soil (kN/m^3)
q	=	Footing pressure (kN/m^2)
q^*	=	Non-dimensional Footing pressure
T	=	Mobilized tensile force in the reinforcement (kN/m)
T^*	=	Non-dimensional mobilized tensile force in the reinforcement
T_p	=	Prestress in the geosynthetic reinforcement (kN/m)
T_p^*	=	Non-dimensional prestress in the geosynthetic reinforcement
t	=	Time (sec)
U	=	Degree of consolidation
u_o	=	Initial pore water pressure (kN/m^2)
u_o^*	=	Non-dimensional initial pore water pressure
w	=	Vertical displacement (m)
W	=	Non-dimensional vertical displacement
x	=	Horizontal space coordinate (m)
θ	=	Slope of the membrane (degrees)
α	=	Spring constant ratio (k_f/k_s)

References

GHOSH, C. (1991) : "Modelling and Analysis of Reinforced Foundation Beds", *Ph.D. Thesis*, Department of Civil Engineering, Indian Institute of Technology, Kanpur.

MADHAV, M.R. and POOROOSHASB, H.B. (1988) : "A New Model for Geosynthetic-Reinforced Soil", *Computers and Geotechnics*, 6, pp.277-290.

MADHAV, M.R. and POOROOSHASB, H.B. (1989) : "Modified Pasternak Model for Reinforced Soil", *Int. J. of Mathematical Modelling*, Vol.12, No.12, pp.1505-1509.

PINTO, M.I.M. (2003) : "Application of Geosynthetics for Soil Reinforcement", *Ground Improvement*, 7, pp.61-72.

SHUKLA, S.K. and CHANDRA, S. (1994a) : "The Effect of Prestressing on the Settlement Characteristics of Geosynthetic-Reinforced Soil", *Geotextiles and Geomembranes*, 13, pp.531-543.

SHUKLA, S.K. and CHANDRA, S. (1994b) : "A Study of Settlement Response of a Geosynthetic-Reinforced Compressible Granular Fill-Soft Soil System", *Geotextiles and Geomembranes*, 13, pp.627-639.

SHUKLA, S.K. and CHANDRA, S. (1994c) : "A Generalized Mechanical Model for Geosynthetic-Reinforced Foundation Soil", *Geotextiles and Geomembranes*, 13, pp.813-825.

YIN, J.H. (1997) : "Modelling of Geosynthetic-Reinforced Granular Fill over Soft Soil", *Geosynthetics International*, Vol.4, No.2, pp.165-185.