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Soil-structure interaction analysis of beams resting on multilayered geosynthetic-reinforced soil

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Abstract. In this paper, soil-structure interaction analysis has been presented for beams resting on multilayered geosynthetic-reinforced granular fill-soft soil system. The soft soil and geosynthetic reinforcements are idealized as nonlinear springs and elastic membranes, respectively. The governing differential equations are solved by finite difference technique and the results are presented in non-dimensional form. It is observed from the study that use of geosynthetic reinforcement is not very effective for maximum settlement reduction in case of very rigid beam. Similarly the reinforcements are not effective for shear force reduction if the granular fill has very high shear modulus value. However, multilayered reinforced system is very effective for bending moment and differential settlement reduction.

Keywords: beams; flexural rigidity; multilayered geosynthetic reinforcement; soft soil; soil-structure interaction.

1. Introduction

Application of geosynthetic reinforcement effectively reduces the settlement and increases the bearing capacity of the poor soil. In many practical situations in the field, granular fills containing multiple layers of geosynthetic reinforcement are often placed over the soft soil to improve the settlement and bearing capacity. In recent years, based on lumped parameter modeling approach many researchers have studied the load-settlement behavior of such reinforced foundation beds (Madhav and Poorooshasb 1988, Ghosh and Madhav 1994, Shukla and Chandra 1994, Yin 1997). Studies have been conducted on soil-structure or soil-foundation interaction and failure load predictions of geosynthetic reinforced soil structures (Jahromi *et al*. 2007, Alsaleh *et al*. 2009, Swamy *et al*. 2011).

Most of the models reported in the literature are developed for single layer reinforced systems. Nagami and Yong (2003) studied the response of a multi layer geosynthetic reinforced soil bed subjected to structural loading. Deb *et al*. (2005) developed a mechanical model for inextensible multilayered reinforced granular fill resting over soft soil. A nonlinear study on extensible multilayered reinforced soil has also been conducted by Deb *et al*. (2007). However, in the developed models for multilayered reinforced system, beams are not considered. In the available models, only the settlement behavior has been studied. In the design of foundation, not only

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settlement is a significant design factor, but bending moment and shear force are also very important design factors. The flexural rigidity of foundation also plays a major role on its behavior. Thus, it is necessary to determine the settlement as well as bending moment and shear force of the foundation. Idealizing foundation as beam, the effect of geosynthetic reinforcements on the settlement, bending moment and shear force can be studied. Maheshwari *et al*. (2004) studied the behavior of beams resting on geogrid-reinforced sand. Studies have also been conducted on beams resting on geosynthetic-reinforced or unreinforced soil subjected to moving load (Mallik *et al*. 2006, Maheshwari and Viladkar 2010). However, only single layer of reinforcement was considered for reinforced cases. Thus, it is necessary to study the behavior of beams resting on multilayered reinforced soil. In this paper, based on soil-structure interaction analysis a mechanical model has been developed to study the effect of flexural rigidity of beams resting on soil reinforced with multiple layers of reinforcement. The effect of properties of soft soil and granular fill on the behavior of beam is also studied.

2. Model development

A multi layer geosynthetic-reinforced granular fill on soft foundation soil is shown in Fig. 1. In this model, the granular fill and the soft soil have been idealized by the Pasternak shear layer and a layer of nonlinear springs, respectively. Stretched rough elastic membranes represent the geosynthetic reinforcement layers. Three geosynthetic layers are considered in the model and they divide the shear layer into four equal parts. The footing is idealized as a beam. Plane strain condition is considered for the loading and the reinforced foundation soil system. A footing load of intensity q is applied over a width of $2B$ on the multi layer geosynthetic-reinforced granular fill of width $2L$ over soft soil as shown in Fig. 1.

The differential equation of bending of a beam is written as

$$EI\frac{d^4w}{dx^4} = q - p \tag{1}$$

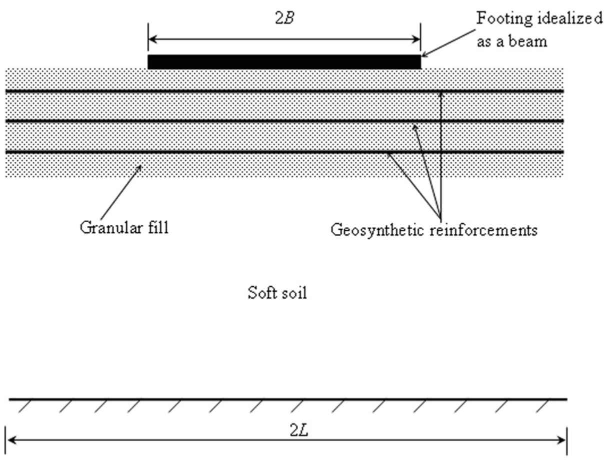

Fig. 1 Multi layer geosynthetic-reinforced granular fill-soft soil system with beam

where EI is the flexural rigidity of the beam, q is surface loads (uniformly distributed load) and p is the foundation bearing pressure. The upward-acting shearing force to the left is considered as positive and the corresponding clockwise bending moment acting from the left is considered as positive bending moment.

According to Deb *et al*. (2005, 2008), the normal stresses and the mobilized tension for elements of the different geosynthetic reinforcement layers are obtained as follows

$$p = \bar{C}_1\bar{C}_3\bar{C}_5 q_s - \left\{ \begin{array}{l} G_1H_1 + \bar{C}_1 G_2H_2 + \bar{C}_1\bar{C}_3 G_3H_3 + \bar{C}_1\bar{C}_3\bar{C}_5 G_4H_4 \\ + \bar{C}_2 T_1\cos\theta + \bar{C}_1\bar{C}_4 T_2\cos\theta + \bar{C}_1\bar{C}_3\bar{C}_6 T_3\cos\theta \end{array} \right\} \frac{d^2w}{dx^2} \tag{2}$$

$$\frac{dT_1}{dx} = -\bar{D}_1\left(p + G_1H_1\frac{d^2w}{dx^2}\right) - \bar{D}_2\left[\begin{array}{c} \bar{C}_3\bar{C}_5 q_s - G_2H_2 + \bar{C}_3 G_3H_3 + \bar{C}_3\bar{C}_5 G_4H_4 \\ + (\bar{C}_4 T_2\cos\theta + \bar{C}_3\bar{C}_6 T_3\cos\theta)\dfrac{d^2w}{dx^2} \end{array} \right] \tag{3}$$

$$\frac{dT_2}{dx} = -\bar{D}_3\left[\frac{1}{\bar{C}_1}\left\{p + (G_1H_1 + \bar{C}_2 T_1\cos\theta)\frac{d^2w}{dx^2}\right\} + G_2H_2\frac{d^2w}{dx^2}\right]$$
$$-\bar{D}_4\left[(\bar{C}_5 q_s - G_3H_3 + \bar{C}_5 G_4H_4 + \bar{C}_6 T_3\cos\theta)\frac{d^2w}{dx^2}\right] \tag{4}$$

and

$$\frac{dT_3}{dx} = -\bar{D}_5\left[\frac{1}{\bar{C}_3}\left\{ \begin{array}{c} \dfrac{1}{\bar{C}_1}\left\{p + (G_1H_1 + \bar{C}_2 T_1\cos\theta)\dfrac{d^2w}{dx^2}\right\} \\ + (G_2H_2 + \bar{C}_4 T_2\cos\theta)\dfrac{d^2w}{dx^2} \end{array} \right\} + G_3H_3\frac{d^2w}{dx^2}\right] - \bar{D}_6\left(q_s - G_4H_4\frac{d^2w}{dx^2}\right) \tag{5}$$

where

$$\bar{C}_1 = \frac{1 + K_0\tan^2\theta - (1 - K_0)\mu_2\tan\theta}{1 + K_0\tan^2\theta + (1 - K_0)\mu_1\tan\theta}$$

$$\bar{C}_2 = \frac{1}{1 + K_0\tan^2\theta + (1 - K_0)\mu_1\tan\theta}$$

$$\bar{C}_3 = \frac{1 + K_0\tan^2\theta - (1 - K_0)\mu_3\tan\theta}{1 + K_0\tan^2\theta + (1 - K_0)\mu_2\tan\theta}$$

$$\bar{C}_4 = \frac{1}{1 + K_0\tan^2\theta + (1 - K_0)\mu_2\tan\theta}$$

$$\bar{C}_5 = \frac{1 + K_0\tan^2\theta - (1 - K_0)\mu_4\tan\theta}{1 + K_0\tan^2\theta + (1 - K_0)\mu_3\tan\theta}$$

$$\bar{C}_6 = \frac{1}{1 + K_0 \tan^2\theta + (1 - K_0)\mu_3 \tan\theta}$$

$$\bar{D}_1 = \mu_1 \cos\theta(1 + K_0 \tan^2\theta) - (1 - K_0)\sin\theta$$

$$\bar{D}_2 = \mu_2 \cos\theta(1 + K_0 \tan^2\theta) + (1 - K_0)\sin\theta$$

$$\bar{D}_3 = \mu_2 \cos\theta(1 + K_0 \tan^2\theta) - (1 - K_0)\sin\theta$$

$$\bar{D}_4 = \mu_3 \cos\theta(1 + K_0 \tan^2\theta) + (1 - K_0)\sin\theta$$

$$\bar{D}_5 = \mu_3 \cos\theta(1 + K_0 \tan^2\theta) - (1 - K_0)\sin\theta$$

$$\bar{D}_6 = \mu_4 \cos\theta(1 + K_0 \tan^2\theta) + (1 - K_0)\sin\theta$$

where H_1, H_2, H_3 and H_4 are the thickness of the granular layer between the geosynthetic reinforcements (stating from top); q_s is the average normal stress acting on the soft soil; w is the vertical displacement; T_1, T_2 and T_3 are mobilized tension in the top, middle and bottom geosynthetic layer, respectively; θ is slope of the membranes; μ_1 and μ_2 are interface friction at the top and bottom of the top geosynthetic layer, respectively; μ_2 and μ_3 are interface friction at the top and bottom of the middle geosynthetic layer, respectively; μ_3 and μ_4 are interface friction at the top and bottom of the bottom geosynthetic layer, respectively; K_0 is the coefficient of lateral earth pressure for the normally consolidated soil at rest is assumed to be equal to $1 - \sin\phi$ and $\tan\theta = -dw/dx$. The expression of q_s is given as (Kondner 1963)

$$q_s = \frac{k_{s0} w}{1 + k_{s0}(w/q_{us})} \tag{6}$$

where k_{s0} is the initial modulus of the subgrade reaction of the soft soil (spring constant per unit area for the spring) and q_{us} is the ultimate bearing capacity of the soft soil. The expression of shear layer for the different shear layer is expressed as (Ghosh and Madhav 1994)

$$G_j = \frac{G_{j0}}{\left[1 + \dfrac{G_{j0}|dw/dx|}{\tau_{uj}}\right]^2}, \quad j = 1, 2, 3, 4 \tag{7}$$

where G_{j0} is initial shear modulus of the shear layer between the geosynthetic reinforcements (stating from top); τ_{uj} is ultimate shear resistance of the shear layer between the geosynthetic reinforcements (stating from top); dw/dx is the shear strain.

Using the non-dimensional parameters as: $X = x / B$; $W_b = w_b / B$; $W_f = w_f / B$; $I^* = EI / k_{s0}B^4$; $G_j^* = G_jH_j / k_{s0}B^2$; $G_{j0}^* = G_{j0}H_j / k_{s0}B^2$; $T_j^* = T_j / k_{s0}B^2$; $q^* = q / k_{s0}B$; $q_{us}^* = q_{us} / k_{s0}B$; $\tau_{uj}^* = \tau_{uj}H_j / k_{s0}B^2$; $q_s^* = q_s / k_{s0}B$; the governing differential equations can be written in non-dimensional form as (within the beam region, i.e., $X \leq 1$)

$$q^* = I^* \frac{d^4 W_b}{dX^4} + \bar{C}_1\bar{C}_3\bar{C}_5 \frac{W_b}{1 + (W_b/q_{us}^*)} - \left\{ \begin{aligned} & G_1^* + \bar{C}_1 G_2^* + \bar{C}_1\bar{C}_3 G_3^* + \bar{C}_1\bar{C}_3\bar{C}_5 G_4^* \\ & + \bar{C}_2 T_1^* \cos\theta + \bar{C}_1\bar{C}_4 T_2^* \cos\theta + \bar{C}_1\bar{C}_3\bar{C}_6 T_3^* \cos\theta \end{aligned} \right\} \frac{d^2 W_b}{dX^2} \tag{8}$$

$$\frac{dT_1^*}{dX} = -\bar{D}_1\left(q^* - I^*\frac{d^4W_b}{dX^4} + G_1^*\frac{d^2W_b}{dX^2}\right) - \bar{D}_2\left[\begin{array}{c}\bar{C}_3\bar{C}_5\dfrac{W_b}{1+(W_b/q_{us}^*)} - G_2^* + \bar{C}_3G_3^* + \bar{C}_3\bar{C}_5G_4^* \\ +(\bar{C}_4T_2^*\cos\theta + \bar{C}_3\bar{C}_6T_3^*\cos\theta)\dfrac{d^2W_b}{dX^2}\end{array}\right] \tag{9}$$

$$\frac{dT_2^*}{dX} = -\bar{D}_3\left[\frac{1}{\bar{C}_1}\left\{q^* - I^*\frac{d^4W_b}{dX^4} + (G_1^* + \bar{C}_2T_1^*\cos\theta)\frac{d^2W_b}{dX^2}\right\} + G_2^*\frac{d^2W_b}{dX^2}\right]$$
$$-\bar{D}_4\left[\left\{\bar{C}_5\frac{W_b}{1+(W_b/q_{us}^*)} - G_3^* + \bar{C}_5G_4^* + \bar{C}_6T_3^*\cos\theta\right\}\frac{d^2W_b}{dX^2}\right] \tag{10}$$

and

$$\frac{dT_3^*}{dX} = -\bar{D}_5\left[\frac{1}{\bar{C}_3}\left\{\begin{array}{c}\dfrac{1}{\bar{C}_1}\left\{q^* - I^*\dfrac{d^4W_b}{dX^4} + (G_1^* + \bar{C}_2T_1^*\cos\theta)\dfrac{d^2W_b}{dX^2}\right\} \\ +(G_2^* + \bar{C}_4T_2^*\cos\theta)\dfrac{d^2W_b}{dX^2}\end{array}\right\} + G_3^*\frac{d^2W_b}{dX^2}\right]$$
$$-\bar{D}_6\left(\frac{W_b}{1+(W_b/q_{us}^*)} - G_4^*\frac{d^2W_b}{dX^2}\right) \tag{11}$$

The governing differential equations beyond the beam region (i.e., $X > 1$) can be written in non-dimensional form as

$$\bar{C}_1\bar{C}_3\bar{C}_5\frac{W_f}{1+(W_f/q_{us}^*)} - \left\{\begin{array}{c}G_1^* + \bar{C}_1G_2^* + \bar{C}_1\bar{C}_3G_3^* + \bar{C}_1\bar{C}_3\bar{C}_5G_4^* \\ +\bar{C}_2T_1^*\cos\theta + \bar{C}_1\bar{C}_4T_2^*\cos\theta + \bar{C}_1\bar{C}_3\bar{C}_6T_3^*\cos\theta\end{array}\right\}\frac{d^2W_f}{dX^2} = 0 \tag{12}$$

$$\frac{dT_1^*}{dX} = -\bar{D}_1G_1^*\frac{d^2W_f}{dX^2} - \bar{D}_2\left[\begin{array}{c}\bar{C}_3\bar{C}_5\dfrac{W_f}{1+(W_f/q_{us}^*)} - G_2^* + \bar{C}_3G_3^* + \bar{C}_3\bar{C}_5G_4^* \\ +(\bar{C}_4T_2^*\cos\theta + \bar{C}_3\bar{C}_6T_3^*\cos\theta)\dfrac{d^2W_f}{dX^2}\end{array}\right] \tag{13}$$

$$\frac{dT_2^*}{dX} = -\bar{D}_3\left[\frac{1}{\bar{C}_1}\left\{(G_1^* + \bar{C}_2T_1^*\cos\theta)\frac{d^2W_f}{dX^2}\right\} + G_2^*\frac{d^2W_f}{dX^2}\right]$$
$$-\bar{D}_4\left[\left\{\bar{C}_5\frac{W_f}{1+(W_f/q_{us}^*)} - G_3^* + \bar{C}_5G_4^* + \bar{C}_6T_3^*\cos\theta\right\}\frac{d^2W_f}{dX^2}\right] \tag{14}$$

and

$$\frac{dT_3^*}{dX} = -\bar{D}_5\left[\frac{1}{\bar{C}_3}\left\{\begin{array}{l}\frac{1}{\bar{C}_1}\left\{(G_1^* + \bar{C}_2 T_1^* \cos\theta)\frac{d^2 W_f}{dX^2}\right\} \\ + (G_2^* + \bar{C}_4 T_2^* \cos\theta)\frac{d^2 W_f}{dX^2}\end{array}\right\} + G_3^* \frac{d^2 W_f}{dX^2}\right]$$

$$-\bar{D}_6\left(\frac{W_f}{1 + (W_f/q_{us}^*)} - G_4^* \frac{d^2 W_f}{dX^2}\right) \tag{15}$$

The W_b is the vertical displacement of the beam and W_f is the vertical displacement of the granular layer beyond the beam in non-dimensional form. In the governing equation beyond the beam, I^* is taken as zero. The bending moment and shear force in the beam can be written in non-dimensional form as

$$M^* = -I^* \frac{d^2 W_b}{dX^2} \quad \text{and} \quad Q^* = -I^* \frac{d^3 W_b}{dX^3} \tag{16}$$

2.1 Method of solution and boundary conditions

Finite difference method has been employed to solve the governing differential equations. In these equations, the derivative d^4W / dX^4 and d^2W / dX^2 have been expressed by central difference scheme while dT_j^* / dX have been expressed by forward difference scheme. The length L/B may be divided into n number of the same increment length with $(n+1)$ number of node points ($i = 1, 2, 3, 4\ldots\ldots\ldots, n$). As the problem is symmetric, only half of the system is considered for the analysis. The boundary conditions chosen as: at $X = 0$, due to symmetry, the slope, $dW_b/dX = 0$ and $Q^* = 0$. At $X = 1$, $M^* = 0$; $Q^* = G^* (dW_f / dX - dW_b / dX)$ and $W_b = W_f$. The distance $X = L/B$ is chosen in such a way that at $X = L/B$, $W_f = 0$. As geosynthetic layers are free at the end, the mobilized tension, $T_1^* = T_2^* = T_3^* = 0$ at $X = L/B$ (or $x = L$). The loading conditions are as: $q_i^*(X) = q^*$ for $|X| \leq 1.0$ and $q_i^*(X) = 0$ for $|X| > 1.0$.

3. Results and discussions

A computer program based on the formulation as described above has been developed and solutions are obtained using an iterative technique with a tolerance value of 10^{-4}. To validate the present model, the results are compared with the results reported by Yin (1997) based on finite element analysis and Deb *et al*. (2005) based on lumped parameter modeling approach for single layered reinforced system (as shown in Fig. 2). In the finite element analysis (Yin 1997), the behaviour of the soft soil and granular fill was assumed to be linear. However, in the present analysis, the behaviour of the soft soil and granular fill is taken as nonlinear. Thus, to compare the results of present analysis with the finite element analysis, the non dimensional ultimate bearing capacity of the soft soil and the ultimate shearing resistance of the granular fill layer are considered to be equal to 10 as beyond this value the behaviour of the soft soil and granular fill layer becomes linear (Deb *et al*. 2005). The lumped parameter model (Deb *et al*. 2005) was developed for

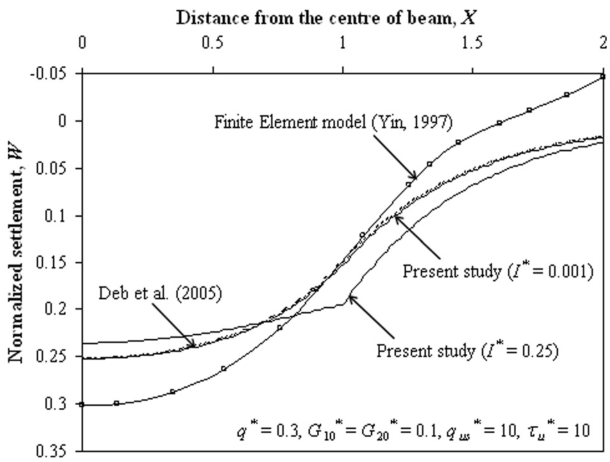

Fig. 2 Comparison of the results of the present model with available results

multilayered reinforced system without considering the beam. During the comparison, the geosynthetic reinforcement was placed in the middle of the granular fill. It has been observed that as the flexural rigidity of the beam decreases, the resulting curve of the present model converges towards the resulting curve presented in Deb *et al*. (2005). This is due to the fact that for very low flexural rigidity of the beam, the present model is identical with the model presented in Deb *et al*. (2005). The difference between the resulting curve for low flexural rigidity value of the beam and the finite element analysis results is due to the fact that finite element program used by Yin (1997) is based on the theory of small deformation, which normally overestimates the settlement for large deformation problems such as the present study and the model reported by Deb *et al*. (2005). However, beyond the loaded region, the present model overestimates the settlement values as compared to the results of the finite element analysis. This is due to the fact that the present model is not capable to predict the heaving beyond the loaded region as the distance $X = L/B$ is chosen in such a way that at $X = L/B$, $W_f = 0$.

In the parametric study, the typical values used are the angle of shearing resistance for the granular fill, $\phi = 36^{\circ}$; the coefficient of lateral stress, $K_0 = 0.41$; the interface friction coefficients equal to 0.5. Fig. 3 shows the effect of flexural rigidity of beams on settlement response of multilayered geosynthetic reinforced system. It has been observed that for $I^* = 0.05$, the reduction of settlement at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 12.2%, 17% and 19.4%, respectively. However, for $I^* = 0.25$ the reduction is only 3.5%, 4.6% and 5%, respectively. It is also observed that for unreinforced case the settlement at the centre of the beam is decreased by 10% as I^* increases from 0.05 to 0.25. Thus, it can be said that as the flexural rigidity of beam increases the effectiveness of the reinforcements to reduce the maximum settlement decreases. It is further observed that multilayered reinforced system is not so effective for maximum settlement reduction of beams with higher flexural rigidity. However, single layered or two layered reinforced system is effective for maximum settlement reduction of beam with lower flexural rigidity value, but for higher flexural rigidity of the beam use of any reinforcement is not so effective for maximum settlement reduction.

The differential settlement (settlement difference between the centre and edge of the foundation or beam) is reduced by 50% as I^* increases from 0.05 to 0.25. For $I^* = 0.05$, the reduction of

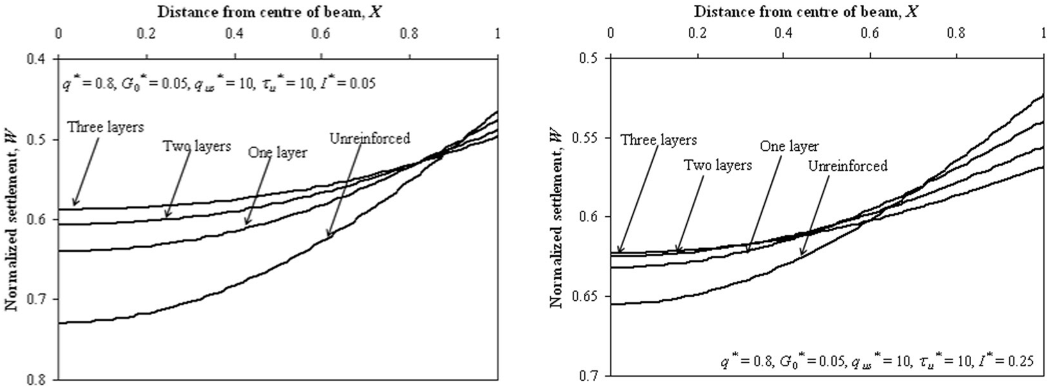

Fig. 3 Effect of flexural rigidity of beams on settlement: (a) $I^* = 0.05$ and (b) $I^* = 0.25$

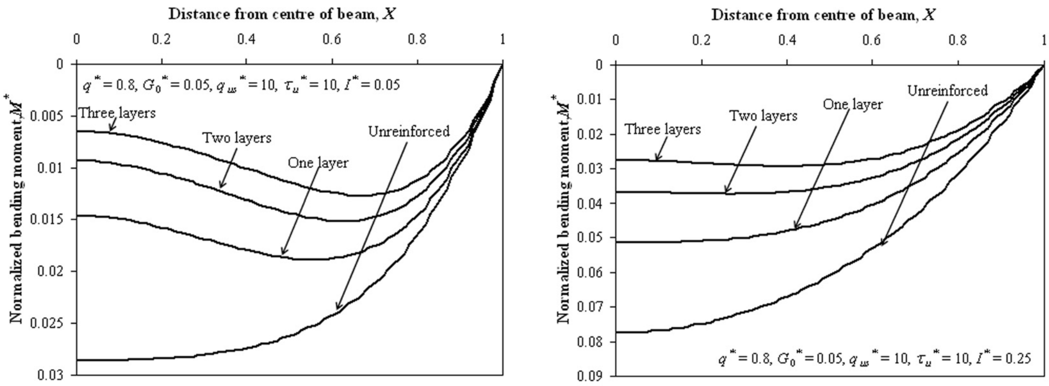

Fig. 4 Effect of flexural rigidity of beams on bending moment: (a) $I^* = 0.05$ and (b) $I^* = 0.25$

differential settlement of the beam from unreinforced case to single, double and three layered reinforced system is 38%, 55.4% and 66%, respectively. However, for $I^* = 0.25$ the reduction is 30%, 47.3% and 58.6%, respectively. Thus, multilayered reinforced system is effective for reduction in differential settlement of the beam or foundation, though the reduction rate decreases as the number of reinforcement layer increases.

Fig. 4 shows the effect of flexural rigidity of beams on bending moment for multilayered reinforced system. It has been observed that for $I^* = 0.05$, the reduction of bending moment at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 49%, 67.4% and 77.3%, respectively. However, for $I^* = 0.25$ the reduction is 33.7%, 52.5% and 64.5%, respectively. It is also observed that for unreinforced case, the bending moment at the centre of the beam is increased by 171% as I^* increases from 0.05 to 0.25. Thus, it can be said that the multilayered reinforced system is effective for bending moment reduction of the beam. However, the effectiveness is more in case of lower flexural rigidity of the beam. It is further observed that for unreinforced case the maximum bending moment is occurred at the centre of the beam irrespective to the flexural rigidity of the beam. For $I^* = 0.05$, the maximum bending moment occurred at $X = 0.55$, 0.65 and 0.67 for single, two and three layered reinforced system, respectively. However, for $I^* = 0.05$, the maximum bending moment occurred at $X = 0$, 0.27 and 0.40,

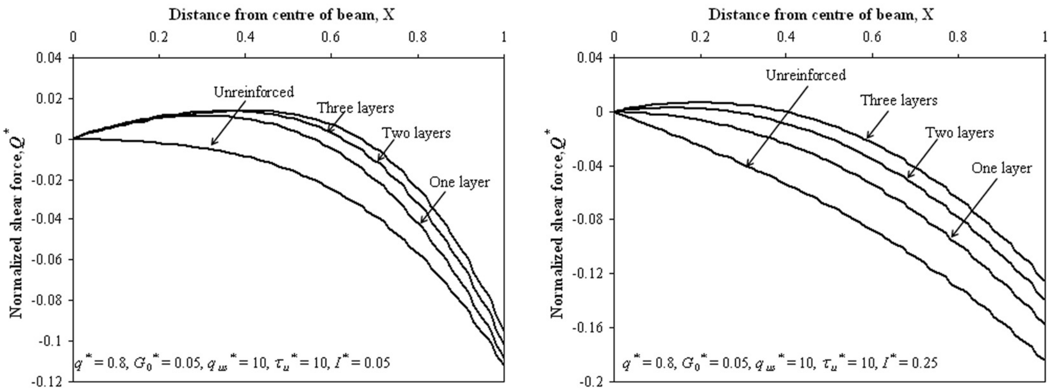

Fig. 5 Effect of flexural rigidity of beams on shear force: (a) $I^* = 0.05$ and (b) $I^* = 0.25$

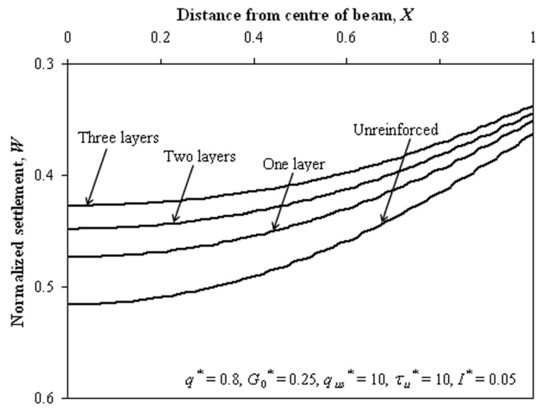

Fig. 6 Effect of shear modulus on settlement response

respectively. Thus, for geosynthetic-reinforced case, the maximum bending moment is occurred at $X > 0.5$ location of the beam for lower flexural rigidity, whereas for higher flexural rigidity of the beam it is occurred at $X < 0.5$ location. As the number of reinforcement layer increases the position of maximum bending moment is shifted away from the centre of the beam.

Fig. 5 shows the effect of flexural rigidity of beams on shear force for multilayered reinforced system. It has been observed that for $I^* = 0.05$, the reduction of shear force at the edge of the beam from unreinforced case to single, double and three layered reinforced system is 3.4%, 9.5% and 15.3%, respectively. However, for $I^* = 0.25$ the reduction is 14.5%, 24.5% and 32%, respectively. It is also observed that for unreinforced case the shear force at the edge of the beam is increased by 64.5% as I^* increases from 0.05 to 0.25. Thus, it can be said that the multilayered reinforced system is also effective for shear force reduction of the beam. The effectiveness in shear force reduction increases as the flexural rigidity of beam increases.

It has been observed that for $G^* = 0.05$, the reduction of settlement at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 12.2%, 17% and 19.4%, respectively (as shown in Fig. 3). However, for $G^* = 0.25$ the reduction is 8.3%, 13.3% and 17.2%, respectively (as shown in Fig. 6). It is also observed that for unreinforced case the settlement at the

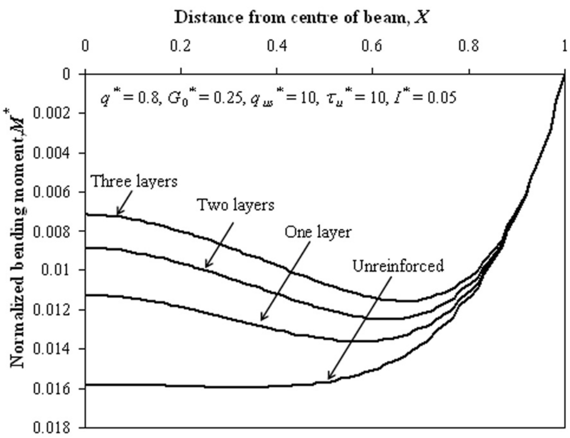

Fig. 7 Effect of shear modulus on bending moment

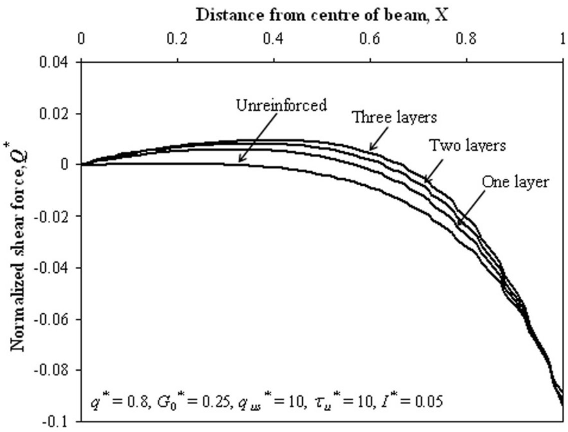

Fig. 8 Effect of shear modulus on shear force

centre of the beam is decreased by 29% as G^* increases from 0.05 to 0.25. Thus, it can be said that as the shear modulus of the granular fill increases the effectiveness of the reinforcements to reduce the maximum settlement decreases.

It has been observed that for $G^* = 0.05$, the reduction of bending moment at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 49%, 67.4% and 77.3%, respectively (as shown in Fig. 4). However, for $G^* = 0.25$ the reduction is 29%, 44% and 55%, respectively (as shown in Fig. 7). It is also observed that for unreinforced case, the bending moment at the centre of the beam is decreased by 45% as G^* increases from 0.05 to 0.25. Thus, it can be said that the effectiveness of multilayered reinforced system for bending moment reduction of the beam is more in case of lower shear modulus of granular layer.

It has been observed that for $G^* = 0.05$, the reduction of shear force at the edge of the beam from unreinforced case to single, double and three layered reinforced system is 3.4%, 9.5% and 15.3%, respectively (as shown in Fig. 5). However, for $G^* = 0.25$ the reduction is 3.3%, 6.3% and 6.1%, respectively (as shown in Fig. 8). It is also observed that for unreinforced case the shear force at the edge of the beam is decreased by 21% as G^* increases from 0.05 to 0.25. Thus, it can be said that

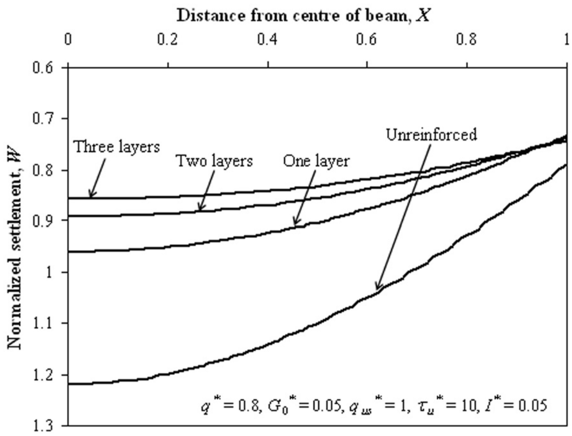

Fig. 9 Effect of ultimate bearing capacity of soft soil on settlement response

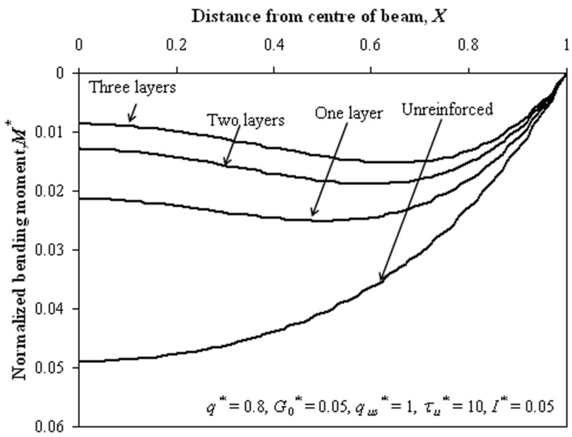

Fig. 10 Effect of ultimate bearing capacity of soft soil on bending moment

the multilayered reinforced system is more effective for shear force reduction of the beam in case of lower shear modulus of the granular layer. For granular layer with higher shear modulus value, use of reinforcement is not so effective to reduce maximum shear force of the beam.

It has been observed that for $q_{us}^* = 10$, the reduction of settlement at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 12.2%, 17% and 19.4%, respectively (as shown in Fig. 3). However, for $q_{us}^* = 1$, the reduction is 21%, 27% and 29.6%, respectively (as shown in Fig. 9). It is also observed that for unreinforced case the settlement at the centre of the beam is increased by 67% as q_{us}^* decreases from 10 to 1. Thus, it can be said that as the ultimate bearing capacity of soft soil decreases the effectiveness of the reinforcements to reduce the maximum settlement increases.

It has been observed that for $q_{us}^* = 10$, the reduction of bending moment at the centre of the beam from unreinforced case to single, double and three layered reinforced system is 49%, 67.4% and 77.3%, respectively (as shown in Fig. 4). However, for $q_{us}^* = 1$ the reduction is 57.3%, 74% and 82.3%, respectively (as shown in Fig. 10). It is also observed that for unreinforced case, the bending moment at the centre of the beam is increased by 71% as q_{us}^* decreases from 10 to 1. Thus, the

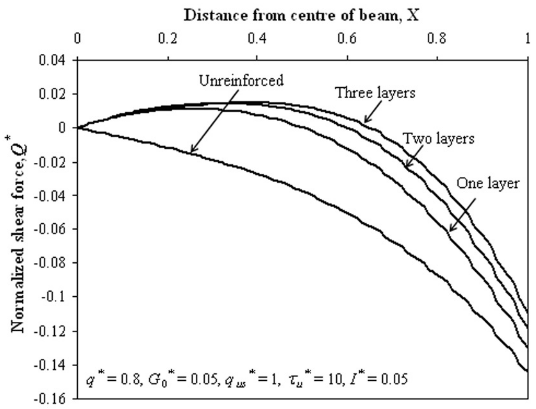

Fig. 11 Effect of ultimate bearing capacity of soft soil on shear force

effectiveness of multilayer reinforced system for bending moment reduction is more in case of lower ultimate bearing capacity of soil.

It has been observed that for $q_{us}^* = 10$, the reduction of shear force at the edge of the beam from unreinforced case to single, double and three layered reinforced system is 3.4%, 9.5% and 15.3%, respectively (as shown in Fig. 5). However, for $q_{us}^* = 1$ the reduction is 10%, 18% and 24.4%, respectively (as shown in Fig. 11). It is also observed that for unreinforced case the shear force at the edge of the beam is increased by 29% as q_{us}^* decreases from 10 to 1. Thus, it can be said that the multilayered reinforced system is more effective for shear force reduction of the beam in case of lower ultimate bearing capacity of the soft soil.

The results obtained from the present study will be helpful to design the foundations resting on multilayered geosynthetic-reinforced granular fill-soft soil system. It is well understood that the use of beam with higher flexural rigidity reduces the maximum and differential settlement of the beam, but it increases the maximum bending moment and shear force of the beam. From the present study it is observed that for very rigid foundation, use of reinforcements will not help for maximum settlement reduction. However, the effectiveness of the multilayered reinforced system for shear force reduction is more in case very rigid beam. Thus, designer has to choose proper rigidity of the beam based on the design requirements and the preference to design variables. The multilayered reinforced system is very helpful for bending moment and differential settlement reduction even in case of very rigid beam. During the design of reinforced foundation soil under beam it has to be considered that up to two layered reinforced system is effective for maximum settlement reduction even in case of lower flexural rigidity of the beam. It is also noticed that for unreinforced case the maximum bending moment is occurred at the centre of the beam (for UDL), but as the number of reinforcement increases the location of maximum bending moment is shifted away from the centre. The use of granular fill with high shear modulus will not effective for shear force reduction if reinforcements are used. These are the useful information for designer to design the foundation resting on multilayered reinforced system.

4. Conclusions

The development of a mechanical model has been presented to study the effect of flexural rigidity of beam, shear modulus of granular layer and ultimate load carrying capacity of soft soil on settlement, bending moment and shear forced behavior of beam resting over multilayered reinforced system. It is observed that use of geosynthetic reinforcement is not effective for maximum settlement reduction in case of very rigid beam. The effectiveness of multilayered reinforced system (upto two layered reinforced system) for settlement reduction is more in case of lower flexural rigidity of the beam, lower shear modulus of granular fill and lower ultimate bearing capacity of the soft soil. The multilayered reinforced system is very effective for bending moment and differential settlement reduction. However, the effectiveness for bending moment reduction is more incase lower flexural rigidity of the beam, shear modulus of granular fill and ultimate bearing capacity of the soft soil. It is also observed that as the number of reinforcement layer increases the location of maximum bending moment is shifted towards the right side of the beam. For unreinforced case the maximum bending moment is occurred at the centre of the beam irrespective to the flexural rigidity of the beam. However, for geosynthetic-reinforced case the location of maximum bending moment is beyond X equal to 0.5 for lower flexural rigidity of the beam and for higher flexural rigidity of the beam it is in between X equal to 0 to 0.5. Use of reinforcement is not so effective for shear force reduction in case of higher shear modulus value of the granular layer. The effectiveness of the multilayered reinforced system in maximum shear force reduction is more for soft soil with lower ultimate bearing capacity, granular layer with lower shear modulus and beam with higher flexural rigidity.

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Notations

B half width of uniform surcharge load (m)
EI flexural rigidity of the beam (kN-m^2)
G_{j0} initial shear modulus of the granular fill layer 1,2,3,4, respectively (kN/m^2)
G^*_{j0} normalized G_{j0} (dimensionless)
H_j thickness of the granular fill layer 1,2,3,4, respectively (m)
H_s thickness of the soft soil (m)
I^* normalized EI, non dimensional
K_0 coefficient of lateral earth pressure at rest (dimensionless)
k_{s0} initial modulus of subgrade reaction for soft foundation soil (kN/m^3)
L half width of geosynthetic-reinforced zone (m)
M^* bending moment, non dimensional
p foundation bearing pressure (kN/m^2)
Q^* shear force, non dimensional
q footing pressure on the beam (kN/m^2)
q^* normalized q (dimensionless)
q_s vertical reaction pressure of the soft foundation soil (kN/m^2)
q^*_s normalized q_s (dimensionless)
q_u ultimate bearing capacity of the soft soil (kN/m^2)
q^*_u normalized q_u (dimensionless)
$T_{j=1,2,3}$ mobilized tension in the top, middle and bottom geosynthetic layer, respectively (kN/m)
$T^*_{j,1,2,3,4}$ normalized $T_{j=1,2,3}$ (dimensionless)
w vertical displacement (m)
W normalized w (dimensionless)
w_b vertical displacement of the beam, m
W_b normalized w_b, non dimensional
w_f vertical displacement of the granular layer beyond the beam, m
W_f normalized w_f, non dimensional
x distance from centre of loading (m)
X normalized x (dimensionless)
μ_1, μ_2 interface friction at the top and bottom of the top geosynthetic layer (dimensionless)
μ_2, μ_3 interface friction at the top and bottom of the middle geosynthetic layer (dimensionless)
μ_3, μ_4 interface friction at the top and bottom of the bottom geosynthetic layer (dimensionless)
ϕ' effective angle of shearing resistance (degree)
θ slope of the membrane (degree)
τ_j shear stresses in the granular layer 1, 2, 3, 4, respectively (kN/m^2)
τ^*_j normalized τ_j (dimensionless)
τ_{uj} ultimate shear resistance of the granular layer 1, 2, 3, 4, respectively (kN/m^2)
τ^*_{uj} normalized $\tau_{uj=1,2,3,4}$ (dimensionless)