Nonlinear Response of Infinite Beams on a Multilayer Tensionless Extensible Geo-Synthetic: Reinforced Earth Beds under Moving Load

K. Karuppasamy

Abstract—In this paper, analysis of an infinite beam resting on multilayer tensionless extensible geosynthetic reinforced granular fill-poor soil system overlying soft soil strata under moving load with constant velocity is presented. The beam is subjected to a concentrated load moving with constant velocity. The upper reinforced granular bed is modeled by a rough membrane embedded in Pasternak shear layer overlying a series of compressible nonlinear winker springs representing the underlying the very poor soil. The multilayer tensionless extensible geosynthetic layer has been assumed to deform such that at interface the geosynthetic and the soil have some deformation. Nonlinear behaviour of granular fill and the very poor soil has been considered in the analysis by means of hyperbolic constitutive relationships. Governing differential equations of the soil foundation system have been obtained and solved with the help of appropriate boundary conditions. The solution has been obtained by employing finite difference method by means of Gauss-Siedal iterative scheme. Detailed parametric study has been conducted to study the influence of various parameters on the response of soil–foundation system under consideration by means of deflection and bending moment in the beam and tension mobilized in the geosynthetic layer. These parameters include magnitude of applied load, velocity of load, damping, ultimate resistance of poor soil and granular fill layer. Range of values of parameters has been considered as per Indian Railway conditions. This study clearly observed that the comparisons of multilayer tensionless extensible geosynthetic reinforcement with poor foundation soil and magnitude of applied load, relative compressibility of granular fill and ultimate resistance of poor soil has significant influence on the response of soil–foundation system.

Keywords—Infinite beams, multilayer tensionless extensible geosynthetic, granular layer, moving load, nonlinear behavior of poor soil.

I. INTRODUCTION

REINFORCED earth is widely in use as the construction material in formation of subgrade for roads, railway tracks and in air strips to reduce the settlement and to increase the bearing capacity. Especially geotechnical engineers face several challenges in the construction of earth structures like retaining walls and embankments which cater to the development of transport infrastructure. Soil reinforcement plays a major part in strengthening of earth structures and utilization of soft foundation soils. Hence the need to develop new analytical methods for nonlinear response of multilayer tensionless extensible geosynthetic-reinforced foundation subjected to moving load under the very poor soil.

In the present work, modeling and analysis of an infinite beam resting on multilayer tensionless extensible geosynthetic reinforced-granular bed on soft soils has been studied with the lifting up partially and losing contact with the soils. The reinforcement has been considered multilayer tensionless extensible and compatibility conditions as suggested by [1], [4]-[8], [10]-[20] have been incorporated, reducing the number of parameters involved in this analysis. The foundation assumed to react only in compression. The nonlinear responses of multilayer tensionless extensible geosynthetic-reinforced foundation and foundation reaction in tension have been compared and further various parametric studies has been conducted considering values of input parameters reverent to the Indian railway conditions and the influence of various parameters of soil–foundation system. Finite difference method is used for the solution of governing differential equations of the model and all the results have been presented in non-dimensional forms. The properties of different layers of base, sub-base and foundation may be incorporated in the model by taking the equivalent stiffness of the nonlinear spring.

II. STATEMENT OF THE PROBLEM AND PROPOSED MODEL

Fig. 1 shows the definition sketch of the problem considered in the infinite beam resting on multilayer tensionless extensible geosynthetic reinforced granular fill – poor soil system. The infinite beam has been founded on a granular fill layer overlying poor foundation soil of thickness (H) and subjected to concentrated moving load (P). A three geosynthetic layers have been placed inside the granular fill layer which divides the granular fill layer into four, having thicknesses as H1, H2, H3 and H4 as shown in Fig. 1. The shear modulus of four layer of granular fill are G1, G2, G3, and G4 respectively while μ1 and μ2 are the interface coefficients at the top and bottom faces of the top geosynthetic layer respectively; μ2 and μ3 are the interface coefficients at the top and bottom faces of the middle geosynthetic layer respectively; μ3 and μ4 are the interface coefficients at the top
and bottom faces of the bottom geosynthetic layer respectively. The geosynthetic reinforcement is assumed to be inextensible with stiffness greater than or equal to 4000 kN/m; as beyond this value the stiffness of the reinforcement has no effect on the settlement response. The creep effect of the geosynthetic is neglected in the analysis. The response of the beam under the action is to be found out.

Fig. 2 depicts the proposed model for the soil – foundation model under consideration. The poor soil subgrade has been idealized as nonlinear Winkler springs and the granular fill layer as Pasternak Shear layer. The granular fill layer has been assumed to be incompressible and the beam has been assumed to have a perfect contact with granular fill layer. A rough elastic membrane has been used to model the geosynthetic layer. A hyperbolic nonlinear stress- displacement relationship proposed by [9] has been considered to exhibit the behaviour of granular fill and poor foundation soil.

Fig. 1 Definition sketch of multilayer geosynthetic- reinforced granular fill soft soil system

Fig. 2 Definition sketch of proposed model for soil foundation system

III. ANALYSIS

Fig. 3 presents the free body diagram of the first shear layer, the rough elastic membrane element on first geosynthetic layer, the second shear layer, the rough elastic membrane element on second geosynthetic layer, the third shear layer, the rough elastic membrane element on third geosynthetic layer and fourth shear layer elements.

The vertical force equilibrium equation of the first shear layer element (Fig. 3 (a)) can be written as;

$$q = q_1 - G_1 H_1 \frac{\partial^2 w(x,t)}{\partial x^2}$$

where, $q$ is the reaction of granular fill on the beam, $q_1$ is the vertical force interaction between the membrane and the first shear layer, $w(x,t)$ is the vertical surface deflection, $G_1$ is the shear modulus first shear layer, $H_1$ is the thickness of the first shear layer, $x$ is the horizontal space coordinate measured along the length of the beam and $t$ is any particular instant of time.
Fig. 3 Definition (a) forces on first shear layer; (b) forces on stretched rough elastic membrane element on first geosynthetic layer; (c) forces on second shear layer; (d) forces on stretched rough elastic membrane element on second geosynthetic layer; (e) forces on third shear layer; (f) forces on stretched rough elastic membrane element on third geosynthetic layer; (g) forces on fourth shear layer

The horizontal / vertical force equilibrium equation of the top rough elastic membrane element (Refer to Fig. 3 (b)) at time \( t > 0 \), can be written as:

\[
\cos \theta \frac{\partial T_1(x, t)}{\partial x} - T_1 \sin \theta \frac{\partial \theta}{\partial x} = -(\mu_1 q_1 + \mu_2 q_2) - K(q_0 - q_1) \tan \theta
\]

where, \( q_1 \) and \( q_2 \) are the vertical force interaction between the membrane, and the first and second shear layer respectively; \( \mu_1 \) and \( \mu_2 \) are the interface coefficients at the top and middle faces of membrane respectively, \( K \) is the coefficient of lateral stress, \( \theta \) is the slope of the membrane, \( T_1(x, t) \) is the tensile force per unit length mobilized in the top face of membrane.

The vertical force equilibrium equation of the second shear layer element (Fig. 3 (c)) can be written as:

\[
q_1 = q_0 - G_1 H_1 \frac{\partial^2 w(x, t)}{\partial x^2}
\]

where, \( q_0, q_1, q_2 \) are the vertical force interaction between the second shear layer and the poor foundation \( G_2 H_2 \) soil, and are the shear modulus and thickness of the second shear layer respectively.

The horizontal / vertical force equilibrium equation of the middle rough elastic membrane element (Fig. 3 (d)) at time \( t > 0 \), can be written as:

\[
\cos \theta \frac{\partial T_2(x, t)}{\partial x} - T_2 \sin \theta \frac{\partial \theta}{\partial x} = -(\mu_2 q_2 + \mu_3 q_3) - K(q_2 - q_3) \tan \theta
\]

where, \( q_1, q_2, q_3 \) are the vertical force interaction between the membrane and the second and third shear layer respectively; \( \mu_2 \) and \( \mu_3 \) are the interface coefficients at the middle and bottom faces of membrane respectively, \( T_2(x, t) \) is the tensile force per unit length mobilized in the middle face of membrane.

The vertical force equilibrium equation of the third shear
layer element (Fig. 3(e)) can be written as:

\[ q_i = q_i - G_i H_i \frac{\partial^2 w(x,t)}{\partial x^2} \]  

(7)

where, \( q_i \) is the vertical force interaction between the third shear layer and the poor foundation \( G_i H_i \) soil, and are the shear modulus and thickness of the third shear layer respectively.

The horizontal / vertical force equilibrium equation of the bottom rough elastic membrane element (Fig. 3(f)) at time \( t > 0 \), can be written as;

\[ \cos \frac{\partial T_i(x,t)}{\partial x} - T_i \sin \frac{\partial \theta}{\partial x} = -(\mu_i q_1 + \mu_i q_4) (q_4 - q_4 - q_3) \tan \theta \]  

(8)

\[ \sin \frac{\partial T_i(x,t)}{\partial x} + T_i \cos \frac{\partial \theta}{\partial x} = -K (q_4 - q_4 - q_3) \tan \theta + (q_3 - q_3) \]  

(9)

where, \( q_3 \) and \( q_4 \) are the vertical force interaction between the membrane and the third and fourth shear layer respectively; \( \mu_3 \) and \( \mu_4 \) are the interface coefficients at the bottom faces of membrane, \( T_i(x,t) \) is the tensile force per unit length mobilized in the bottom face of membrane.

The vertical force equilibrium equation of the fourth shear layer element (Fig. 3(g)) can be written as;

\[ q_i = q_i - G_i H_i \frac{\partial^2 w(x,t)}{\partial x^2} \]  

(10)

where, \( q_i \) is the vertical force interaction between the fourth layer of granular fill \( G_i H_i \) and the poor foundation soil, and are the shear modulus and thickness of the fourth layer of granular fill respectively from (4) and (5), one can write,

\[ q_i = q_3 + \frac{T_i \sec \theta \frac{\partial \theta}{\partial x} - (1 - K) (\mu_i q_3 + \mu_i q_4) \tan \theta}{(1 + K \tan^2 \theta)} \]  

(11)

Substituting for \( \frac{\partial \theta}{\partial x} \) in terms of vertical displacement, \( w(x,t) \) in to (13), and one can write;

\[ q_i = \frac{\partial T_i(x,t)}{\partial x} - T_i \frac{\partial \theta}{\partial x} = (1 - K) (\mu_i q_4 + \mu_i q_4) \tan \theta \]  

(12)

Similarly; from (7) and (8), we get,

\[ q_i = \frac{\partial T_i(x,t)}{\partial x} + T_i \frac{\partial \theta}{\partial x} \]  

(13)

From (10) and (11), we get

\[ q_i = \frac{\partial T_i(x,t)}{\partial x} - T_i \frac{\partial \theta}{\partial x} \]  

(14)

where,

\[ \frac{\partial T_i(x,t)}{\partial x} = \frac{1 + K \tan^2 \theta - (1 - K) \mu_i \tan \theta}{1 + K \tan^2 \theta + (1 - K) \mu_i \tan \theta} \]  

(15a)

\[ \frac{\partial T_i(x,t)}{\partial x} = \frac{1}{1 + K \tan^2 \theta + (1 - K) \mu_i \tan \theta} \]  

(15b)

\[ \frac{\partial T_i(x,t)}{\partial x} = \frac{1}{1 + K \tan^2 \theta + (1 - K) \mu_i \tan \theta} \]  

(15c)

\[ \frac{\partial T_i(x,t)}{\partial x} = \frac{1}{1 + K \tan^2 \theta + (1 - K) \mu_i \tan \theta} \]  

(15d)

\[ \frac{\partial T_i(x,t)}{\partial x} = \frac{1}{1 + K \tan^2 \theta + (1 - K) \mu_i \tan \theta} \]  

(15e)

Combining (1), (4), (7), (10), (12), (13) and (14),

\[ q_i = q_i - G_i H_i \frac{\partial^2 w(x,t)}{\partial x^2} \]  

(16)

From (4), (5), (7), (8), (10), and (11), we get

\[ \frac{\partial T_i(x,t)}{\partial x} = (q_i - q_i) (1 - K) \sin \theta - (1 + K \tan^2 \theta) \cos \theta \]  

(17)

\[ \frac{\partial T_i(x,t)}{\partial x} = -(q_3 - q_3) \tan \theta \]  

(18)

\[ \frac{\partial T_i(x,t)}{\partial x} = -(q_4 - q_4) \tan \theta \]  

(19)

\[ \frac{\partial T_i(x,t)}{\partial x} = -(q_4 - q_4) \tan \theta \]  

(20)

Combining (1), (4), (7), (10) and (11) following equation can be obtained:

\[ \frac{\partial T_i(x,t)}{\partial x} = (q_3 - q_3) (1 - K) \sin \theta - (1 + K \tan^2 \theta) \cos \theta \]  

(21)

\[ \frac{\partial T_i(x,t)}{\partial x} = \left( q_i + G_i H_i \frac{\partial^2 w(x,t)}{\partial x^2} \right) - \left( G_i H_i + X_i G_i H_i \right) \]  

(22)

\[ \frac{\partial T_i(x,t)}{\partial x} = \left( q_i + G_i H_i \frac{\partial^2 w(x,t)}{\partial x^2} \right) - \left( G_i H_i + X_i G_i H_i \right) \]  

(23)

where,

\[ \eta_i = (\mu_i \cos \theta (1 + K \tan^2 \theta) - (1 - K) \sin \theta) \]  

(24a)

\[ \eta_i = (\mu_i \cos \theta (1 + K \tan^2 \theta) + (1 - K) \sin \theta) \]  

(24b)

\[ \eta_i = (\mu_i \cos \theta (1 + K \tan^2 \theta) - (1 - K) \sin \theta) \]  

(24c)
\[
\bar{Y}_0 = (\mu_0 \cos(1 + K \tan^2 \theta)) + (1 - K) \sin \theta)
\]

(24d)

\[
\bar{Y}_1 = (\mu_0 \cos(1 + K \tan^2 \theta)) - (1 - K) \sin \theta)
\]

(24e)

\[
\bar{Y}_2 = (\mu_0 \cos(1 + K \tan^2 \theta)) + (1 - K) \sin \theta)
\]

(24f)

Considering the hyperbolic shear stress – shear strain response of the granular fill proposed by [3]; the shear modulus of different granular layers \((G_1, G_2, G_3, G_4)\) can be expressed as;

\[
G_i = \frac{G_{so}}{1 + \frac{k_{sw}}{G_{so}}}
\]

(25)

where, \(G_{so}\) is initial shear modulus of granular layer 1, 2, 3 and 4 respectively; \(\tau_0\) is the ultimate shear resistance of the granular layer 1, 2, 3 and 4 respectively; \(dw/dx\) is the shear strain.

Considering the hyperbolic nonlinear Stress- displacement relationship [9], \(q_0\) can be expressed as,

\[
q_i = \frac{k_{vo}}{1 + \frac{k_{vo}}{\mu}}
\]

(26)

where, \(k_{so}\) is the initial modulus of subgrade reaction of poor soil and \(q_0\) is the ultimate bearing capacity of the poor soil.

The differential equation of a moving load on the beam may be obtained by considering the bending of an elemental segment. The differential equation of the beam with uniform cross section can be written as:

\[
EI \frac{d^4w}{dx^4} + \rho \frac{d^2w}{dt^2} + c \frac{dw}{dt} + q = P(x,t)
\]

(27)

where, \(w(x, t)\) is the deflection of the beam, \(EI\) is the flexural rigidity of the beam, \(\rho\) is the mass per unit length of the beam, \(c\) is the coefficient of viscous damping per unit length of the beam, \(P(x, t)\) is the applied load intensity, In the absence of damping (14) can be written as,

\[
EI \frac{d^4w}{dx^4} + \rho \frac{d^2w}{dt^2} + q = P(x,t)
\]

(28)

Equations (16), (21), (22) and (23) govern the response of the proposal model in the absence of damping. For particular values of the parameters, these equations govern the response of existing models for beams on elastic foundation subjected to moving load [2].

**IV. SOLUTION OF THE GOVERNING EQUATIONS**

The response of system has been represented as a function of distance \((x)\) from the center of the beam at time \((t)\). For simplicity, substituting \(\xi = x - vt\) where, 'v' is the distance from the point of action of loading at time 't'. The governing differential equations have only one variable \(\xi\).

From (16), (21), (22) and (23) can be written as;

\[
q = q_i (\bar{X}_1, \bar{X}_2, \bar{X}_3, \bar{X}_4) - (G_1 H_1 + G_2 H_2 + G_3 H_3 + G_4 H_4) + \bar{X}_1 T_1 \cos \theta
\]

(29)

\[
\frac{\partial T_i}{\partial \xi} = -\bar{Y}_i \left( q_i + \frac{G_i H_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1^2} \right) - \bar{Y}_1 \left( \bar{X}_1 \bar{X}_1 T_1 \cos \theta \right)
\]

(30)

\[
\frac{\partial T_i}{\partial \xi} = -\bar{Y}_i \left( q_i + \frac{G_i H_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1^2} \right) - \bar{Y}_1 \left( \bar{X}_1 \bar{X}_1 T_1 \cos \theta \right)
\]

(31)

\[
\frac{\partial T_i}{\partial \xi} = -\bar{Y}_i \left( q_i + \frac{G_i H_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1^2} \right) - \bar{Y}_1 \left( \bar{X}_1 \bar{T}_1 \cos \theta \right)
\]

(32)

\[
EI \frac{d^2w}{d\xi^2} + \rho \frac{dw}{dt} + q = P(\xi)
\]

(33)

To observe the settlement response of the proposed model, (29)-(33) have been written in non-dimensional of finite difference from within the specified space domain for an interior node \(i\), one obtains;

\[
q' = A W, X_i, X_i - (G_1 + G_2 + G_3 + G_4) + \bar{X}_1 T_1 \cos \theta
\]

(34)

\[
T_{i+1} = T_i - \bar{Y}_i \Delta \xi \left( q_i + \frac{G_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1} \right) - \bar{Y}_1 \Delta \xi \left( \bar{X}_1 \bar{X}_1 A W, - (G_1 + G_2 + G_3 + G_4) + \bar{X}_1 T_1 \cos \theta \right)
\]

(35)

\[
T_{i+1} = T_i - \bar{Y}_i \Delta \xi \left( q_i + \frac{G_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1} \right) + \bar{Y}_1 \Delta \xi \left( (A W, \bar{X}_1) \cos \theta \right)
\]

(36)

\[
T_{i+1} = T_i - \bar{Y}_i \Delta \xi \left( q_i + \frac{G_i \frac{\partial^2 w}{\partial \xi^2}}{\bar{X}_1} \right) + \bar{Y}_1 \Delta \xi \left( (A W, \bar{X}_1) \cos \theta \right)
\]

(37)
where; \( P = \frac{P}{E_0} \), \( k_g = k_{g0} \), \( k_p = k_{p0} \), \( G = G_0 \), \( E \), \( I \), and \( \tau \). The solutions have been obtained with a tolerance limit of 10^{-5}. The finite difference formulation has been employed to solve the differential equations. In these equations, the derivatives are expressed by central difference method as follows:

\[
W_i = \frac{1}{6 - 2 \Delta x^2} \times \left[ \frac{P \Delta x^4}{\Delta z^2} - q_i^* \Delta x^2 \right] \times W_{i-2} - W_{i-1} - \left[ 4 \Delta x^2 \right] \times W_{i+1} - \left[ 4 \Delta x^2 \right] \times W_{i+2}
\]

(38)

RESULTS AND DISCUSSIONS

BOUNDARY CONDITIONS

Boundary conditions have been considered at the edge of the beam. At both ends of the beam, the deflection of the beam, the slope of the deflected shape of the beam and the mobilized tension are zero. These boundary conditions can be written in non-dimensional form as: at \( \xi^* = 0 \) and 1,

\[ W = 0, \frac{dW}{d\xi} = 0 \] and \( T^* = 0 \).

Since, from (34) to (38) are all nonlinear equations; an iterative computing procedure has been used for obtaining solutions. The solutions have been obtained with a convergence criterion as,

\[
\left| W_{i} - W_{i+1} \right| < \varepsilon \quad \left| \frac{T_{i}^* - T_{i+1}^*}{T_{i}^*} \right| < \varepsilon
\]

where; i is the no of elements; k & k-1 are the present and previous iteration values, respectively; \( \varepsilon \) is the specified tolerance limit, which is 10^{-10} in the present study.

VI. RESULTS AND DISCUSSIONS

Based on the formulation presented in previous chapter, a computer program was developed using finite difference scheme. Complete region of the problem (-1 \( \leq x \leq 1 \)) was divided into different numbers of elements and it was observed that the difference in results corresponding to 800 and 1000 numbers of elements was less than 0.5% hence 800 elements were used and the solution was obtained with a tolerance limit of 10^{-5}. Half the length of the beam is taken to be large enough for the beam to be assumed to act as an infinite beam. The following values of parameters have been adopted for the parameter study as shown in Table 1.

For a typical set of parameters, i.e., \( P = 5 \times 10^7 \), \( G_{00} = 3 \times 10^7 \), \( I = 6 \times 10^{-16} \), \( \tau_{u} = \tau_{ub} = 2.7 \times 10^{-9} \), \( q_{0} = 1.8 \times 10^{-5} \), \( \rho = 1.2 \times 10^{-7} \), \( \mu = 0.5 \) and \( K = 0.172 \). The comparison of nonlinear responses of the multilayer tension and multilayer tensionless extensible geosynthetic resting on nonlinearity in the behavior at poor soil which responds in infinite beam in terms of normalized deflection, normalized mobilized tension in top, middle & bottom geosynthetic layer, normalized bending moment, normalized soil reaction has been presented in Figs. 4-10. As expected, the deflection of beam has been found to increase the negative deflection of beam as the analysis considers non-linearity in the behavior at poor soil (Tensionless foundation) as shown in Fig. 5. So it is clear that a tensionless foundation affects the uniformly lift up of the beam (negative deflection) more as compared to its settlement (positive deflection). The comparisons are also made with respect to normalized bending moment of the beam and it is normalized mobilized tension in top, middle & bottom geosynthetic layer in Figs. 6 and 9 respectively for the same parameters as in case of deflection of beam. The maximum positive normalized bending moment is almost same for both the cases but the negative normalized bending moment decrease by around 4.6% in case of soil unable to take any tension under the nonlinear behavior of poor soil. The normalized mobilized tension is significantly affected and is negligible in case of tension foundation under the nonlinearity soil. The mobilized tension at the point of loading increases 10.8% when the soil react the tension and compression.
take any nonlinearity in the behaviour at poor foundation soil. The comparisons are also made with respect to normalized mobilized tension in multilayer extensible geosynthetic layers and they are normalized bending moment of the beam in Figs. 4 and 5 respectively for the same parameters as in case of deflection of beam. The maximum mobilized tension of multilayer extensible geosynthetic layers and normalized bending moment of the beam have been found to be almost same for both the cases for linear analysis as compared to that for nonlinear analysis.

### TABLE I

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Range of values</th>
<th>Unit</th>
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<tr>
<td>Applied Load</td>
<td>P</td>
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<td>KN</td>
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<td>Mass per unit length of beam</td>
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<td>Velocity of applied load</td>
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<td>Km/hr</td>
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<td>Ultimate shear resistance of granular fill</td>
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<td>KN/m²</td>
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<td>Half-length of beam</td>
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<td>m</td>
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<td>Coefficient of lateral earth pressure</td>
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<td>Interfacial friction coefficient at top and bottom reinforcement</td>
<td>μt0 to μt4</td>
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Fig. 4 Typical Settlement profiles for soil responding to tension and soil not responding to tension (i.e. Tensionless Foundation)

Fig. 5 Typical negative settlement profiles for soil responding to tension and soil not responding to tension

Fig. 6 Typical bending moment of beam for soil responding to tension and soil not responding to tension

Fig. 7 Typical mobilized tension profiles of top geosynthetic layer for soil responding to tension and soil not responding to tension
Fig. 8 Typical mobilized tension profiles of middle geosynthetic layer for soil responding to tension and soil not responding to tension

Fig. 9 Typical mobilized tension profiles of bottom geosynthetic layer for soil responding to tension and soil not responding to tension

Fig. 10 Typical settlement profiles for linear and nonlinear response of multilayer tensionless extensible geosynthetic reinforced earth beds

Fig. 11 Typical bending moment of beam for linear and nonlinear response of multilayer tensionless extensible geosynthetic reinforced earth beds

Fig. 12 Typical mobilized tension of multilayer geosynthetic layer for linear and nonlinear response of multilayer tensionless extensible geosynthetic reinforced earth beds

Fig. 13 Typical soil reaction profiles for linear and nonlinear response of multilayer tensionless extensible geosynthetic reinforced earth beds

The influence of ultimate resistance of poor foundation soil for multilayer extensible geosynthetic reinforced on response
of soil-foundation system has been depicted in Figs. 14-18 for parameters, i.e., $P^* = 5.42 \times 10^{-5}$, $G_0^* = G_4^* = 1.63 \times 10^{-5}$, $K = 0.172$, $I^* = 3.31 \times 10^{-8}$, $\tau_{u1}^* = \tau_{u4}^* = 1.5 \times 10^{-4}$; $\mu_1 = \mu_4 = 0.5$; and $\rho^* = 3.12 \times 10^{-5}$.

It can be observed that ultimate resistance of poor foundation soil significantly affect the response of system under consideration. The maximum normalized deflection has been found to reduce by 56% (Fig. 14) as the normalized ultimate resistance of poor foundation soil increase from $2.0 \times 10^4$ to $6.0 \times 10^4$. Figs. 15-17 show the effect of parameter $q_u$ on normalized mobilized tension in top, middle & bottom of tensionless extensible geosynthetic layers. As expected, the mobilized tension in multilayer tensionless extensible geosynthetic layers have been found to reduce with an increase in the parameter, $q_u$. A reduction of about 67.3% in tension mobilized in multilayer tensionless extensible geosynthetic layers have been observed for various ultimate resistance of poor soil. The corresponding reduction in maximum normalized bending moment has been found to be about 77% (Fig. 18).
The influence of ultimate resistance of poor foundation soil for multilayer extensible geosynthetic reinforced on response of soil-foundation system for the effect of magnitude of applied load has been depicted in Figs. 19-24 for the parameters, i.e. $G_{lt}=G_{bo}=3 \times 10^7$, $l^*=6 \times 10^{-10}$, $\tau_{ut}=\tau_{ub}$.

Fig. 19 Typical soil reaction variations for various load intensity.

Fig. 20 Typical mobilized tension of top geosynthetic layer for various velocity values.

Fig. 21 Typical mobilized tension of middle geosynthetic layer for various velocity values.

Fig. 22 Typical mobilized tension of bottom geosynthetic layer for various velocity values.

Fig. 23 Typical soil reaction variations for various load intensity.

Fig. 24 Typical bending moment of beam for various soil reactions.
2.7 \times 10^{-9}, \mu_b = 0.5, K = 0.172, q_0^* = 1.8 \times 10^{-5} \text{ and } \rho^* = 1.2 \times 10^{-7}.

The normalized magnitude of applied load has been varied from 3.0 \times 10^{-7} to 7.5 \times 10^{-7} and corresponding reduction in the maximum deflection of beam has been found to be 67 \% (Fig. 19). It can be seen that deflection of the ground surface becomes zero when the deflection of the beam is negative, i.e., when beam is lifted up; there is a separation between the beam and the ground surface. The corresponding reduction in maximum bending moment in the beam has been found to be 66 \% (Fig. 24). Any increase in magnitude of load intensity causes more deflection and hence more bending moment. The influence of applied load on the tension mobilized in multilayer tensionless geosynthetic layer has been depicted in Figs. 20-22. The maximum mobilized tension occurs at the point of application of load and reduces on either side. This has been found to reduce by 66.7 \% as the applied load reduces from 6.75 \times 10^{-8} to 2.23 \times 10^{-8}. Soil reaction has been shown in Fig. 23 for different values of applied load as considered and the reduction in maximum soil reaction has been found to be about 66.7\% for the reduction in applied load from 4.26 \times 10^{-6} to 1.421 \times 10^{-6}.
Effect of velocity of applied load, for multilayer extensible geosynthetic reinforced on response of soil-foundation system has been depicted in Figs. 25-29. For parameters, i.e., $P^* = 5.42 \times 10^{-6}$; $G_0^* = 1.63 \times 10^{-6}$; $K^* = 0.172$; $I^* = 3.31 \times 10^{-6}$, $\tau_{u1}^* = 1.5 \times 10^{-4}$; $\mu_1 = \mu_4 = 0.5$ and $q_u^* = 5.0 \times 10^{-4}$ on the deflection profile of infinite beam has been studied for the parameter velocity varying from 10 to 60. The maximum deflection has been found to reduce by about 3% as the velocity ranging from $3.43 \times 10^{-6}$ to $3.3 \times 10^{-6}$, the maximum normalized deflection of the beam can reduce by about 3% for the range of parameters considered in the study.

The ultimate resistance of poor soil has been found significantly affect the response of infinite beam and the geosynthetic layer. Deflection (about 56%) and bending moment in the beam (about 77%) has been found to reduce with an increase in the ultimate resistance of poor soil. Tension mobilized in the geosynthetic layer has been found to reduce by extent 67.3% for an increase in parameter $q_u$ from $2.0 \times 10^{-6}$ to $6.0 \times 10^{-6}$.

REFERENCES


Fig. 30 Typical bending moment of beam for various velocity values

VII. CONCLUSIONS

The proposed model analysis an infinite beam resting on multilayer tensionless extensible geosynthetic reinforced granular fill poor soil system under a concentrated load moving with constant velocity is presented. The analysis takes into account multilayer tensionless extensible geosynthetic layers. As expected, the mobilized tension in multilayer tensionless extensible geosynthetic layers has been found to reduce with an increase in the velocity of applied load. A reduction of about 10% in tension mobilized in multilayer extensible geosynthetic layers have been observed corresponding to an increase in velocity varying from $7.31 \times 10^{-6}$ to $7.22 \times 10^{-6}$. Figs. 29 and 30 depict the soil reaction and bending moment respectively for various velocity of applied load. Although there is same influence of velocity of applied load on the response of soil - foundation system, however this has not been found to be significant.


