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Influence of inclusion of geosynthetic layer on response of combined footings on stone column reinforced earth beds

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Abstract. The present paper deals with the analysis of combined footings resting on geosynthetic reinforced granular fill overlying stone column improved poor soil. An attempt has been made to study the influence of inclusion of geosynthetic layer on the deflection of the footing. The footing has been idealized as a beam having finite flexural rigidity. Granular fill layer has been represented by Pasternak shear layer and stone columns and poor soil have been represented by nonlinear Winkler springs. Nonlinear behavior of granular fill layer, stone columns and the poor soil has been considered by means of hyperbolic stress strain relationships. Governing differential equations for the soil-foundation system have been derived and solution has been obtained employing finite difference scheme by means of iterative Gauss Elimination method. Results of a detailed parametric study have been presented, for a footing supporting typically five columns, in non-dimensional form in respect of deflection with and without geosynthetic inclusion. Geosynthetic layer has been found to significantly reduce the deflection of the footing which has been quantified by means of parametric study.

Keywords: Stone columns, combined footing, geosynthetic layer, nonlinear behavior.

1. introduction

Geosynthetic - reinforced granular fill - poor soil system is quite widely used as a foundation for unpaved roads, residential buildings, low embankments, storage tanks etc. (Han and Gabr, 2002). Piles or columnar element in the form of stone columns or granular piles are also used in conjunction with geosynthetic - reinforced granular fills. This is quite effective in reducing the excessive settlement and enhancing the bearing capacity. Further, rate of consolidation increases with the provision of stone columns and as a result most of the settlement occurs during the construction period.

Many studies have been conducted for the analysis of stone column treated soil system. Some of these include Balaam and Booker (1981), Alamgir *et al.* (1996), Poorooshasb and Meyerhof (1997), Lee and Pande (1998), Shahu *et al.* (2000), Deb (2008) and Maheshwari and Khatri (2010) etc. However, all these studies do not make use of geosynthetic as a part of soil-foundation system.

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Some of the studies pertaining to the analysis of geosynthetic reinforced earth beds include Madhav and Poorooshasb (1998), Ghosh and Madhav (1994), Shukla and Chandra (1994), Yin (1997), Nogami and Yong (2003), Maheshwari *et al.* (2004) etc. In all these studies, the natural soil was not reinforced with stone columns. Deb *et al.* (2007) presented a generalized model for analysis of geosynthetic - reinforced granular fill - soft soil with stone columns system. However, the flexural rigidity of the footing was not considered in the analysis and the stone columns were assumed to exhibit linear stress - strain behavior.

In the present work, a mechanical model for the combined footing (having finite flexural rigidity) resting on geosynthetic - reinforced granular fill - stone column treated poor soil system has been proposed. The model incorporates the nonlinear behavior of granular fill, stone columns and the poor soil. The governing differential equations have been derived and their numerical solution has been obtained by an iterative finite difference scheme with due consideration of appropriate boundary and loading conditions. Detailed parametric study has been carried out and all the results have been presented in non-dimensional form.

2. Modeling and analysis of soil-foundation system

Fig. 1 shows a combined footing resting on geosynthetic - reinforced granular fill - soft soil system with stone column inclusions. This soil - foundation system has been modeled as shown in Fig. 2. The footing has been assumed to have finite flexural rigidity (EI) and therefore idealized as a beam of finite length (= $2B$). The footing is resting on the geosynthetic-reinforced granular fill of width $2L$ over poor soil which has been treated by stone columns. Stone columns and poor soil have been represented by layers of Winkler springs of different stiffnesses. The granular fill layer has been idealized as Pasternak shear layer. Granular fill, poor soil and the stone columns has been assumed to exhibit nonlinear stress - strain behavior. Geosynthetic layer in between the granular fill layer has been idealized as linear, elastic and rough membrane. While modeling the system, some of the assumptions have been made. The influence of disturbance due to installation of stone columns has

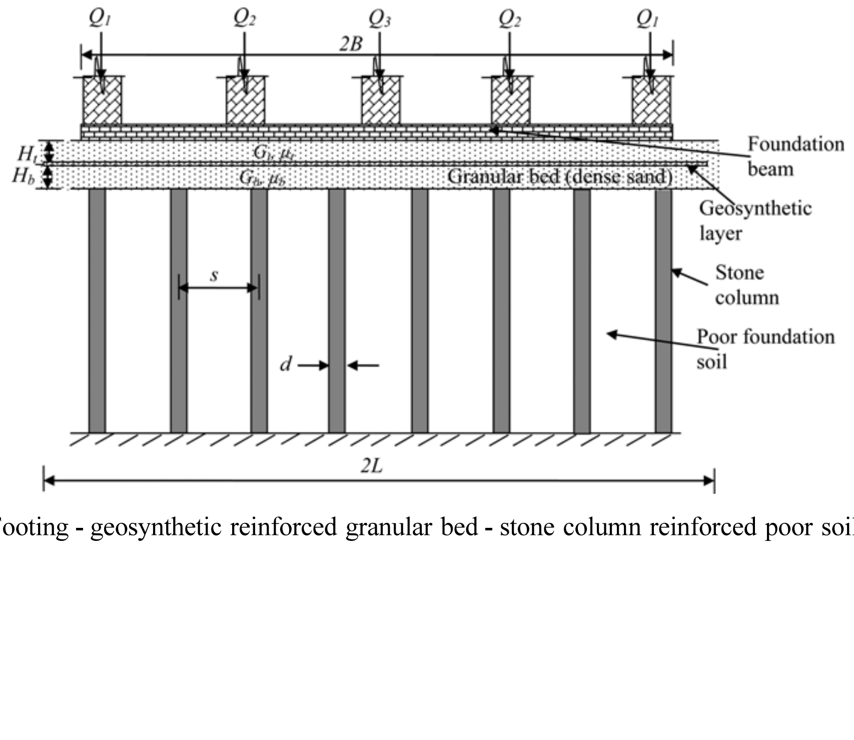

Fig. 1 Footing - geosynthetic reinforced granular bed - stone column reinforced poor soil system

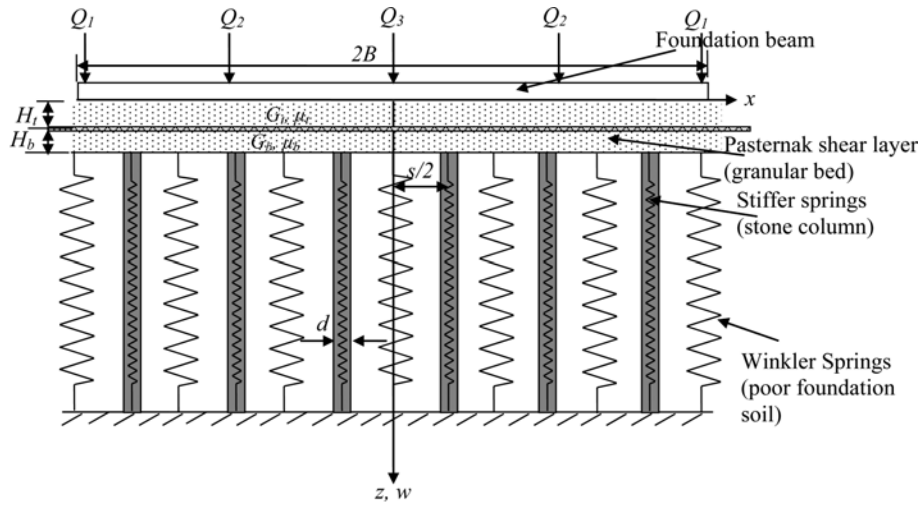

Fig. 2 Proposed model for soil - foundation system

not been considered in the analysis. The length of stone columns has been assumed to be equal to the thickness of soft soil stratum. Small deformations have been considered in the analysis.

Fig. 1 depicts the definition sketch of a combined footing of length $2B$ supporting typically five columns. The loads getting transferred to the footing through equi-spaced columns are Q_1, Q_2 and Q_3 as shown in the Figure. The footing is resting on a granular bed of width $2L$ on top of the stone column treated poor soil. There exists a layer of geosynthetic in between the granular bed. The top and bottom thicknesses of granular fill layer are H_t and H_b respectively and shear modulii are G_t and G_b respectively. μ_t and μ_b are the interfacial friction coefficients at the top and bottom of the reinforcement layer respectively. Diameter and spacing of stone columns is d and s respectively. The flexural rigidity of footing is EI.

Fig. 3 presents the free body diagram of the top shear layer, membrane and the bottom shear layer elements. The vertical force equilibrium equation of the top shear layer element, at time $t > 0$, can be written as

$$q = q_t - \frac{\partial \tau_t}{\partial x} H_t \tag{1}$$

Similarly, the vertical force equilibrium of bottom shear layer element, at time $t > 0$, can be written as

$$q_b = q_s - \frac{\partial \tau_b}{\partial x} H_b \tag{2}$$

where, q is the reaction of granular fill on beam, q_t is the vertical force interaction between the membrane and the top shear layer, q_b is the normal stress acting on the top of bottom shear layer, q_s is the vertical force interaction between bottom shear layer and the poor soil, τ_t and τ_b are shear stresses in the top and bottom shear layer respectively, w is the vertical surface deflection, x is the horizontal space coordinate measured along the length of the beam.

The nonlinear shear stress - shear strain response of the granular fill has been considered as

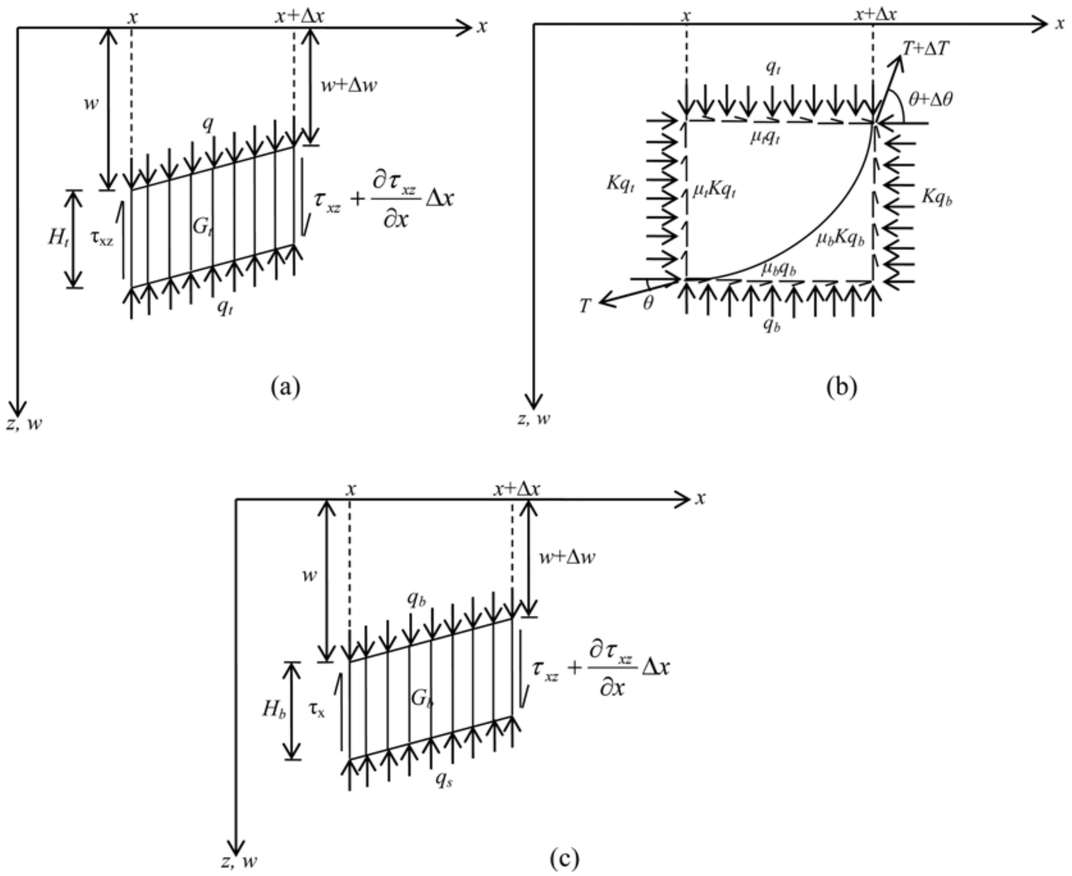

Fig. 3 Definition sketch: (a) forces on the top shear layer, (b) forces on the stretched, rough, elastic membrane element and (c) forces on the bottom shear layer element

$$\tau_t = \frac{G_{to}(\partial w/\partial x)}{1 + \frac{G_{to}|\partial w/\partial x|}{\tau_{ut}}} \tag{3}$$

and,

$$\tau_b = \frac{G_{bo}(\partial w/\partial x)}{1 + \frac{G_{bo}|\partial w/\partial x|}{\tau_{ub}}} \tag{4}$$

where, G_{to} and G_{bo} are the initial shear modulus of top and bottom shear layers respectively and τ_{ut} and τ_{ub}, the ultimate shear resistance of top and the bottom shear layers respectively.

Combining Eqs. (1) and (3); Eqs. (2) and (4), and denoting

$$G_t = \frac{G_{to}}{\left[1 + \frac{G_{bo}|\partial w/\partial x|}{\tau_{ut}}\right]^2} \tag{5}$$

and $$G_b = \frac{G_{bo}}{\left[1 + \frac{G_{bo}|\partial w/\partial x|}{\tau_{ub}}\right]^2} \tag{6}$$

where, it has been assumed that $\frac{\partial^2 w}{\partial x^2}\left|\frac{\partial w}{\partial x}\right| = \frac{\partial w}{\partial x}\left|\frac{\partial^2 w}{\partial x^2}\right|$

Eqs. (1) and (2) can be rewritten as

$$q = q_t - G_t H_t \frac{\partial^2 w}{\partial x^2} \tag{7}$$

and $$q_b = q_s - G_b H_b \frac{\partial^2 w}{\partial x^2} \tag{8}$$

Considering the hyperbolic nonlinear stress-displacement relationship (Kondner and Zelasko 1963), q_s can be expressed as

$$q_s = \frac{k_{so} w}{1 + k_{so}(w/q_u)}, \text{ within the poor foundation soil} \tag{9}$$

and

$$q_s = \frac{k_{co} w}{1 + k_{co}(w/q_{cu})}, \text{ within the stone column region} \tag{10}$$

where, k_{so} and k_{co} are initial modulus of subgrade reaction of poor foundation soil and the stone columns respectively. q_u and q_{cu} are ultimate bearing capacity of poor foundation soil and the stone columns respectively.

The horizontal force equilibrium equation of the rough elastic membrane element (Fig. 3) at time $t > 0$, can be written as

$$\cos\theta \frac{\partial T}{\partial x} - T \sin\theta \frac{\partial \theta}{\partial x} = -(\mu_t q_t + \mu_b q_b) - K(q_t - q_b)\tan\theta \tag{11}$$

where, K is the coefficient of lateral stress, θ is the slope of the membrane, T is the mobilized tensile force per unit length in the membrane.

The vertical force equilibrium equation for the rough elastic membrane element at time, $t > 0$, can be written as

$$\sin\theta \frac{\partial T}{\partial x} + T \cos\theta \frac{\partial \theta}{\partial x} = (q_t - q_b) - K(\mu_t q_t + \mu_b q_b)\tan\theta \tag{12}$$

From Eqs. (11) and (12), one can write

$$q_t = q_b + \frac{T \sec\theta}{1 + K\tan^2\theta}\left(\frac{\partial \theta}{\partial x}\right) - \frac{(1-K)(\mu_t q_t + \mu_b q_b)\tan\theta}{1 + K\tan^2\theta} \tag{13}$$

Substituting for $\frac{\partial \theta}{\partial x}$ in terms of vertical displacement, w, into Eq. (13), one can write

$$q_t = \overline{X_1} q_b - \overline{X_2} T \cos\theta \frac{\partial^2 w}{\partial x^2} \tag{14}$$

where

$$\overline{X_1} = \frac{1 + K \tan^2\theta - (1-K)\mu_b \tan\theta}{1 + K \tan^2\theta + (1-K)\mu_b \tan\theta} \tag{15a}$$

$$\overline{X_2} = \frac{1}{1 + K \tan^2\theta + (1-K)\mu_t \tan\theta} \tag{15b}$$

From Eqs. (12) and (13)

$$\frac{\partial T}{\partial x} = (q_t - q_b)(1-K)\sin\theta - (\mu_t q_t + \mu_b q_b)(1 + K \tan^2\theta)\cos\theta \tag{16}$$

or

$$\frac{\partial T}{\partial x} = -\overline{X_3} q_t - \overline{X_4} q_b \tag{17}$$

where

$$\overline{X_3} = \mu_t \cos\theta(1 + K \tan^2\theta) - (1-K)\sin\theta \tag{18a}$$

$$\overline{X_4} = \mu_b \cos\theta(1 + K \tan^2\theta) + (1-K)\sin\theta \tag{18b}$$

Combining Eqs. (7), (8), (9) and (14), one can obtain the equation within poor foundation soil as

$$q = \overline{X_1} \frac{k_{so} w}{[1 + k_{so}(w/q_u)]} - (G_t H_t + \overline{X_2} T \cos\theta + G_b H_b \overline{X_1}) \frac{\partial^2 w}{\partial x^2} \tag{19a}$$

Similarly, combining Eqs. (7), (8), (10) and (14), the resulting equation within stone column region can be obtained as

$$q = \overline{X_1} \frac{k_{co} w}{[1 + k_{co}(w/q_{cu})]} - (G_t H_t + \overline{X_2} T \cos\theta + G_b H_b \overline{X_1}) \frac{\partial^2 w}{\partial x^2} \tag{19b}$$

The equation for tension mobilized in the geosynthetic layer within poor soil, at any time $t > 0$, can be obtained by combining Eqs. (7), (8), (9) and (17) as

$$\frac{\partial T}{\partial x} = -\overline{X_3}\left(q + G_t H_t \frac{\partial^2 w}{\partial x^2}\right) - \overline{X_4}\left(\frac{k_{so} w}{[1 + k_{so}(w/q_u)]} - G_b H_b \frac{\partial^2 w}{\partial x^2}\right) \tag{20a}$$

Similarly, within the stone column region

$$\frac{\partial T}{\partial x} = -\overline{X_3}\left(q + G_t H_t \frac{\partial^2 w}{\partial x^2}\right) - \overline{X_4}\left(\frac{k_{co} w}{[1 + k_{co}(w/q_{cu})]} - G_b H_b \frac{\partial^2 w}{\partial x^2}\right) \tag{20b}$$

The differential equation of a beam can be obtained by considering the bending of an elemental segment. The differential equation of the beam with uniform cross section can be written as follows

$$EI \frac{d^4 w}{dx^4} + q = p \tag{21}$$

where, EI is the flexural rigidity of the beam and p, the externally applied load intensity.

Eqs. (19-21) are the governing differential equations of the soil - foundation system below the beam. Beyond the length of footing and in the absence of any external loading, the governing differential equations can be obtained by substituting q equal to zero in Eqs. (19-21).

The governing differential equations have been converted into their non-dimensional form employing following non-dimensional parameters

$X = x/B$, $W = w/B$, $G_t^* = G_t H_t/k_{so} B^2$, $G_{to}^* = G_{to} H_t/k_{so} B^2$, $G_b^* = G_b H_b/k_{so} B^2$, $G_{bo}^* = G_{bo} H_b/k_{so} B^2$, $I^* = EI/k_{so} B^4$, $q^* = q/k_{so} B$, $q_u^* = q_u/k_{so} B$, $q_{cu}^* = q_{cu}/k_{co} B$, $t_{ut}^* = t_{ut} H_t/k_{so} B^2$, $t_{ub}^* = t_{ub} H_b/k_{so} B^2$, $Q_1^* = Q_1/k_{so} B^2$, $Q_2^* = Q_2/k_{so} B^2$, $Q_3^* = Q_3/k_{so} B^2$, $p^* = p/k_{so} B$ and $a = k_{co}/k_{so}$.

Eqs. (19-21) can be written in non-dimensional form as

$$q^* = \overline{X_1}\frac{W}{1+(W/q_u^*)} - (G_t^* + \overline{X_2}T^*\cos\theta + G_b^*\overline{X_1})\frac{\partial^2 W}{\partial X^2}, \text{ within poor soil} \tag{22a}$$

and

$$q^* = \overline{X_1}\frac{\alpha W}{1+(W/q_{cu}^*)} - (G_t^* + \overline{X_2}T^*\cos\theta + G_b^*\overline{X_1})\frac{\partial^2 W}{\partial X^2}, \text{ within stone column region} \tag{22b}$$

$$\frac{\partial T^*}{\partial X} = -\overline{X_3}\left(q^* + G_t^*\frac{\partial^2 W}{\partial X^2}\right) - \overline{X_4}\left(\frac{W}{[1+(W/q_u^*)]} - G_b^*\frac{\partial^2 W}{\partial X^2}\right), \text{ within poor soil} \tag{23a}$$

and

$$\frac{\partial T^*}{\partial X} = -\overline{X_3}\left(q^* + G_t^*\frac{\partial^2 W}{\partial X^2}\right) - \overline{X_4}\left(\frac{\alpha W}{[1+(W/q_{cu}^*)]} - G_b^*\frac{\partial^2 W}{\partial X^2}\right), \text{ within stone column region} \tag{23b}$$

where, $G_t^* = \dfrac{G_{to}^*}{\left[1+\dfrac{G_{to}^*|dW/dX|}{\tau_{ut}^*}\right]^2}$ and $G_b^* = \dfrac{G_{bo}^*}{\left[1+\dfrac{G_{bo}^*|dW/dX|}{\tau_{ub}^*}\right]^2}$

$$\frac{d^4 W}{dX^4} + \frac{q^*}{I^*} - \frac{p^*}{I^*} = 0 \tag{24}$$

A finite difference scheme has been employed to solve the governing differential equations (Eqs. 22, 23 and 24) of the soil - foundation system. Due to symmetry, half of the spatial domain has been considered in the analysis. The equations can be written in finite difference form for a node, i, at any time, $t > 0$, as

$$q_i^* = \overline{X_{1i}}\frac{W_i}{[1+(W_i/q_u^*)]} - (G_{ti}^* + \overline{X_{2i}}T_i^*\cos\theta_i + G_{bi}^*\overline{X_{1i}})\left\{\frac{W_{i-1} - 2W_i + W_{i+1}}{(\Delta X)^2}\right\}, \text{ within poor soil} \tag{25a}$$

and

$$q_i^* = \overline{X_{1i}}\frac{\alpha W_i}{[1+(W_i/q_{cu}^*)]} - (G_{ti}^* + \overline{X_{2i}}T_i^*\cos\theta_i + G_{bi}^*\overline{X_{1i}})\left\{\frac{W_{i-1} - 2W_i + W_{i+1}}{(\Delta X)^2}\right\}, \text{ within stone column region} \tag{25b}$$

$$\frac{T^*_{i-1}-T^*_i}{\Delta X} = -\overline{X_{3i}}\left(q^*_i + G^*_{ti}\frac{\partial^2 W}{\partial X^2}\bigg|_i\right) - \overline{X_{4i}}\left(\frac{W_i}{[1+(W_i/q^*_u)]} - G^*_{bi}\frac{\partial^2 W}{\partial X^2}\bigg|_i\right), \text{ within poor soil} \tag{26a}$$

and

$$\frac{T^*_{i-1}-T^*_i}{\Delta X} = -\overline{X_{3i}}\left(q^*_i + G^*_{ti}\frac{\partial^2 W}{\partial X^2}\bigg|_i\right) - \overline{X_{4i}}\left(\frac{\alpha W_i}{[1+(W_i/q^*_{cu})]} - G^*_{bi}\frac{\partial^2 W}{\partial X^2}\bigg|_i\right), \text{ within stone column region} \tag{26b}$$

$$\frac{W_{i-2}-4W_{i-1}+6W_i-4W_{i+1}-W_{i+2}}{(\Delta X)^4} + \frac{q^*_i}{I^*} - \frac{p^*_i}{I^*} = 0 \tag{27}$$

where, at $X = 1/2$, $p^*_i = Q^*_2/\Delta X$ and elsewhere, $p^*_i = 0$.

The above Eqs. (25-27) represent the finite difference form of the governing equations below the footing, i.e., $0 \leq X \leq 1$. For $1 \leq X \leq L/B$, the equations are obtained by substituting $q^* = 0$ in Eqs. (25-27).

2.1 Boundary conditions

Due to simplicity, symmetry with respect to geometry and the loading has been assumed and therefore, half of the soil - foundation system has been considered in the analysis. At the centre of foundation beam, i.e., at $x = 0$, slope of deflected shape of beam is zero and the shear force is $Q_3/2$. Further, due to symmetry and the nature of applied load, mobilized tension in geosynthetic layer at $x = 0$ is zero. At the edge of the beam ($x = B$), bending moment is zero and the shear force is the summation of shear force due to applied concentrated load, Q_1, shear force due to Pasternak shear layer and the tension mobilized in geosynthetic layer. The geosynthetic layer is horizontal at its edges and therefore the slope dw/dx, will be zero at the location $x = L$. Further, due to free end of geosynthetic at $x = L$, the mobilized tension will be zero. The continuity at the edges of the stone columns has been automatically satisfied. The boundary conditions can be written in non-dimensional form as

$$\text{At } X = 0,\ \frac{dW}{dX} = 0,\ \frac{\partial^3 W}{\partial X^3} = -\frac{Q^*_3}{2I^*},\ \text{and } T^* = 0 \tag{28}$$

$$\text{At } X = 1,\ \frac{dW}{dX} = 0 \text{ and } \frac{\partial^3 W}{\partial X^3} - \left[\frac{G^*_t + X_2 T^* \cos\theta + G^*_b X_1}{I^*}\right]\frac{dW}{dX} = \frac{Q^*_1}{I^*} \tag{29}$$

$$\text{At } X = L/B,\ \frac{dW}{dX} = 0 \text{ and } T^* = 0 \tag{30}$$

Gauss Elimination iterative scheme has been employed for obtaining the solution of the governing differential equations along with appropriate boundary conditions as mentioned above. Once the deflection of foundation beam all along its length and the mobilized tension in geosynthetic layer has been determined, bending moment in the beam has been obtained by taking the derivatives of deflection profile.

3. Convergence criterion and range of parametric values

Half length of the spatial domain under consideration (L) was discretized finite difference wise and it was observed that the difference in response corresponding to finite difference mesh with 801 nodes and 1001 nodes was less than 1.0% and hence the mesh with 801 nodes was preferred for all parametric studies. The solution has been obtained following the convergence criterion as

$$\frac{W_i^k - W_i^{k-1}}{W_i^k} < \varepsilon \tag{31}$$

where, for every node i, k and $(k-1)$ are present and previous iteration respectively. e is the tolerance limit which has been assigned as 10^{-10} in the present study.

Influence of inclusion of geosynthetic layer in the soil foundation system under consideration has been studied by means of a detailed parametric study. The stone columns have been arranged in a symmetrical manner as shown in Fig. 2. It has been observed that the stone columns are effective when provided below the footing. No significant reduction in the settlement has been observed in the case when stone columns are provided beyond the footing. Therefore, for parametric study, the stone columns have been considered below the footing only. Further, optimum width of the reinforcement has been observed to be 2 times the width of loaded region (Maheshwari *et al.* 2004, Deb *et al.* 2007) and therefore length of reinforcement has been taken as twice the length of footing in the present study.

Realistic and relevant values of different input model parameters have been adopted for the parametric study. These have been presented in Table 1. These parameters have been non-dimensionalized (as presented in above section) and presented in Table 2. For simplicity, all column loads have been considered to be of same magnitude, i.e., $Q_1^* = Q_2^* = Q_3^* = Q^*$. The geosynthetic layer has been placed in the middle of granular fill layer. The ultimate resistance and initial shear modulus of top

Table 1 Range of values of various parameters considered for parametric study

Parameter	Symbol	Range of values	Unit
Applied load	Q_1=Q_2=Q_3=Q	100-300	kN
Flexural Rigidity of beam	EI	15-300	MN-m^2
Initial modulus of subgrade reaction for soft foundation soil	k_{so}	10 (Bowles 1997, Das 1999)	MN/m^3
Half length of foundation beam	B	10.0	m
Thickness of granular fill layers	H_t and H_b	0.15	m
Diameter of stone columns	d	0.2-1.0	m
Spacing to diameter ratio for stone column	s/d	2.5-4 (Som and Das 2003)	-
Initial shear modulus of granular fill	G_{to} and G_{bo}	652.4 (Desai and Abel 1987)	kN/m^2
Ultimate bearing capacity of soft foundation soil	q_u	20-60	kN/m^2
Ultimate bearing capacity of stone column	q_{cu}	100-200	kN/m^2
Ultimate shear resistance of granular fill layer	τ_{ut} and τ_{ub}	4-10	kN/m^2
Coefficient of lateral earth pressure	K	0.172	-
Interfacial friction coefficient at top and bottom of reinforcement	μ_t and μ_b	0.5	-

Table 2 Range of values of non-dimensional parameters considered for parametric study

Non-dimensional parameter	Expression	Range of values
Q^*	$Q/k_{so}\,B^2$	$1 \times 10^{-4} - 3 \times 10^{-4}$
I^*	$E\,I/k_{so}\,B^4$	$1.5 \times 10^{-4} - 3 \times 10^{-3}$
$G_{to}{}^*$ and $G_{bo}{}^*$	$G_{to}\,H_t/k_{so}\,B^2$; $G_{bo}\,H_b/k_{so}\,B^2$	9.8×10^{-5}
$q_u{}^*$	$q_u/k_{so}\,B$	$2 \times 10^{-4} - 6 \times 10^{-4}$
$q_{cu}{}^*$	$q_{cu}/\alpha\,k_{so}\,B$ (corresponding to $\alpha = 25$)	$4 \times 10^{-5} - 8 \times 10^{-5}$
$\tau_{ut}{}^*$ and $\tau_{ub}{}^*$	$\tau_{ut}\,H_t/k_{so}\,B^2$; $\tau_{ub}\,H_b/k_{so}\,B^2$	$6 \times 10^{-7} - 1.5 \times 10^{-6}$

and the bottom shear layer have been assumed to be same. However, the analysis procedure is general enough to take care of unequal column loads on the footing, different ultimate resistance in shear and shear modulus of top and the bottom granular fill layer.

4. Results and discussion

First of all, an attempt has been made to validate the proposed model by comparing the results from degenerated case of present study with those already available in the literature. This was then followed by a detailed parametric study to quantify the influence of geosynthetic layer on the deflection of footing. Due to the symmetrical boundary and loading conditions, results are valid only for symmetrical loading on combined footings. However, the proposed approach is general enough to take care of any other loading and boundary conditions. It is not possible to present all the results; however, some typical results have been presented in this section.

4.1 Validation

In the absence of granular fill layer and stone columns, a linear model of soil - foundation system under consideration reduces to a Winkler model. A study has been conducted for shear modulus of granular fill layer equal to zero and relative stiffness of stone columns with surrounding soft soil (α) as one in the linear model of soil - foundation system. The results from this study have been found to identical with those obtained from the closed form solution of the Winkler model which validated the proposed model and developed computer program.

Further, the proposed model has also been validated by comparing the results from Deb *et al.* (2007) to that from a degenerated case of present study. Deb *et al.* (2007) did not consider finite flexural rigidity of foundation and stone column was considered to be linear. In view of this, in the degenerated case of present study, flexural rigidity of footing has been considered to be zero and stone columns have been considered to be linear. Further, the expression for relative stiffness of stone columns with surrounding soil, α has been substituted as (Deb *et al.* 2007)

$$\alpha = \frac{(1+\nu_s)(1-2\nu_s)E_c}{(1+\nu_c)(1-2\nu_c)E_s} \tag{32}$$

where, (E_s, u_s) and (E_c, u_c) are the elastic modulus and Poisson's ratio of soil and the stone columns respectively.

The results from such a degenerated case of present study have been compared with those from

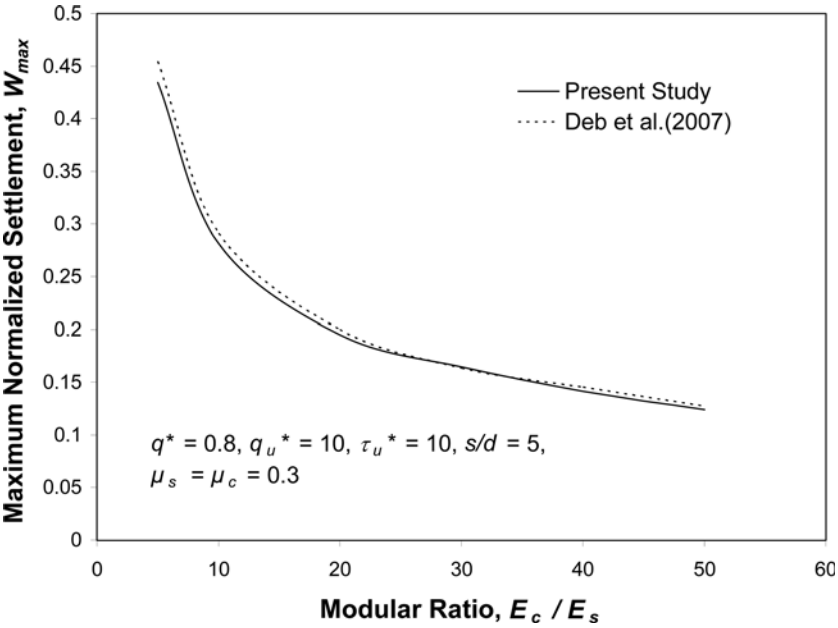

Fig. 4 Comparison of maximum settlement with Deb *et al.* (2007)

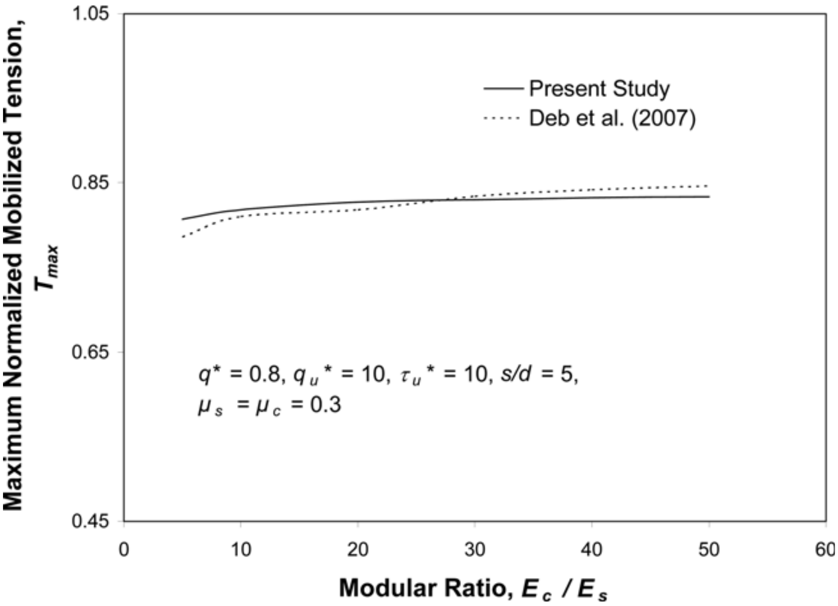

Fig. 5 Comparison of maximum mobilized tension with Deb *et al.* (2007)

Deb *et al.* (2007) and have been presented in Figs. 4 and 5 with respect to settlement and the mobilized tension respectively, for ratio of extent of considered spatial domain to width of loaded region equal to 2. A very good agreement can be observed from the figures as the difference in both the results is of the order of 2-3%.

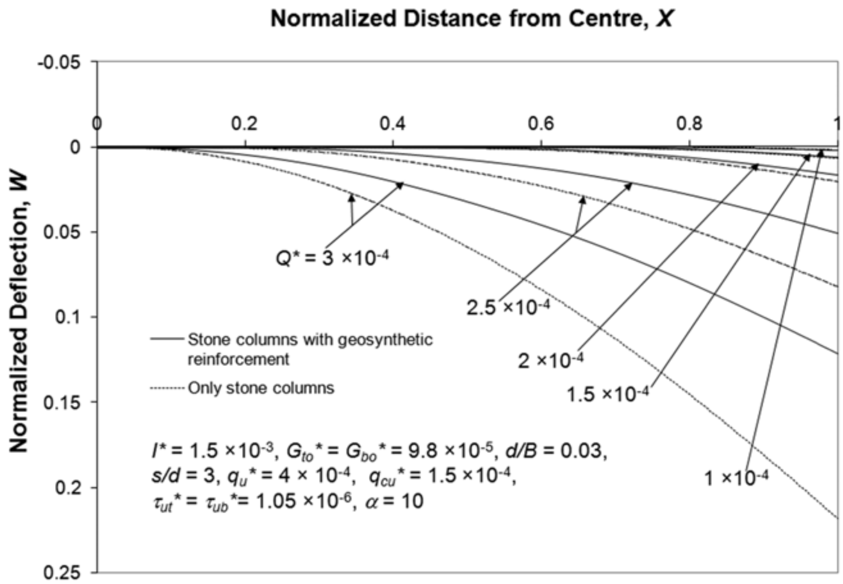

Fig. 6 Influence of geosynthetic layer for various values of applied loads on normalized deflection of footing

4.2 Influence of applied loads ($Q_1 = Q_2 = Q_3 = Q$)

Fig. 6 depicts the influence of geosynthetic layer on the deflection of footing for typical values of input parameters as: $I^* = 1.5 \times 10^{-3}$, $G_{to}^* = G_{bo}^* = 9.8 \times 10^{-5}$, $d/B = 0.03$, $s/d = 3$, $q_u^* = 4 \times 10^{-4}$, $q_{cu}^* = 1.5 \times 10^{-4}$, $t_{ut}^* = t_{ub}^* = 1.05 \times 10^{-6}$ and $\alpha = 10$. The nondimensional column loads have been varied from 1×10^{-4} to 3×10^{-4}. It can be observed that as the load increases, the reduction in the maximum deflection due to inclusion of geosynthetic layer also increases. This reduction has been found to be about 2.5%, 7.4%, 18.7%, 38.1% and 44.4% corresponding to normalized load, Q^* of 1×10^{-4}, 1.5×10^{-4}, 2×10^{-4}, 2.5×10^{-4} and 3×10^{-4} respectively.

4.3 Influence of flexural rigidity (EI) of footing

The deflection profiles of footing for different values of its flexural rigidity have been depicted in Fig. 7 for both cases, i.e., with and without geosynthetic layer for typical set of input parameters as mentioned in the figure. Geosynthetic layer has been found to be more effective in reducing the deflection for relatively flexible footings. The reduction in deflection has been found to reduce from about 37.7% to 1.9% as the normalized flexural rigidity increases from 5×10^{-5} to 2×10^{-3}.

4.4 Influence of diameter of stone columns (d)

The influence of diameter of stone columns on the deflection of footing with geosynthetic layer has been depicted in Fig. 8 for input parameters as $I^* = 5 \times 10^{-4}$, $G_{to}^* = G_{bo}^* = 9.8 \times 10^{-5}$, $Q^* \times 10^{-4}$, $s/d = 2.5$, $q_u^* = 4 \times 10^{-4}$, $q_{cu}^* = 6 \times 10^{-5}$, $t_{ut}^* = t_{ub}^* = 1.05 \times 10^{-6}$ and $\alpha = 25$. The parametric study was also conducted for the case when geosynthetic layer is absent and a reduction of about 25-30% in the maximum deflection of footing was observed for different normalized diameters with the

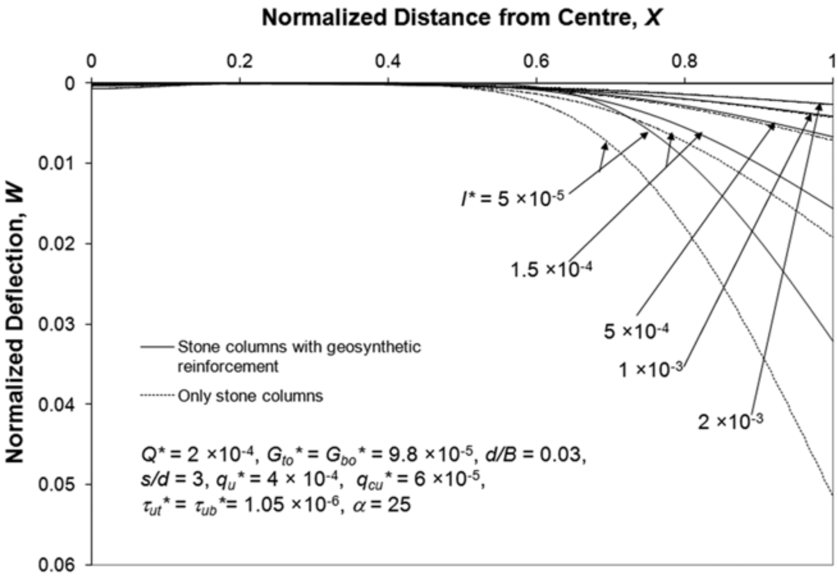

Fig. 7 Influence of geosynthetic layer for various values of flexural rigidity on normalized deflection of footing

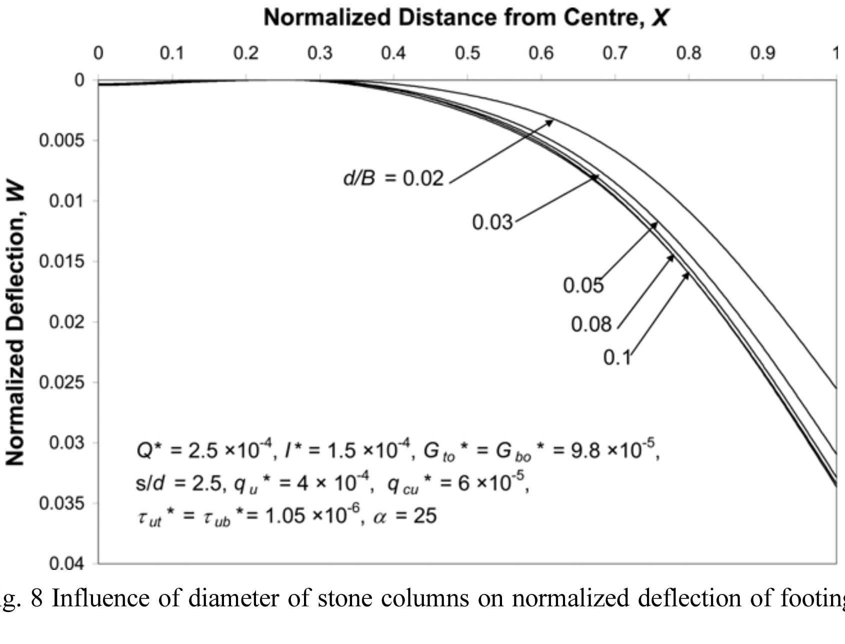

Fig. 8 Influence of diameter of stone columns on normalized deflection of footing

inclusion of geosynthetic layer and therefore these results have not been presented here. Fig. 8 shows that for a constant value of spacing to diameter ratio, deflection of footing increases as the diameter of stone columns increases except for d/L = 0.03 and 0.05. The deflection has been found to be independent of any variation in diameter of stone columns for d/L > 0.05. As the diameter increases, the number of stone columns beneath the footing reduces which causes an increase in the deflection. However, increase in stone column diameter also results into replacement of larger portion of soft soil by granular material thereby reducing the settlement. The influence of reduction

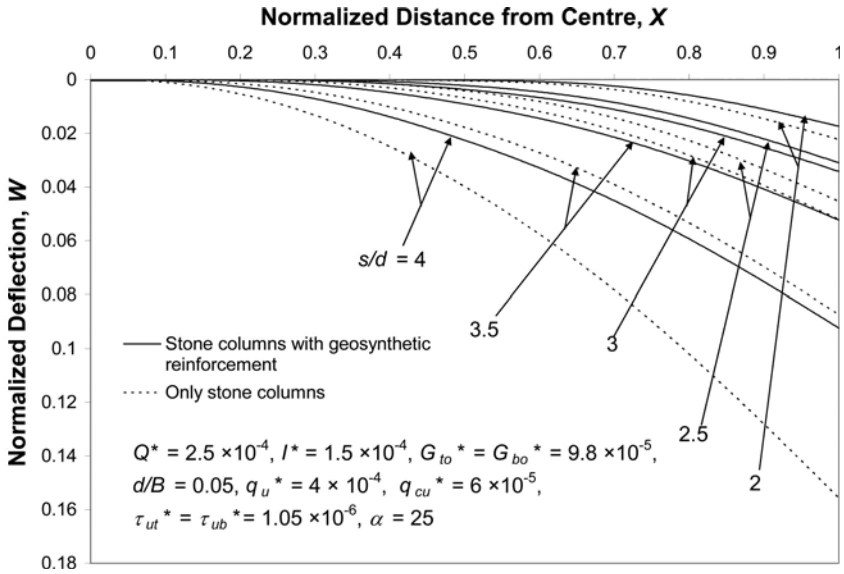

Fig. 9 Influence of geosynthetic layer for various values of spacing to diameter ratio on normalized deflection of footing

in number of stone columns has been found to be dominating as compared to the influence due to replacement of soft soil by better granular material for all d/L values considered in the study expect for d/L = 0.03 and 0.05. For d/L = 0.03 and 0.05, the effect of replacement of larger portion of soft soil by granular material has been observed to be more pronounced. This explains the influence of diameter of stone columns on the response of footing.

4.5 Influence of spacing to diameter ratio (s/d)

The influence of geosynthetic layer on settlement of footing for different values of s/d ratio has been presented in Fig. 9 for typical values of input parameters as mentioned in the figure. Stone columns have been placed symmetrically on both sides of footing. It has been observed that as ratio s/d reduces, the reduction in deflection of footing due to inclusion of geosynthetic layer also reduces. This decrease has been found to be reducing from 40.7% to 22.3% for ratio, s/d reducing from 4 to 2. This implies that for larger s/d ratio, inclusion of geosynthetic layer is more effective in reducing the deflection.

4.6 Influence of ultimate bearing capacity of soft foundation soil (q_u)

Fig. 10 depicts the influence of ultimate bearing capacity of natural occurring poor foundation soil, q_u on deflection of the footing. The normalized input parameters considered for this study are: $I^* = 1.5 \times 10^{-3}$, $G_{to}^* = G_{bo}^* = 9.8 \times 10^{-5}$, $Q^* = 2 \times 10^{-4}$, $s/d = 3$, $d/L = 0.05$, $q_{cu}^* = 6 \times 10^{-5}$, $t_{ut}^* = t_{ub}^* = 1.05 \times 10^{-6}$ and $\alpha = 25$. The inclusion of geosynthetic layer has been found to be more effective for low values of the parameter, q_u^* and the reduction in maximum deflection of footing due to inclusion of geosynthetic layer can be to the extent of 25% for the range of parameters considered in the analysis.

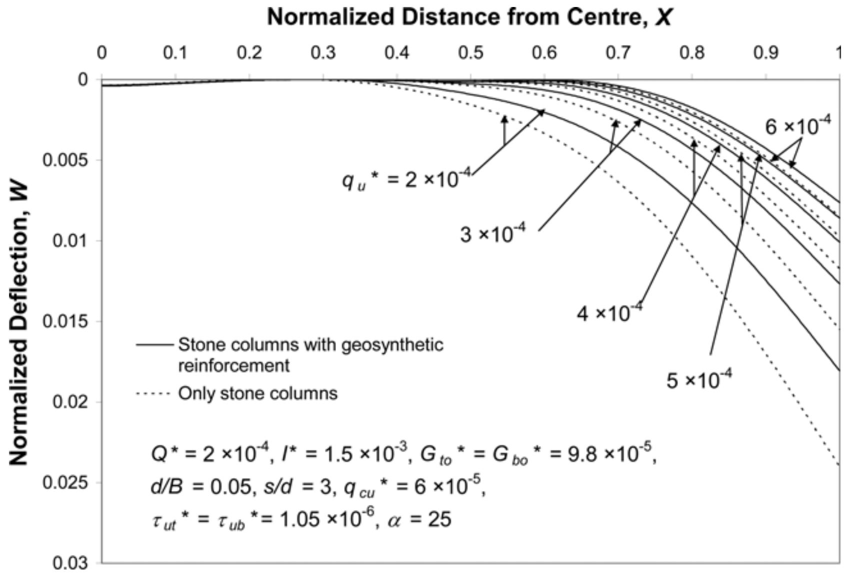

Fig. 10 Influence of geosynthetic layer for various values of ultimate bearing capacity of foundation soil on normalized deflection of footing

4.7 Influence of ultimate bearing capacity of stone columns (q_{cu})

The effect of inclusion of geosynthetic layer on normalized deflection of footing for different values of ultimate bearing capacity of stone columns, q_{cu} has been presented in Fig. 11. Geosynthetic layer has been found to be more effective in reducing the deflection of footing for smaller values of the

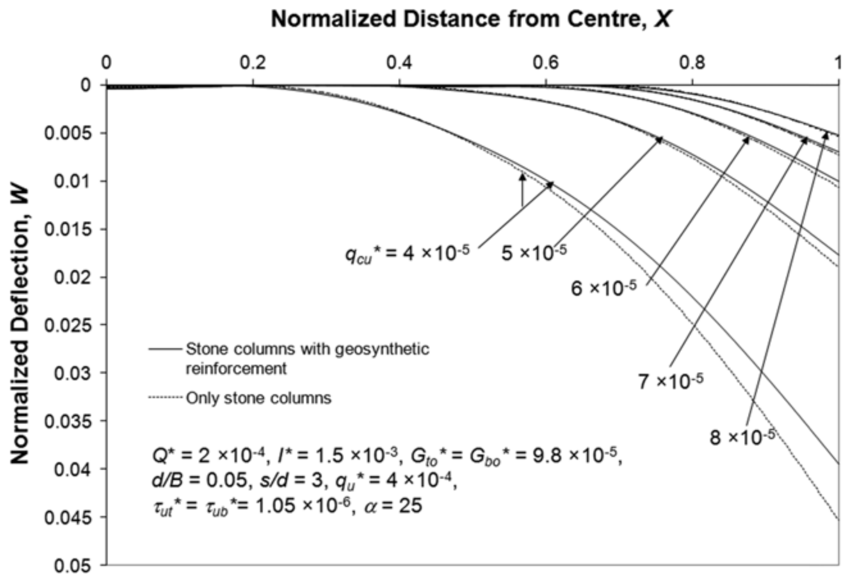

Fig. 11 Influence of geosynthetic layer for various values of ultimate bearing capacity of stone columns on normalized deflection of footing

parameter, q_{cu}. The maximum reduction in maximum normalized deflection has been observed as 12.9% for $q_{cu}^* = 6 \times 10^{-5}$.

5. Conclusions

A method has been proposed for the analysis of combined footing resting on geosynthetic reinforced-granular fill stone columns improved poor soil. The results from degenerated case of present study have been found to be in good agreement with those from previous study. Detailed parametric study has been carried out to study the influence of inclusion of geosynthetic layer.

Geosynthetic layer has been found to be quite effective in reducing the deflection of combined footing resting on stone column improved earth beds. At higher values of applied loads and for highly flexible footings, the geosynthetic layer has been found to be more effective in reducing the deflection of footing. For the range of parameters considered, the maximum reduction can be to the extent of 40-45% for larger applied loads and lesser flexural rigidity of footing.

The reduction in the deflection of beam due to inclusion of geosynthetic layer has been found to be independent of any change in the diameter of stone columns and this has been observed between 25% to 30%. Keeping diameter of stone columns constant, inclusion of geosynthetic layer is more effective in reducing the deflection for larger s/d ratio.

It has been observed that for low values of ultimate bearing capacity of poor soil and the stone columns, geosynthetic layer is more effective in reducing the deflection.

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