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Ring Footings on Reinforced Sand

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Introduction

Soil reinforcement in crude forms has been used since ancient times. It was, however, in the late sixties that a systematic study on reinforced soil was proposed by Vidal (1966). Since then, several investigators have been involved in research works all over the world, to investigate the behaviour of reinforced soil structures, so that rational design method can be developed.

Many experimental studies to investigate the behaviour of footings on reinforced soil have been conducted by (Binquet and Lee, 1975a; Saran et al., 1978; Dembicki et al., 1986; Verma and Char, 1986; Singh, 1988 ; Huang and Tatsuoka, 1990; Mandal and Manjunath, 1990; Dembicki and Jermolowicz, 1991; King et al., 1993; Omar et al., 1993b; Murthy et al., 1993; Al-Ashou et al., 1994; Youssef, 1995 and Kumar, 1997) on strip footings; (Milovic, 1977; Akinmusuru and Akinbolade, 1981; Singh, 1984; Guido et al., 1985; Shaw, 1985; Guido et al., 1986; Singh, 1988; Sreekantiah, 1990; Shankariah, 1991; Omar et al., 1993b; Al-Ashou et al., 1994 and Kumar, 1997) on square footings; (Fragaszy and Lawton, 1984; Sreekantiah, 1987; Singh, 1988; Sridharan et al., 1988; Ayyar et al., 1990; Omar et al., 1993a and Yetimoglu et al., 1994) on rectangular footings; (Milovic, 1977; Singh, 1988 and Sridharan et al., 1988) on circular footings whereas (Haroon et al., 1990 and Galav, 1997) on ring footings.

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Analytical solutions, however have been proposed by (Binquet and Lee, 1975b; Brown and Poulos, 1981; Huang and Tatsuoka, 1990; Karmarkar and Kumar, 1990; Desai and Vimawala, 1991; Dixit and Mandal, 1993; Murthy et al., 1993; Ghosh and Bhasin, 1995; Sitharam and Raghavendra, 1995; Youssef, 1995; Raghvendra et al., 1996; Saran and Agnihotri, 1996; Kumar, 1997; and Dey, 2002) for strip footings; (Singh, 1988; Yetimoghu et al., 1994 and Kumar, 1997) for square or rectangular footings. All above analyses have been developed for footings subjected to central vertical load resting on reinforced sand.

It is evident from the above that no work is available on ring footings subjected to eccentric-inclined load resting on reinforced sand. Ring footings are very common type of foundations for tall structures like smoke stacks, cement-silos, water towers etc. Such foundations are usually subjected to eccentric-inclined load as horizontal load due to wind act on the structure in addition to the vertical load. In this paper this problem has been studied both analytically and experimentally.

The analysis is based on the approach proposed by Binquet and Lee (1975b) by modifying some of the assumptions made by them to more realistic ones. The analysis, however, requires, as a prerequisite, the pressure-settlement curve of the same footing on unreinforced sand, which can be obtained by the procedure given by Al-Smadi (1998).

Analysis

The analysis has been developed to predict the pressure ratio (p_r) which is defined as below:

$$p_r = \frac{q}{q_o} \tag{1}$$

where q_o = Average contact pressure of the ring footing resting on unreinforced sand at settlement S

q = Average contact pressure of the same ring footing resting on reinforced sand at the same settlement S

Knowing the values of p_r and q_o, one can obtain the value of q. As mentioned above, the value of q_o can conveniently obtained from the pressure settlement curve of the ring footing on unreinforced sand. The procedure developed by Al-Smadi (1998), which is based on the use of non-linear constitutive law of soil may be used for getting the pressure-settlement characteristics of ring footings on un-reinforced sand. The details of this methodology are given elsewhere (Al-Smadi, 1998; Saran and Al-Smadi, 2001).

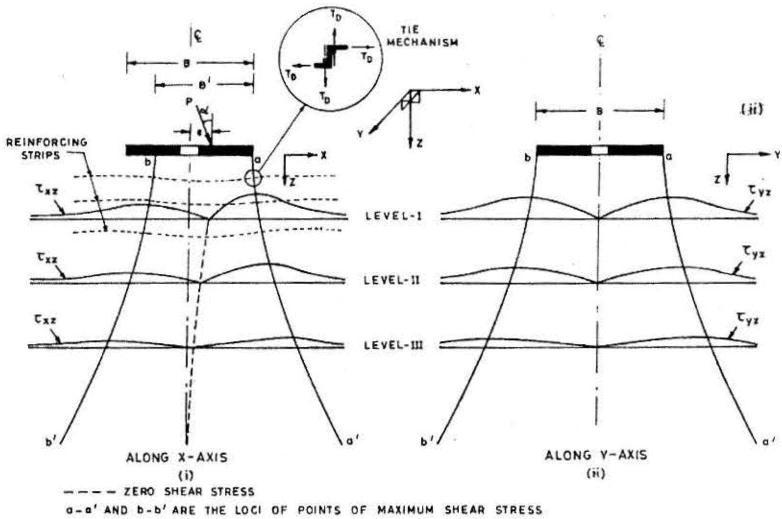

FIGURE 1 : Distribution of Shear Stresses beneath Ring Footings subjected to Eccentric-Inclined Load

The present analysis is an extension of the work of Binquet and Lee (1975b) to ring footing subjected to eccentric-inclined load, and is based on the following assumptions:

1) The boundary between the downward moving and the outward moving zones has been assumed as the locus of points of maximum shear stress at every depth z (Fig.1).

2) At the surface separating the downward moving and outward moving zones, the ties have been assumed to undergo two right angle bends around two frictionless rollers. At the slip surface, the tie resistance is a vertically acting tensile force (Fig.1).

3) The tie-soil friction coefficient (f_c) has been assumed to vary with depth in accordance with following relationship (Murthy et al., 1993).

$$f_c = mf \tag{2}$$

where f = Coefficient of friction between soil and reinforcement

m = Mobilization factor given by:

$$(1 - z/B)0.7 + 0.3 \quad \text{for } z/B \leq 1.0 \tag{3a}$$

$$(2 - z/B)0.3 \quad \text{for } z/B > 1.0 \tag{3b}$$

4) The developed tie force among the number of reinforcing layers, N has been assumed to vary in proportion of $m_1 : m_2 : m_3 : \ldots : m_N$, such that $m_1 + m_2 + m_3 + \ldots + m_N = 1$. The failure mechanism has been assumed for various combinations of the tie pull-put and tie breakage at different levels.

5) Forces evaluated throughout the analysis are obtained for the same ring footing, subjected to the same loading system, at a given settlement, for footing on reinforced and unreinforced soil.

6) Normal and shear stresses have been obtained using equations of the theory of elasticity.

Evaluation of Developed Tie Force, T_D

Shear stress distribution at various depths z in the x and y-directions is shown in Figs.1(i) and (ii). The shear stress distribution has been obtained considering both non-uniform normal and tangential loading at the interface between the footing and the soil (Al-Smadi, 1998). Let X_0, X_1 and X_2 be the horizontal positions, along x-axis, of zero shear stress, maximum shear stress to the side of the eccentricity and the maximum shear stress to the opposite side of the eccentricity, respectively, as shown in Fig.2(i). Also, let Y_0 define the horizontal position, along the y-axis, of the maximum shear stress (t_{yz}), as shown in Fig.2(ii). However, the distribution of the stress along the y-axis is symmetrical about the x-axis. The extent of X_0, X_1 and X_2 varies with the change in the eccentricity and inclination of the load and the depth beneath the footing.

The loci of points of maximum shear stress, which are equally the planes separating the downward and outward moving zones, at every depth z in the x and y-directions are shown in Figs.2(i) and (ii).

Consider an element ABCD at depth z, as shown in Fig.2(i) in the x-direction, which is the volume of soil inside the downward moving zone lying between two vertically adjacent layers of reinforcement. The forces acting on this element are shown in Fig.2(i) for the cases of unreinforced and reinforced foundation soil. The developed tie force will be in both the x and y-directions, since the reinforcement is provided in these directions. Considering the equilibrium of all such elements along the whole length of the reinforcement at depth, z and integrating, we get:

$$T_{Dx} = \left[J_{xz}A - I_{xz}B\Delta H\right]q_o\left(p_r - 1\right) \tag{4}$$

where T_{Dx} = Force in the x-direction for one layer of reinforcement provided at depth, Z.

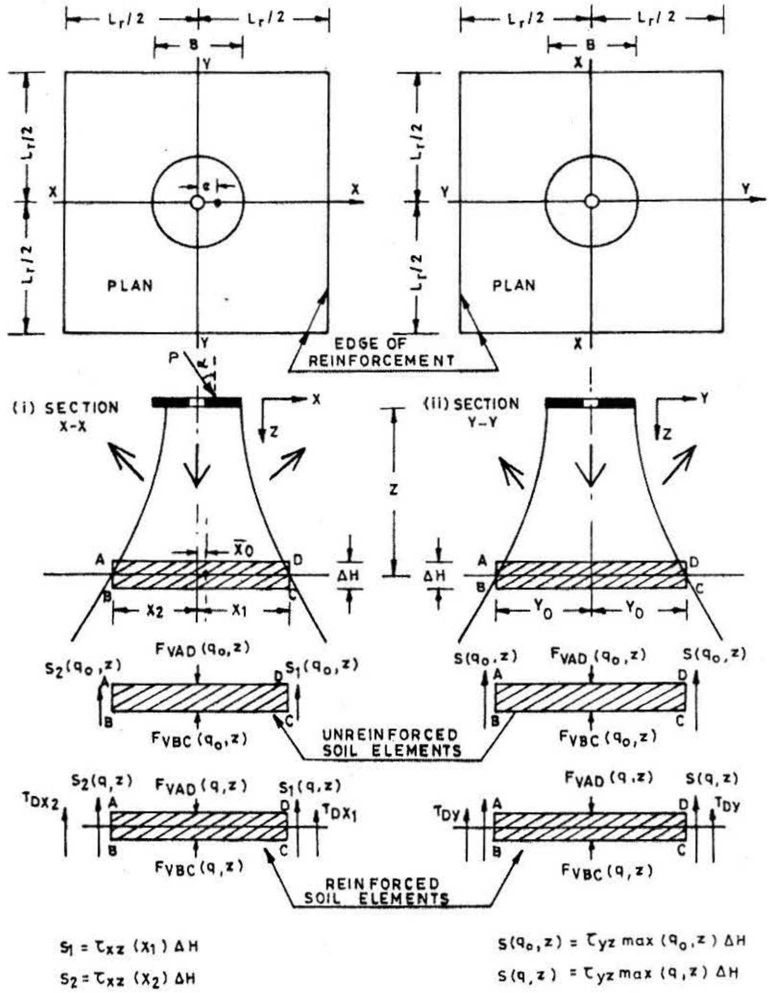

FIGURE 2 : Assumed Separating Planes and Components of Forces for Pressure Ratio Calculations for Ring Footings subjected to Eccentric-Inclined Load on Reinforced Sand

$$T_{Dx} = T_{Dx1} + T_{Dx2} \tag{5}$$

$$I_{xz} = I_{xz1} + I_{xz2} \tag{6}$$

$$I_{xz1} = \frac{1}{qB}\int_{-L_r/2}^{+L_r/2} \tau_{xz}\left(\frac{x_1}{B}, \frac{z}{B}\right) dy \tag{7}$$

$$I_{xz2} = \frac{1}{qB}\int_{-L_r/2}^{+L_r/2} \tau_{xz}\left(\frac{x_2}{B}, \frac{z}{B}\right) dy \tag{8}$$

$$J_{xz} = \frac{1}{qA} \int_{Y=-L_r/2}^{Y=+L_r/2} \left(\int_{x=-X_2}^{x=+X_1} \sigma_z dx \right) dy \tag{9}$$

X_1 and X_2 = distances of maximum shear stresses, as shown in Fig.2.

L_r = Length of reinforcing layer

B = Size of footing (diameter)

ΔH = Thickness of element

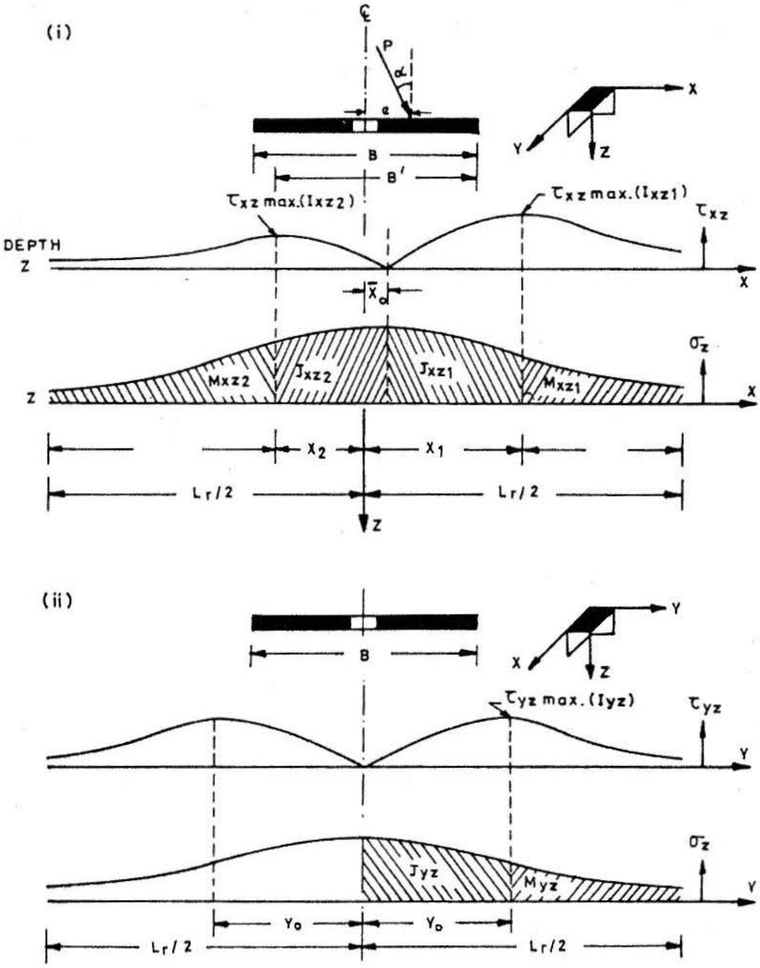

FIGURE 3 : Shear and Normal Stresses and Non-Dimensional Factors (J_s', I_s' and M_s') for (i) X-Direction and (ii) Y-Direction

A = Area of solid circular footing (i.e. n = 0.0)

τ_{xz} = shear stresses in the x-directions as shown in Fig.3(i).

σ_z = normal stresses in the x-directions as shown in Fig.3(i).

Similarly the developed tie force in the y-direction is given by

$$T_{Dy} = \left[J_{yz}A - I_{yz}B\Delta H\right]q_o\left(p_r - 1\right) \tag{10}$$

In which,

$$I_{yz} = \frac{2}{qB}\int_{X=-L_r/2}^{X=+L_r/2} \tau_{yz}\left(\frac{Y_0}{B}, \frac{z}{B}\right)dx \tag{11}$$

$$J_{yz} = \frac{1}{qA}\int_{X=-L_r/2}^{X=+L_r/2}\left(\int_{Y=-Y_0}^{Y=+Y_0} \sigma_z dy\right)dx \tag{12}$$

where Y_0 = Distance of maximum shear stress in y-direction (Fig.2)

τ_{yz} = shear stresses in the y-directions as shown in Fig.3(ii)

σ_z = normal stresses in the y-directions as shown in Fig.3(ii)

The total tie force, T_D is given by

$$T_D = T_{Dx} + T_{Dy} \tag{13}$$

$$= \left[\left(J_{xz} + J_{yz}\right)A - \left(I_{xz} + I_{yz}\right)B\Delta H\right]q_o\left(p_r - 1\right) \tag{14}$$

Special case

When the footing is subjected to symmetrical loading, i.e. α = 0° and e/B = 0.0,

$X_0 = 0$

$X_1 - X_2 = X_0$ and

$Y_0 = X_0$

$$J_{xz} = J_{yz} = \frac{1}{qA}\int_{-L_r/2}^{+L_r/2}\left(\int_{-x_0}^{+x_0}\sigma_z dx\right)dy \tag{15}$$

$$I_{xz} = I_{yz} = \frac{2}{qB}\int_{-L_r/2}^{+L_r/2}\tau_{xz\,max}\,dy \tag{16}$$

$$T_D = 2\,T_{Dx}, \tag{17}$$

where $T_{Dx} = T_{Dy}$

Numerical integration has been used to solve the above equations.

Evaluation of Tie Pullout Frictional Resistance, T_f

The tie pullout frictional resistance will be due to the total normal force on the plan area of the reinforcement lying outside the assumed boundary separating the downward and outward moving zones, as shown in Fig.2. The normal force, however, consists of two components, one is due to the applied bearing pressure and the other is due to the normal overburden pressure of the soil. Considering both components over the whole area of reinforcement outside the separating plane, the tie pullout frictional resistance in the x-direction at depth, z, for a footing at depth D_f, is as follows:

$$T_{fX} = 2f_c LDR\left[M_{xz}Aq_o p_r + \gamma A_{xz}\left(z+D_f\right)A\right] \tag{18}$$

where
$$T_{fx} = T_{fx1} + T_{fx2} \tag{19}$$

$$M_{xz} = M_{xz1} + M_{xz2} \tag{20}$$

$$M_{xz1} = \frac{1}{qA}\int_{-L_r/2}^{+L_r/2}\left(\int_{+x_1}^{-L_r/2}\sigma_z dx\right)dy \tag{21}$$

$$M_{xz2} = \frac{1}{qA}\int_{-L_r/2}^{+L_r/2}\left(\int_{-x_1}^{-L_r/2}\sigma_z dx\right)dy \tag{22}$$

$$A_{xz} = A_{xz1} + A_{xz2} \tag{23}$$

$$A_{xz1} = \frac{1}{A}\int_{-L_r/2}^{+L_r/2}\left(\frac{L_r}{2} - x_1\right)dy \tag{24}$$

$$A_{xz2} = \frac{1}{A}\int_{-L_r/2}^{+L_r/2}\left(\frac{L_r}{2} - x_2\right)dy \tag{25}$$

in which γ = dry unit weight of soil;

A = area of solid circular footing ($n = 0.0$) and

D_f = embedment depth of footing.

$$LDR = \frac{\text{Plan Area of Reimbursement}}{\text{Total Area of Reinforced Soil Layers}}$$

$= 1$ for Geogrid reinforcement

f_c is as given by Eqn.2.

Similarly, the tie pullout frictional resistance in y-direction is expressed as follows

$$T_{fY} = 2f_c LDR\left[M_{yz}Aq_o p_r + \gamma A_{yz}\left(z + D_f\right)A\right] \tag{26}$$

where
$$M_{yz} = M_{yz1} + M_{yz2} \tag{27}$$

but
$$M_{yz1} = M_{yz2} = \frac{1}{qA}\int_{-L_r/2}^{+L_r/2}\left(\int_{+x_1}^{-L_r/2}\sigma_z dy\right)dx \tag{28}$$

$$A_{yz} = A_{yz1} + A_{yz2} \tag{29}$$

but
$$A_{yz1} = A_{yz2} = \frac{1}{A}\int_{-L_r/2}^{+L_r/2}\left(\frac{L_r}{2} - Y_0\right)dx \tag{30}$$

Therefore the total tie pullout frictional resistance is given by

$$T_f = T_{fx} + T_{fy} \tag{31}$$

Numerical integration has been used to solve the above equations.

For the special case of ring footing subjected to symmetrical loading i.e. $\alpha = 0°$ and $e/B = 0.0$, where;

$X_1 = X_2 = X_0$ and $X_0 = Y_0$

$$M_{xz1} = M_{xz2} = M_{yz1} = M_{yz2} = \frac{1}{qA} \int_{-L_r/2}^{+L_r/2} \left(\int_{+x_0}^{-L_r/2} \sigma_z dx \right) dy \tag{32}$$

$$A_{xz1} = A_{xz2} = A_{yz1} = A_{yz2} = \frac{1}{A} \int_{-L_r/2}^{+L_r/2} \left(\frac{L_r}{2} - x_0 \right) dy \tag{33}$$

$$M_{xz} = M_{yz} \tag{34}$$

$$A_{xz} = A_{yz} \tag{35}$$

$$T_f = T_{fx} = T_{fy} = 2T_{fx} \tag{36}$$

Non-dimensional charts for J_{xz}, J_{yz}, I_{xz}, I_{yz}, M_{xz}, M_{yz}, A_{xz} and A_{yz} at different layers levels, z/B = 0.25, 0.5, 0.75, 1.0, 1.25, 1.5, 1.75 and 2, for

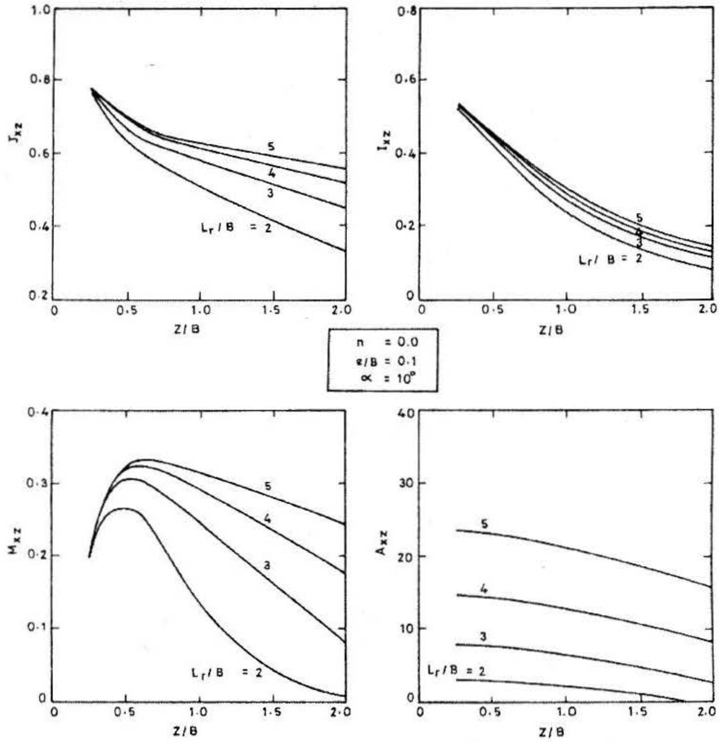

FIGURE 4 : Non-Dimensional Area and Force Components for Computing Pressure Ratio for Ring Footing (n = 0) on Reinforced Soil for e/B = 0.1 and α = 10° in X-Direction

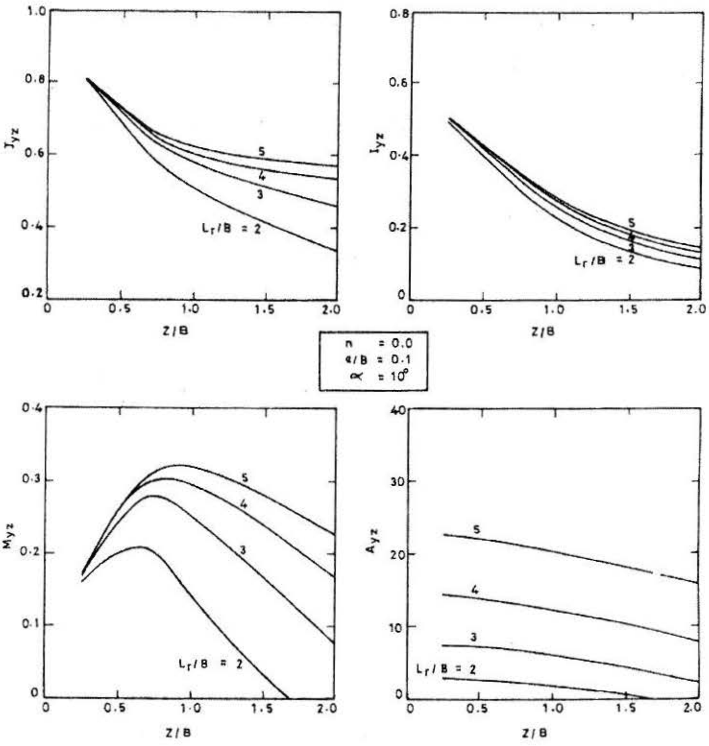

FIGURE 5 : Non-Dimensional Area and Force Components for Computing Pressure Ratio for Ring Footing (n = 0) on Reinforced Soil for e/B = 0.1 and α = 10° in Y-Direction

ring footings subjected to eccentric-inclined load have been prepared for various combinations of n, e/B ratio, α and L_r/B. The values of n e/B, α and L_r/B were taken as below:

$$n = 0,\ 0.2,\ 0.4,\ 0.6$$

$$e/B = 0,\ 0.1,\ 0.2$$

$$\alpha = 0°,\ 10°,\ 20°$$

$$L_r/B = 2,\ 3,\ 4,\ 5$$

The non-dimensional charts for all above-mentioned parameters are given elsewhere (Al-Smadi, 1998). Here few typical non-dimensional charts are presented in Figs.4 to 12 in terms of annular ratio 'n', eccentricity ratio 'e/B', load inclination 'α' and the size of reinforcing layers 'L_r/B'.

The pressure ratio p_r has been computed by applying the following conditions;

(1) The developed tie force in any layer should not exceed the tie pullout frictional resistance, in the same layer

i.e. $m_i\, T_{Di} < T_{fi}$ (37)

where $i = 1, 2, \ldots, N$

(2) The developed tie force, in any layer, should not exceed the tie breaking strength of the same layer

i.e. $m_i\, T_{Di} < T_{RF}$ (38)

where T_{RF} = the total breaking force in that layer

i.e. $$T_{RF} = T_R \times \left[\begin{array}{c}\text{Length of reinforcement along} \\ \text{which breakage may take place}\end{array}\right] \quad (39)$$

where T_R = Allowable tensile strength of reinforcement per unit length

m_i's = Distribution factors assumed for the distribution of the tie force in N-layers, such that $m_1 + m_2 + \ldots + m_N = 1$.

The check is applied for different combinations of tie pullout and tie breaking failures. The minimum value shall be the critical p_r.

Model Tests

A total of ninety eight model tests were conducted on model ring footings of size (external diameter) B = 200 mm, resting on unreinforced and reinforced Amanatgarh sand (SP, D_{10} = 0.15 mm, C_u = 2.0, C_c = 1.39) at a relative density of 70%. Details of the test programme are as follows; Sixty three tests on solid circular footings subjected to eccentric-inclined load having eccentricity ratio e/B = 0.0, 0.1, 0.2 and load inclinations, $\alpha = 0°$, 10°, 20°. Fourteen tests on ring footings (n = 0.4, 0.6) subjected to central vertical load and twenty one tests on ring footings (n = 0.4) subjected to eccentric-inclined load, i.e. e/B = 0.3 and α = 0°, 10°, 20°. Tensar SS20 Geogrid, square in shape were used throughout the study. The length of the reinforcing layer (L_r) was kept as 2B and 3B, while the number of the reinforcing layers (N) was kept as two, three and four. In all tests the footings were placed at the surface of the sand. The upper most layer of reinforcement was placed at a distance of 0.25B from the footing base, while the vertical spacing between adjacent reinforcing layers was kept as 0.25B. The positions of kinks or depression of reinforcing layers were inspected visually after the completion of each test.

The parameters 'a' and 'b' of the hyperbola were established by conducting triaxial tests at different confining pressures on sand (D_r = 70%), as follows:

$$1/a = K_1(\sigma_3)^2 \tag{40}$$

$$1/b = K_2 + K_3\sigma_3 \tag{41}$$

in which $K_1 = 3.8 \times 10^3$, n = 0.58, $K_2 = 90$, $K_3 = 3.35$ and σ_3 = confining pressure in (kN/m^2).

Interpretation

Justification of Assumptions

The boundary between the downward moving and the outward moving zones has been assumed as the locus of points of maximum shear stress at every depth. The location of the separating plane can only be inferred from the location of the broken ties, if any, and the deformation pattern of the soil and reinforcing layers after failure. Binquet and Lee (1975) and Kumar (1997) observed that the breaking or deformation of ties approximately coincided with the assumed separating plane. The authors, in their experimental study, also observed that, the ties failed at locations very close to the assumed separating plane.

Formation of the kinks in the ties may occur due to the fact that, as the central zone of the soil moves downward with respect to the outer zone along the slip surface, it drags the ties along with it. However, the extent of the relative vertical movement diminishes at higher depths; consequently the formation of the kink in the ties becomes less likely. Due to economic consideration coupled with construction difficulties, the reinforcing layers are usually provided within a depth equal to, or less than, the footing width. This fact is also supported by results of all model tests conducted by many investigators. Therefore, it is reasonable to assume that, the ties, at the slip surface, undergo two right angle bends around two frictionless rollers, within this depth. Furthermore, from basic mechanics, the tensile force in the ties can be considered equally effective to act in the vertical direction, at the slip surface, elsewhere, on either side, the tie resistance acts horizontally along the axis of the tie.

The mobilization of the angle of interfacial friction is dependent on the relative movement between soil and reinforcement. The relative movement varies with depth, being maximum at the first layer. It has been assumed that the vertical settlement at any layer level is proportional to the vertical stress at that level. Murthy et al. (1993) showed about 30% of the surface settlement at depth of B and negligible settlement at depth of 2B, as shown in Fig.6. Hence, assumption 3 seems to be realistic.

It is reasonable to distribute the developed tie force among the number of reinforcing layers N, to vary in proportion of $m_1 : m_2 : m_3 : \ldots : m_N$, such that $m_1 + m_2 + m_3 + \ldots + m_N = 1$.

The total load carried by the footing resting on a reinforced soil bed will have two components such that; load transferred through the soil mass directly and load transferred through the reinforcing layers. The component of the load transferred directly through the soil alone causes settlement of the footing, while, the component of the load transferred through the reinforcing layers will have insignificant contribution to the settlement of the footing. It is, therefore, assumed that for the same footing size and shape, subjected to the same loading system, the settlement is the same for both

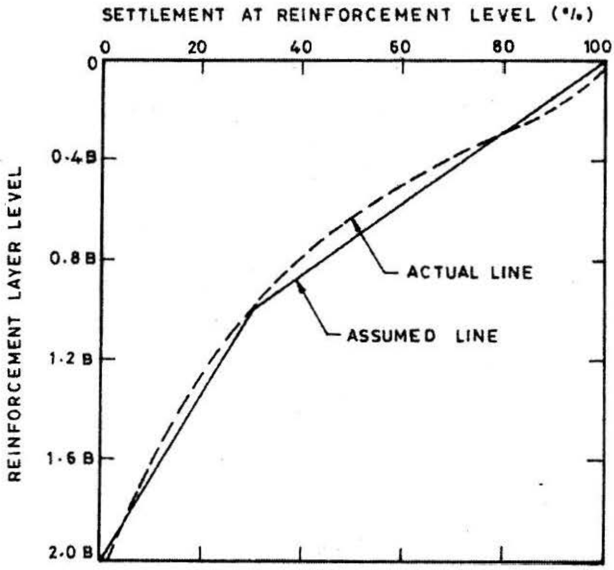

FIGURE 6 : Effective Settlement at Different Reinforced Levels (After Murthy et al., 1993)

unreinforced and reinforced soil beds; however, the additional load is resisted by the reinforcing layers.

Stress equations based on the theory of elasticity have been used to compute stresses in the soil mass, throughout the proposed analysis. Reinforced sand bed, however, is an anisotropic and non-homogeneous medium. Stress equations for such composite material are not available. Therefore, stress equations based on the theory of elasticity (Boussinesq, 1885 and Cerruti, 1888) have been used. It will be seen later that the results are not affected by this assumption.

Experimental Verification

The model test results have been utilised to validate the proposed analysis. In the analysis presented herein, the pressure ratio ($p_r = q/q_o$), can be obtained for known values of settlement at which pressure intensities 'q_o' are considered. Thus the pressure-settlement curves of the model footings on un-reinforced sand have been used to obtain the pressure-settlement of the same footing resting on reinforced sand. Comparison between results obtained from model tests and those from the proposed analysis are shown in Figs.7 to 8. These figures show clearly that the predicted results are in good agreement with the observed results.

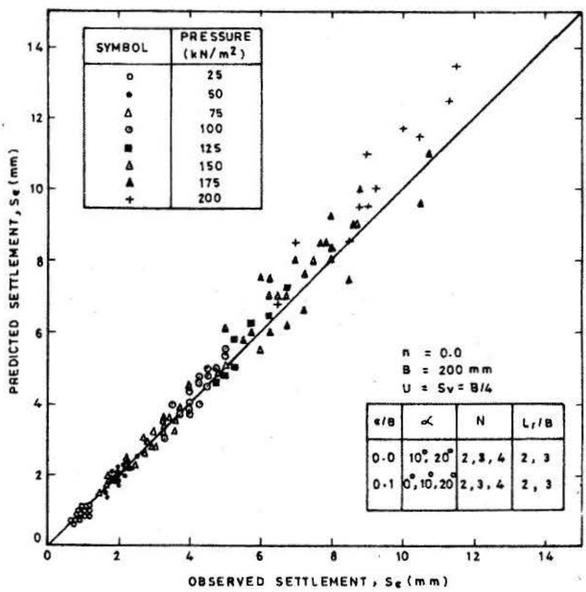

FIGURE 7 : Comparison of Observed and Predicted Values of Settlement of Ring Footing (n = 0) subjected to Eccentric-Inclined Load (e/B = 0 and 0.1, α = 0°, 10° and 20°) on Reinforced Sand

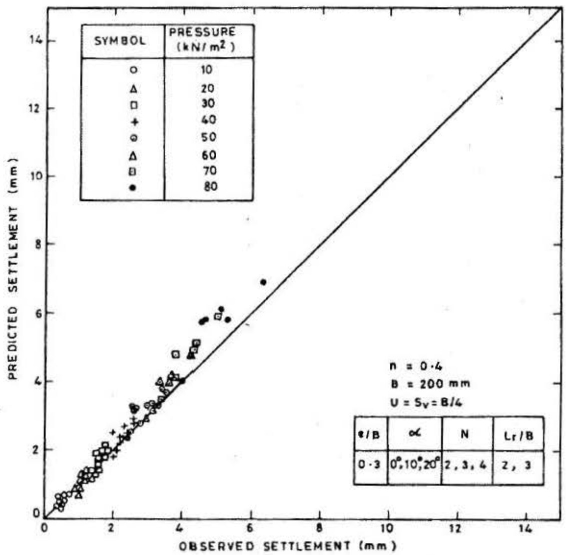

FIGURE 8 : Comparison of Observed and Predicted Values of Settlement of Ring Footing (n = 0) subjected to Eccentric-Inclined Load (e/B = 0.2, α = 0°, 10° and 20°) on Reinforced Sand

Illustrative Example

Statement of Problem

A circular footing of diameter (B) 2000 mm rests on reinforced sand (N = 3; L_r = 3B; U/B = S_r/B = 0.25). Geogrids have been used as reinforcement with allowable tensile strength (T_R) of 10 kN/m. The footing is subjected to eccentric inclined load (e/B = 0.1, α = 10°). The coefficient of friction between soil and reinforcement (f_c) is 0.404. The pressure intensity on the footing (B = 2000 mm) resting on un-reinforced sand is 285 kN/m^2 for a settlement of 20 mm. Determine the pressure ratio (p_r) of the footing on reinforced sand for the settlement equal to 20 mm.

Solution

(i) As mentioned in the text, the charts (Figs. 4-12) have been developed considering U/B = S_r/B = 0.25. Using Figs.9 and 10, determine the values of J_{xz}, J_{yz}, I_{xz}, I_{yz}, M_{xz}, M_{yz}, A_{xz} and A_{yz} for (e/B = 0.1, α = 10°, n = 0). Also calculate f_e, T_D, T_f and T_{Rf} using Eqns.2, 13, 31 and 39 respectively. These values are given in Table 1.

(ii) For number of reinforcing layers (N) equal to 3, there will be eight independent cases for which the values of p_r are obtained (Table 2).

TABLE 1 : Calculation of Forces for Determining the Pressure Ratio, P_r

Z/B	0.25	0.50	0.75
$J_{xz}+J_{yz}$	1.580	1.375	1.240
$I_{xz}+I_{yz}$	1.035	0.865	0.680
$M_{xz}+M_{yz}$	0.375	0.550	0.565
$A_{xz}+A_{yz}$	15.50	15.25	14.00
f_e	0.333	0.263	0.192
T_D (kN)	$3.929q_o(p_r-1)$	$3.455q_o(p_r-1)$	$3.216q_o(p_r-1)$
T_f (kN)	$0.785q_op_r+778.34$	$0.909q_op_r+806.41$	$0.682q_op_r+675.57$
T_{RF} (kN)	120	120	120

TABLE 2 : Pressure Ratio (p_r) Values for All Classes

Case No.	p_r	Tie Failure Mechanisms
1	10.07	All ties fail in pull-out
2	1.360	All ties fail in tension
3	2.446	1st tie fails in pull-out, others fail in tension
4	2.802	2nd tie fails in pull-out, others fail in tension
5	2.506	3rd tie fails in pull-out, others fail in tension
6	4.956	1st & 2nd ties fail in pull-out, others fail in tension
7	5.103	2nd & 3rd ties fail in pull-out, others fail in tension
8	4.373	1st & 2nd ties fail in pull-out, others fail in tension

Cases 3 and 4 are detailed as below :

Case 3: First tie fails in pull-out, while the rest fail in breaking

$$m_1 \times \{3.929 \times 285 \times (p_r - 1)\} < 0.785 \times 285 \times p_r + 778.34$$

$$m_2 \times \{3.455 \times 285 \times (p_r - 1)\} < 120$$

$$m_3 \times \{3.216 \times 286 \times (p_r - 1)\} < 120$$

$$m_1 + m_2 + m_3 = 1$$

From which, $p_r = 2.446$

Case 4: Second tie fails in pull-out, while the others fail in breaking

$$m_1 \times \{3.929 \times 285 \times (p_r - 1)\} < 120$$

$$m_2 \times \{3.455 \times 285 \times (p_r - 1)\} < 0.909 \times 285 \times p_r + 806.41$$

$$m_3 \times \{3.216 \times 286 \times (p_r - 1)\} < 120$$

$$m_1 + m_2 + m_3 = 1$$

From which, $p_r = 2.802$

The minimum value of p_r (= 1.360, Table 2) is the critical value. Therefore,

$$p_r = q/q_o = 1.360$$

Hence $q = 1.360 \times 285 = 386$ kN/m^2

Therefore, the pressure intensity of 386 kN/m^2 on the circular footing (B = 2000 mm) resting on reinforced sand will give the settlement equal to 20 mm.

Conclusions

1. An analytical analysis is developed for ring footing subjected to eccentric inclined load and resting on reinforced sand to predict the pressure ratio p_r (= q/q_o), where q_o is the pressure on the ring footing resting on un-reinforced sand at a given settlement S, and q is the pressure on the same ring footing resting on reinforced sand at the same settlement S.

2. The results have been presented in the form of non-dimensional charts convenient for designing the ring footings. An illustrative example has been included to demonstrate the use of these charts.

3. The analytical results have been compared with the data of model tests. A reasonable good tally has been observed between the two.

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