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ANALYSIS OF SHORT RIGID CAISSONS WITH GRANULAR CORE FOR ALLUVIAL LOWLANDS

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ABSTRACT: A new composite foundation is being proposed for foundations in saturated loose alluvial deposits. A simplified theoretical approach using linear Winkler type responses, is presented in this paper for the analysis of the proposed foundation. Extensive parametric studies in terms of load sharing and settlement reduction as functions of various parameters, e.g. length to diameter ratio and inner to outer diameter ratio of the caisson, Poission's ratios of soil and granular core material, relative stiffnesses of core, shear and bearing stiffnesses, are presented to predict the response of the composite foundation.

Key words: Coastal lowlands, earthquake, liquefaction

INTRODUCTION

Saturated alluvial deposits exist all over the world, e.g. the flood plains of South and Southeast Asia, China, Japan, United States etc. (Miura et al. 1994). The subsoil profile is highly variable in these deposits (Madhav and Miura 1994). Reasonable bearing strata are often available only below 2.0 to 4.0 m from the natural ground level. But, due to high ground water table, conventional shallow foundations are difficult to construct due to flooding of the foundation pits. De-watering techniques are very expensive especially for small and medium projects. Deep foundations and dynamic methods of ground improvement are costly and often the necessary equipment is not available locally. Indigenously, concrete pipes or shells are pushed into the ground and a concrete raft laid on top. The concrete pipes provide stiffness and transfer the loads to layers at depth. "Caker Ayam foundation' (also known as chicken foot) used in marshy lowlands of Indonesia (Hadmodjo 1985) is a similar technique. Caissons (well foundation) with rigid bottom plug are extensively used in India as foundations for bridges (IRC 1981). Shallow open caissons were employed as cutoff walls of bridges (Namjoshi and Kulkarni 1992).

To overcome the above problem, a new composite foundation (Fig.1) is being proposed for saturated loose alluvial deposits. It consists of shallow pipes or well steinings (outer diameter of 1.0 to 1.5 m, thickness of 10-15 cm and length of 1.0 to 3.0 m) with granular core inside. The steining is sunk to the desired depth by conventional sinking techniques. Soil within the steining is removed and granular material filled in and compacted to enhance the stability and load carrying capacity of the foundation. The details of the same are given elsewhere (Jawaid and Madhav 2000). The analysis of the proposed foundation-soil interaction is carried out using a Winkler's type representations for the soil and the granular core material responses. A detailed parametric study is carried out to evaluate the relative influences of each parameter on the overall response of the composite foundation.

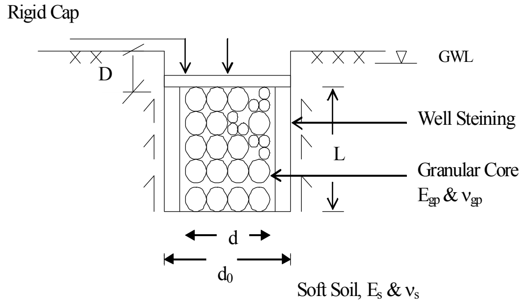

Fig. 1 The proposed composite foundation

PROBLEM STATEMENT AND ANALYSIS

A composite foundation of length, L, with granular core inside is considered. The outer and inner diameters of the steining are d_0 and d respectively. The moduli and Poisson's ratios of soil and granular core are E_s and ν_s and E_{gp} and ν_{gp} respectively. A vertical load, Q, is applied at the top of the proposed foundation.

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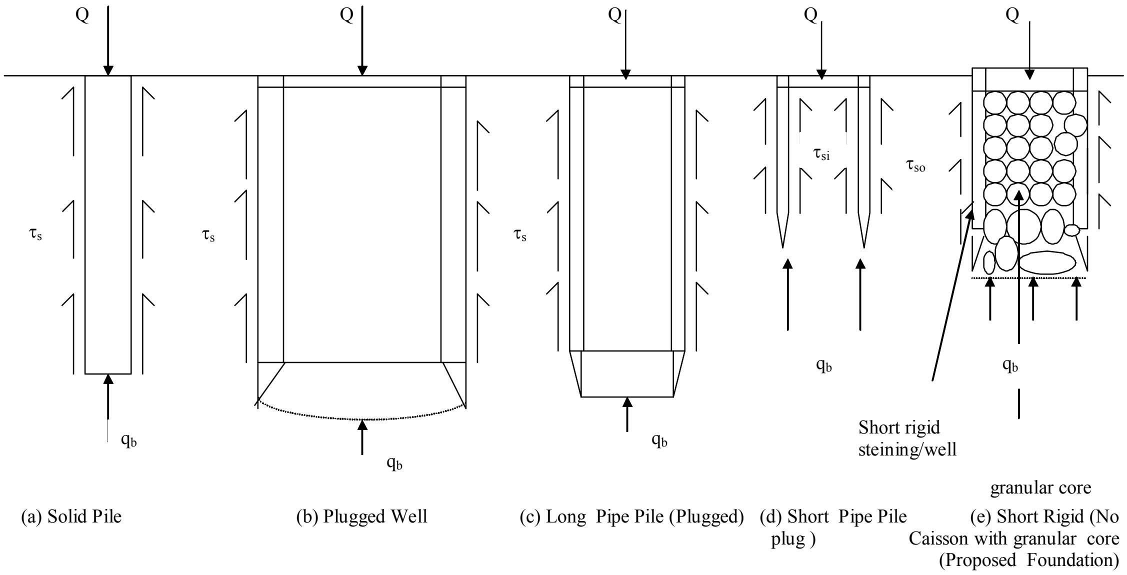

Fig. 2 Comparison of load transfer mechanism of various foundations

MECHANISM OF THE PROPOSED NEW COMPOSITE FOUNDATION

A solid pile or a plugged caisson or a well foundation transfer (Fig. 2a, b and c) the applied load through shaft resistance and end-bearing. A long hollow pipe pile also behaves similar to a solid pile if its length is sufficient to form a plug (Randolph 1987) but very little load is transferred through base resistance. A short pipe pile (Fig. 2 d) on the other hand mobilizes only positive skin resistance on both its inner and outer surfaces. The proposed composite foundation functions similar to a short pipe pile except that the granular infill is much stronger and stiffer than the original ground. Hence, it can carry and transfer part of the applied load (Fig. 2c). While the inner surface of the pipe or caisson resists the applied load by positive resistance, the granular infill is subjected to down drag or negative skin resistance because of which larger loads are transferred through base. Granular material, if confined, deforms one dimensionally and becomes stiffer with increasing confining stress.

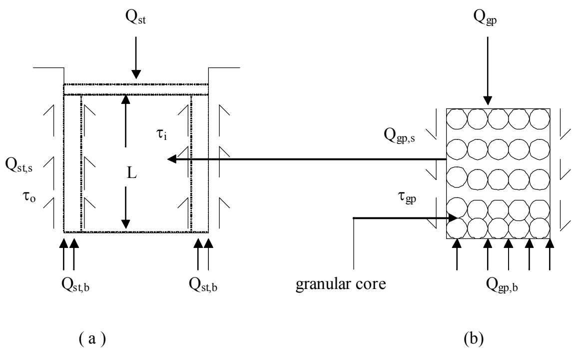

Fig. 3 Distribution of stresses on (a) well steining and (b) granular core

ASSUMPTIONS

The following assumptions are made for analyzing the proposed foundation:

1) The soil is homogeneous and isotropic.
2) The interaction on inner side of steining and granular core generates negative skin friction that is imposed on the core.
3) Granular material (core material) deforms one dimensionally and becomes stiffer with increasing confining stress

The top vertical displacements of the steining and core are equal because the load is transmitted through a rigid cap.

OVERALL EQUILIBRIUM

The ultimate or maximum load, Q_{ult}, the new foundation can carry is

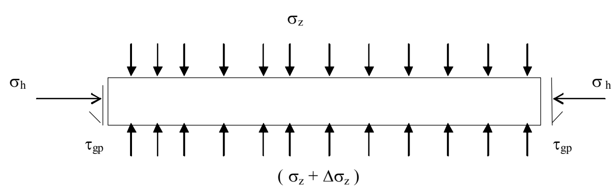

Fig. 4 Stresses acting on an element of granular core

$$Q_{ult} = Q_s+Q_b= \pi d_o L\tau_m + (\frac{\pi}{4})d_0^2 q_{ub} \ldots\ldots\ldots (1)$$

where Q_s and Q_b are respectively the maximum shaft and base resistances and π_m and q_{ub} the maximum unit shaft and end bearing resistances respectively.

The allowable load, Q_{all}, is

$$Q_{all} = \frac{Q_s}{FS_1} + \frac{Q_{ub}}{FS_2} \qquad (2)$$

where FS_1 and FS_2 are the factors of safety for ultimate shaft and end bearing resistances. Usually FS_1 = 1.5 while FS_2 ranges between 2.5 and 3.0. For applied load $Q \leq Qall$, the response of the soil around and beneath the proposed foundation can be assumed to be linear.

The applied load, Q is shared at the top (Fig. 3) by the well steining, $Q_{st,}$ and the granular core, Q_{gp}. The relative proportions of loads transferred to the steining Q_{st}, and the core Q_{gp}, are controlled by the stiffness, geometry and interfacial shear stresses. The vertical force equilibrium for the composite foundation with granular core inside is expressed as

$$Q = Q_{st}+Q_{gp} \qquad (3)$$

or $$= Q_{st,s}+ Q_{st,b}+ Q_{gp,L} \qquad (3a)$$

where $Q_{st,s}$ and $Q_{st,b}$ are respectively the loads shared by the outer steining surface and of the steining base and $Q_{gp,b}$ the load transferred by the base of granular core. Expressing the forces, Q , in terms of stresses

$$Q = \pi\, d_0 L\tau + [\pi(d_0^2 - d^2)/4]q_{st,b} + \pi(d^2/4)q_{gp,L} \qquad (4)$$

Simplifying

$$q = 4(L/d_0)\tau+[(1-(d^2/d_0^2))q_{st,b}+(d^2/d_0^2)q_{gp,L} \qquad (5)$$

where q, τ, $q_{st,b}$ and q_{gp} are the average stresses on the foundation outer surface of steining, steining base and the granular core respectively.

ANALYSIS OF GRANULAR CORE

The vertical equilibrium of an element of the granular core (Fig.4), neglecting its weight, is

$$(\sigma_z+\Delta\sigma_z)(\pi/4)d^2-\sigma_z(\pi/4)d^2-\tau_{gp}(\pi\, d) = 0 \qquad (6)$$

$$(d\sigma_z/dz)-(4/d)\tau_{gp} = 0 \qquad (6a)$$

Assuming full mobilization of shaft resistance between granular core and inner surface of the caisson/steining, i.e. τ_{gp} = K σ_z tanδ where K is the lateral coefficient of earth pressure, σ_z , the vertical stress, τ_{gp} , shearing stress on the surface of granular core and δ , the wall friction angle. The above equation is integrated as

$$\sigma_z = c_o \exp(c_1 z) \qquad (7)$$

where $c_1 = (4/d)\, K \tan\delta$ and c_0 is a constant.

At the top of the granular core, i.e z = 0, $\sigma_z = q_{gp}$, and hence, one gets $c_o = q_{gp.}$

The stress transferred by the granular core, $q_{gp,\,L}$ to the soil below i.e. at z = L becomes

$$q_{gp,\,L} = q_{gp}\ d_1 \qquad (8)$$

where $d_1= \exp(c_1L)$. The settlement, $w_{s,L}$ of the soil below the granular core, i.e. at z = L, from Poulos and Davis (1980)

$$w_{s,L} = q_{gp,L}\,[d\,(1-\nu_s^2)\,C/\,E_s] = [q_{gp,\,L}\,/\,(k_{s,L}\, I_f)] \qquad (9)$$

where C = an influence factor (Fox 1948), I_f = (1/C) = constant and $k_{s,L,}$ modulus of subgrade reaction of the soil below the granular core = $[E_s/d\,(1 - \nu_s^2)]$. The granular core is under $K \geq K_o$ condition and its compression, Δw_{gp}, is evaluated by integrating the one dimensional compression equation for an element as

$$\Delta\, w_{gp} = \int_0^L \left(\frac{\sigma z}{Dgp}\right) dz$$

$$\Delta w_{gp} = (q_{gp\,/}\, D_{gp})[d_r d_o(d_1 - 1)/t\,] \qquad (10)$$

where $d_1= \exp(tl_r\,/d_r)$, $d_r = d/d_o$, $l_r = L/d_o$, $t = 4K\tan\delta$, the constrained modulus $D_{gp} = [E_{gp}\,(1-\nu_{gp})\,/\,(1+\nu_{gp})\,(1-2\nu_{gp})] = \beta\, E_{gp,}$ and $\beta = (1 - \nu_{gp})\,/\,[(1+ \nu_{gp})\,(1- 2\nu_{gp})]$.

The settlement at the top of the granular core, $w_{gp,0}$, i.e. at z = 0, is the sum of the compression of the core and the settlement of the soil below, and is obtained as

$$w_{gp,0} = w_{s,L} + \Delta w_{gp} = q_{gp}\ f_1 \qquad (11)$$

where

$$f_1 = [\{d_1\,/\,(k_{s,L}\,\alpha_f\,)\} + \{d\,(\,d_1 - 1)\,/\,D_{gp}\,t\}] \qquad (12)$$

The steining assumed to be rigid settles by w_{st}. Compatibility of displacements of the well and the top of the granular core requires

$$w_{st} = w_{gp,0} \qquad (13)$$

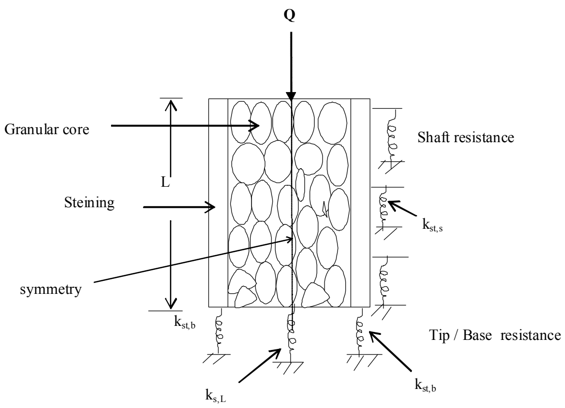

Fig. 5 Winkler representation of soil response to load on composite foundation

Hence, based on Scott (1981)

$$\tau = k_{st,s}.w_{st} = k_{st,s}\, w_{gp,0} \qquad (14)$$

and

$$q_{st,b} = k_{st,b}..w_{st} = k_{st,b}\, w_{gp,0} \qquad (15)$$

where $k_{st,s}$ the spring constants for the shaft resistance is related to base stiffness,$k_{st,b}$ (Scott 1981) as $k_{st,s} = \alpha_{s,b}\ k_{st,b}$ where $\alpha_{s,b}$ is a constant of proportionality and $k_{st,b}$ is the stiffness of base of steining which in turn is related to stiffness of soil below the granular base as $k_{st,b} = \alpha_b.k_{s,L}$ where α_b is a constant of proportionality.

Combining Eqs. (5), (8), (11), (13), (14) and (15) and simplifying

$$q = q_{gp}F \qquad (16)$$

or in terms of forces

$$Q_{gp} = (Q/F)\ d_r^2 \qquad (17)$$

and $(Q_{ST}/Q) = [1-(Qgp/Q)]$ (18)

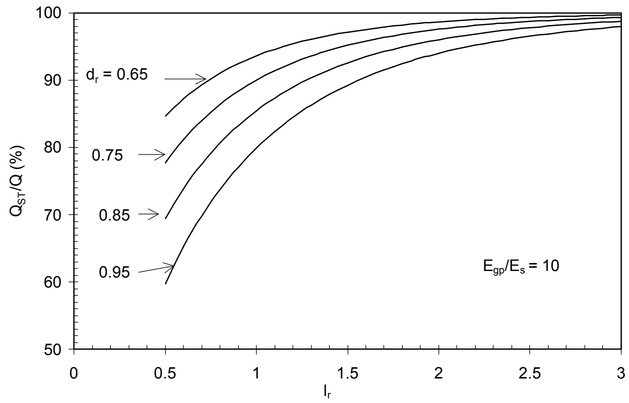

Fig. 6 Effect of l_r and d_r on percent load carried by the steining

where $F = f_1\ f_2 + f_3 = [4\alpha_{s,b}\ l_r + (1 - d_r^2)]\ [\{\alpha_b\ d_1 / I_f\} + \{\alpha_b\ d_r\,(d_1 - 1) / R\ t\}] + d_r^2 d_1$, $R = (D_{gp} / k_{s,L}\ d) = \beta^*\ (E_{gp} / E_s)$, is relative granular core stiffness and $\beta^* = (1- \nu_s^2)\ \beta$ and $f_2 = [4(l_r)k_{st,s} + (1- d_r^2)k_{st,b}]$ and $f_3 = d_r^2 d_1$.

Combining Eqs. (11), (13) and (17) and normalizing, w_{st}, the settlement of the composite foundation is

$$w_{st}E_s d_0/Q = [4d_r(1-\nu_s^2)/\pi](1/F)[(d_1/I_f) + \{(d_r/t)(d_1-1)/R\}] \qquad (19)$$

and the settlement of the soil below the granular core, $w_{s,L}$

$$w_{s,L}E_s d_0/Q = [(4d_r(1- \nu_s^2)/\pi)(1/F)(d_1/I_f)\ ] \qquad (20)$$

RESULTS AND DISCUSSION

Results have been obtained for a range of parameters to illustrate their influences on the behavior of the proposed composite foundation with granular core. The typical values of the parameters considered in this study are given in Table 1. In the following sections, solutions are presented and discussed for the effects of l_r, d_r, ν_s, (E_{gp} / E_s), α_b, $\alpha_{s,b}$, ϕ and K.

Table 1 Typical values of parameters considered

Parameter	Range / assigned value
Poisson's ratio of soil (ν_s)	0.3 - 0.5
Poisson's ratio of granular core (ν_{gp})	0.25
Modular ratio, E_{gp} / E_s	1, 2, 5, 10 and 100
Relative steining / granular core stiffness (α_b)	0.25, 1.0 & 4.0
Relative steining surface / base stiffness ($\alpha_{s,b}$)	0.25, 1.0 & 4.0
Diameter ratio (d_r)	0.65 – 0.95
Length to diameter ratio (l_r)	0.50 – 3.00
Friction angle of soil, φ	10^0 - 40^0
Coefficient of lateral pressure, k	k_0 (at rest) - $2.0k_0$
Influence factor, I_f	1.0

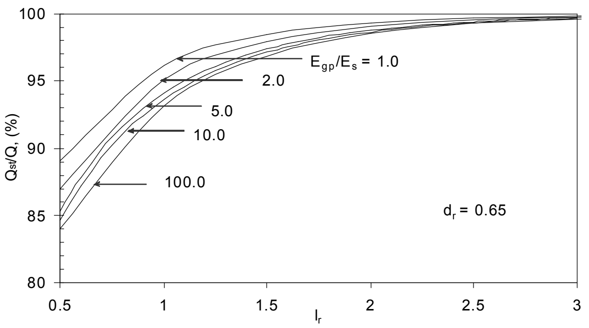

Fig. 7 Effect of modular ratio (E_{gp}/E_s) on percent load carried by steining

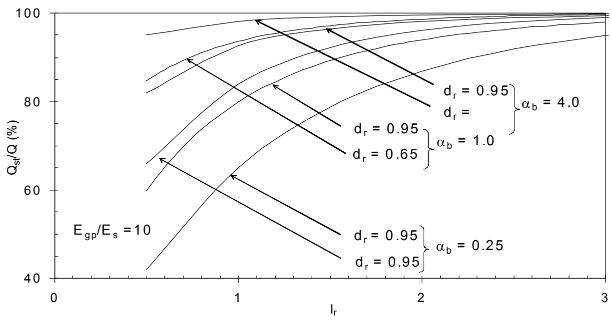

Fig. 8 Effect of α_b on percent load carried by steining

LOAD SHARING BETWEEN WELL STEINING AND GRANULAR CORE

Effects of Length-Diameter Ratio (l_r) and Diameter Ratio (d_r)

The variation of the percentage of load carried by well steining with l_r and d_r for $\nu_s = 0.5$, $\nu_{gp} = 0.25$, $\alpha_b = 1.0$, $\alpha_{s,b} = 0.25$, and $(E_{gp} / E_s) = 10.0$ is shown in Fig. 6. The load carried by the well steining increases with increases in the length to diameter ratio, l_r. For a given value of d_r, and for l_r in the range 0.5 to 1.0, Q_{st}/Q % increases sharply. But for $l_r > 1.0$, the rate of increase of Q_{st}/Q % with l_r becomes small and asymptotic to 100%. For $l_r > 3.0$, all the applied load is taken by the steining alone with the granular core not carrying any load. Thus, the proposed foundation is applicable only for $l_r \leq 3.0$.

For a given l_r, the load carried by steining increases with decrease in d_r i.e. with increasing steining thickness. The percent load taken by the steining is approximately 60% & 90% for d_r equal to 0.95 and 0.65 respectively and for l_r=0.5. A decrease in diameter ratio (d_r) implies an increase in the base area of the steining and a corresponding decrease of the granular core area. Similar trend is reported by Desai and Chandrasekharan (1985) for short concrete caissons ($E_{gp}/Es = 1.0$, $d_r = 1.0$). For steining with $l_r \geq 3$, the diameter ratio, d_r, has no effect on the load carried by the steining, as all the load gets supported by the steining alone.

Effect of Relative Steining Base Stiffness (α_b)

The effect of modular ratio i.e. the ratio of modulus of deformation of granular core to that of soil, E_{gp}/E_s, on percent load carried by the steining for $\nu_s = 0.5$, $\nu_{gp} = 0.25$, $\alpha_b = 1.0$, $\alpha_{s,b} = 0.25$, and $d_r = 0.65$ is depicted in Fig. 7. The percent load carried by the steining decreases with increase in the modular ratio, E_{gp}/E_s. A higher (E_{gp}/E_s) ratio reflects stiffer granular core and hence less load is carried by the steining. The decrease is significant for E_{gp}/E_s increasing from 1 to 10. Further increase in the modular ratio E_{gp}/E_s from 10 to 100, leads to only a marginal decrease in the percentage load carried by the steining.

The stiffnesses of the soil beneath the steining and that of the soil beneath granular core are characterized by the subgrade moduli, $k_{st,s}$ and $k_{s,L}$ respectively. The stiffness of the soil beneath the steining, $k_{st,b}$, is related to that beneath the granular core, $k_{s,L}$, through a factor α_b. The difference arises from the relative sizes of the areas. While the base of the steining is an annulus with inner and outer diameter, d and d_0 respectively, the granular core transfers the load uniformly over an area with a diameter d.. The stiffnesses $k_{st,b}$ and $k_{s,L}$ are evaluated from the elastic continuum theory (Poulos and Davis 1980, Scott 1981). For convenience, a parametric study is presented for different values of α_b (0.25 - 4.0).

The load taken by steining increases with increase in

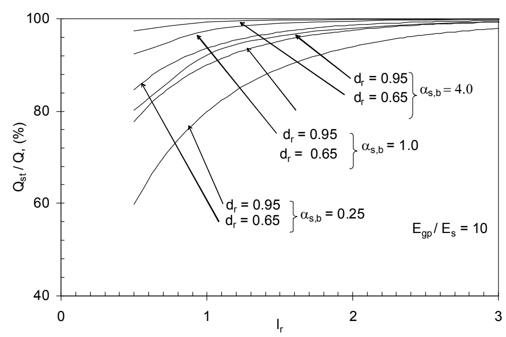

Fig. 9 Effect of $\alpha_{s,b}$ on percent load carried by steining

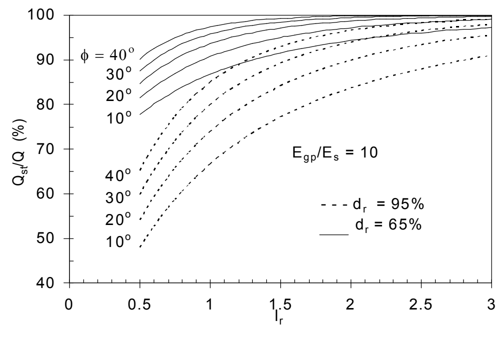

Fig. 10 Effect of angle of friction of soil on percent load carried by steining

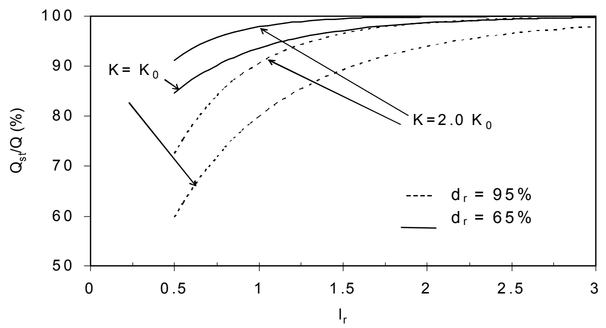

Fig. 11 Effect of coefficient of lateral pressure (K_0) on percent load shared by steining

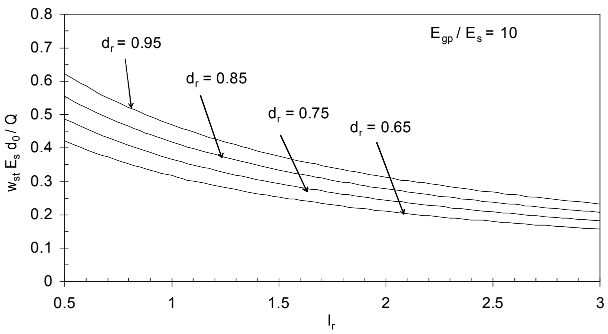

Fig. 12 Variation of settlement of composite foundation with d_r and l_r

values of α_b (Fig. 8). For $E_{gp}/E_s = 10.0$, $\nu_s = 0.5$, $\alpha_{s,b}= 0.25$, $d_r = 0.95$ and $l_r = 0.50$, the percent load carried by the steining is approximately 41, 58 and 77 at relative steining base stiffness, α_b, of 0.25, 1.0 and 4.0 respectively. Similar trends are observed at $d_r = 0.65$. An increase in α_b reflects a stiffer soil beneath the base of the steining and hence, leads to an increase in the load carried by the well steining.

Effect of Relative Steining Surface to Base stiffness ($\alpha_{s,b}$)

The ratio, $\alpha_{s,b}$, reflects the relative shear stiffness of the steining surface ($k_{st,s}$) with respect to that of the soil beneath its base ($k_{st,b}$). The load taken by the steining increases with increase in the relative stiffness of steining - soil surface with respect to that of the base ($\alpha_{s,b}$) as shown in Fig. 9. For $E_{gp}/E_s = 10.0$, $\nu_s = 0.5$, $\alpha_b = 1.0$, $d_r = 0.95$ and $l_r = 0.50$, the percent load carried by the steining is approximately 60, 79 and 93 for values of $\alpha_{s,b}$ of 0.25, 1.0 & 4.0 respectively. Similar trends are observed for $d_r = 0.75$ and 0.65. An increase in $\alpha_{s,b}$ implies a stiffer soil -steining interface and hence, to an increase in the load carried by the steining.

Effect of Angle of Friction of Granular Core (ϕ)

The percentage of load carried by the steining decreases with increase in the angle of friction of the granular core (ϕ) for constant l_r and d_r. Also for a constant l_r and ϕ the percentage of load carried by steining decreases with increase in d_r (Fig. 10). It is due to fact that the ratio of point and skin resistances, f_b / f_s, increase with ϕ (Vesic 1963). At a constant l_r, f_b will increase with ϕ. But as the base area of the granular core is more than that of the steining, the steining will carry less load with increase in ϕ.

Effect of Coefficient of Lateral Pressure (K)

The percentage of load carried by the steining decreases with increase in coefficient of lateral pressure (K) for a constant d_r. Also for a constant l_r, the percentage of load carried by steining decreases with increase in K and d_r (Fig. 11). It is due to the fact that the shaft resistance increases with increase in K, causing an increase in percent load carried by steining.

SETTLEMENTS OF WELL STEINING AND GRANULAR CORE

Effect of Length to Diameter Ratio (l_r) and Diameter Ratio (d_r)

The variation of normalized settlement of composite foundation (w_{st} E_s d_0 / Q) with l_r for $\nu_s = 0.5$, $\alpha_b = 1.0$, $\alpha_{s,b} = 0.25$ and E_{gp} / E_s = 10.0 is shown in Fig. 12. For $l_r = 0.5$, the normalized settlements of the composite foundation are

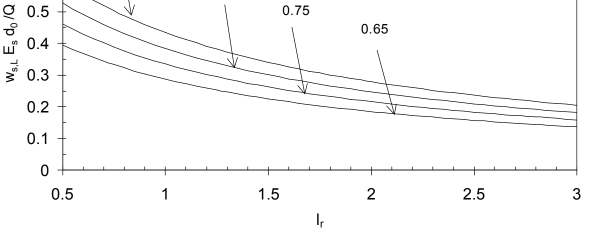

Fig. 13 Variation of settlement of granular core at base with l_r and d_r

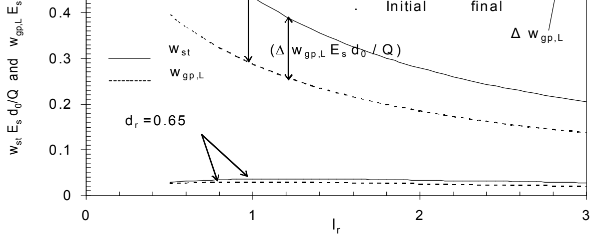

Fig. 14 Variation of settlement difference ($\Delta w_{gpL}E_sd_0/Q$) with l_r and d_r

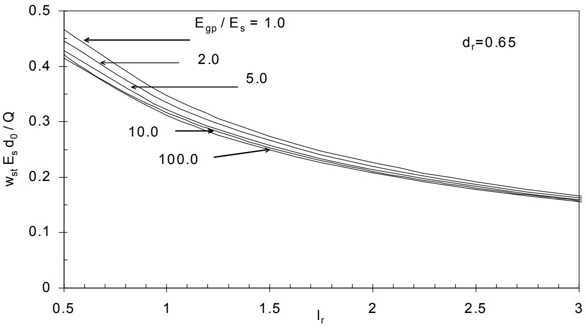

Fig. 15 Effect of modulus ratio (E_{gp} / E_s) on settlement of composite foundation

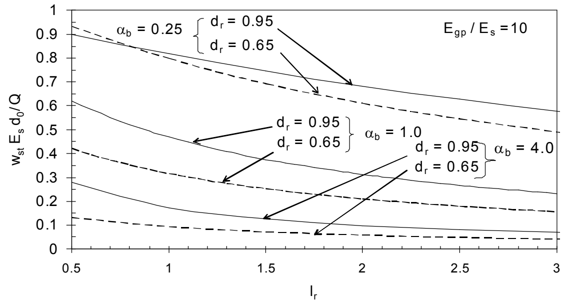

Fig. 16 Effect of α on settlement of steining / composite foundation

0.62, 0.56, 0.49 and 0.43 for d_r = 0.95, 0.85, 0.75 and 0.65 respectively. Similar trends are observed for all values of l_r. The normalized settlement of the composite foundation decreases with increase in l_r for all ratios of d_r, while it increases with increase in d_r for all l_r.

The variation of the normalized settlement of the ground beneath the granular core, $w_{gp,L}$ E_s d_0 / Q with l_r for ν_s = 0.5, α_b = 1.0, $\alpha_{s,b}$ = 0.25 and E_{gp} / E_s = 10 is presented in Fig. 13. For a constant l_r = 0.5, the normalized settlements of the soil beneath the granular core are 0.59, 0.52, 0.45, and 0.4 for d_r = 0.95, 0.85, 0.75 and 0.65 respectively. With the decrease of d_r , the normalized settlement beneath the granular core decreases.

The settlements of both the well steining and the granular core of the proposed foundation are the same at the top, i.e. z = 0. Due to differences in the stiffnesses of the concrete steining and the granular core, steining penetrates relatively more into the soil, resulting in difference in the settlement of steining and of the soil beneath the granular core at the base. The difference, Δw_{gp}, between these two settlements is due the compressibility of the granular core (Fig. 14).

Effect of Modular Ratio (E_{gp}/E_s)

Fig. 15 shows the effect of modular ratio (E_{gp}/E_s) on normalized settlement of composite foundation (w_{st} E_s d_0 / Q), for ν_s = 0.5, α = 1.0, α_1 = 0.25 and d_r =0.65. For l_r = 0.5, the normalized settlements of the composite foundation are 0.46, 0.45, 0.43, 0.425 and 0.42 at (E_{gp}/E_s) = 1.0, 2.0, 5.0, 10.0 and 100.0 respectively. Similar trend is observed for all l_r. An increase in modulur ratio from 10 to 100, leads to only a small decrease in the settlement of composite foundation. Similar trend is reported by Desai and Chandrasekharan (1985) for short concrete caissons (E_{gp}/Es = 1.0, d_r= 1.0).

Effect of Relative Steining Base Stiffness (α_b)

The normalized settlement of proposed foundation/steining decreases with increase in relative steining base / granular core stiffness (α) as shown in Fig. 16. For E_{gp}/E_s = 10.0, ν_s = 0.5, $\alpha_{s,b}$= 0.25, d_r = 0.95 and l_r = 0.50, the normalized settlements of composite foundation are approximately 0.90, 0.63 and 0.28 at relative steining base / granular core stiffness (α_b) of 0.25, 1.0 & 4.0 respectively. Similar trend is observed for d_r = 0.65. It is due to the fact that an increase in α_b reflects a stiffer soil beneath the steining base and hence, to an increase in the load carried by the well steining and consequently, reduction in settlement.

Effect of Relative Steining Surface to Base stiffness ($\alpha_{s,b}$)

The normalized settlement of the proposed foundation decreases with increase in relative stiffness of steining

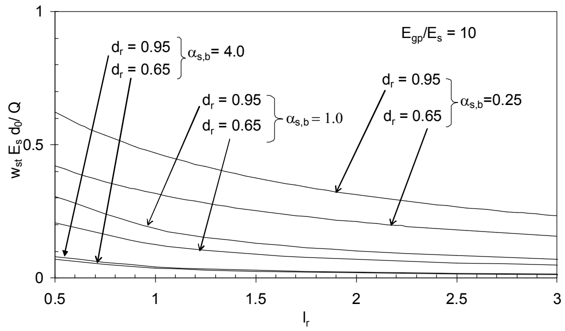

Fig. 17 Effect of $\alpha_{s,b}$ on settlement of steining / composite foundation

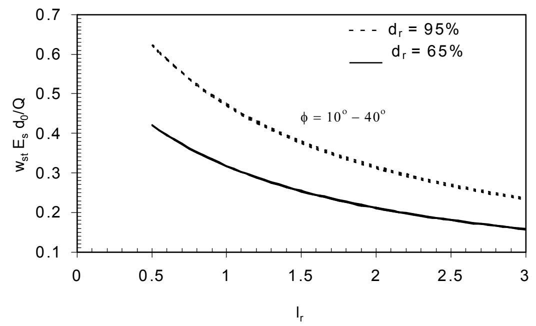

Fig. 18 Effect of angle of friction of soil on settlement

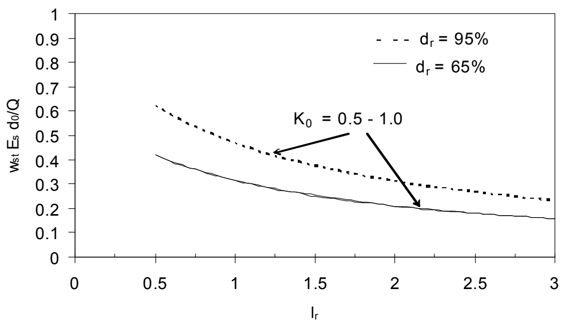

Fig. 19 Effect of coefficient. of lateral pressure, (K_0) on normalized settlement

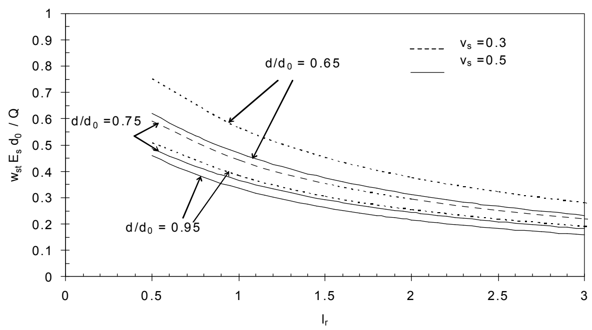

Fig. 20 Effect of Poisson's ratio of soil on settlement of steining / composite foundation

surface with respect to base ($\alpha_{s,b}$) as shown in Fig. 17. For $E_{gp}/E_s = 10.0$, $\nu_s = 0.5$, $\alpha_b = 1.0$, $d_r = 0.95$ and $l_r = 0.50$, the normalized settlements of foundation are 0.65, 0.30 & 0.10 at relative steining surface / base stiffnesses ($\alpha_{s,b}$) of 0.25, 1.0 and 4.0 respectively. Similar trend is observed for $d_r = 0.65$. It is due to the fact that an increase in $\alpha_{s,b}$ reflects a stiffer steining base and hence, to an increase in the load carried by the well steining and consequently, reduction in settlement.

Effect of Angle of Friction of Soil (ϕ)

The normalized settlement of composite foundation is almost independent of the angle of friction of soil (ϕ) for a constant l_r and d_r (Fig. 18). Also for all l_r, the settlement of steining is less for d_r=65% in comparison with that for d_r=95% and is almost independent of ϕ. As the base area of the granular core is more as compared to the steining base area d_r=65%, the steining undergoes less settlement.

Effect of Coefficient of Lateral Pressure (k)

The settlement of steining is almost independent of the variation in coefficient of lateral pressure (K) for all d_r. Also for a constant l_r, the settlement of steining decreases with decrease in d_r (Fig. 19).

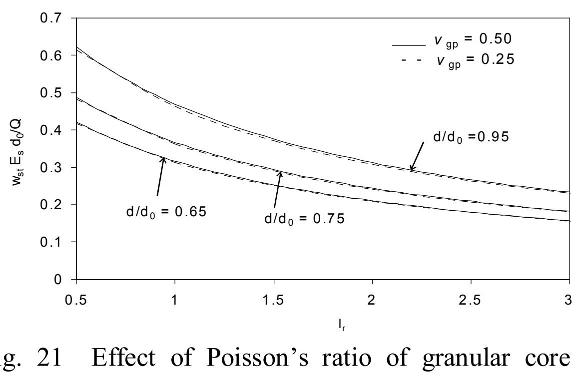

Fig. 21 Effect of Poisson's ratio of granular core on settlement of steining / composite foundation

Effect of Possion's Ratio of Soil & Granular Core

Effects of Poisson's ratio of soil on normalized settlement is depicted in Fig. 20. It is evident from the figure that settlement decreases from 0.51 to 0.45 for Poisson's ratio increasing from 0.3 to 0.5 for $E_{gp} / E_s = 10.0$, $\alpha_b = 1.0$, $d_r = 0.95$ and $l_r = 0.50$. Similar trend is observed for $d_r = 0.65$. Higher Poission's ratio, υ_s, corresponds to stiff soil and to a reduction in settlement.

Effect of Poisson's ratio of granular core, ν_{gp}, on normalized settlement is depicted in Fig. 21. It is evident from the above figure that Poisson's ratio, ν_{gp}, considered has almost no influence on the settlement. Similar trend was reported for granular piles (Alamgir et al. 1996).

CONCLUSIONS

A new composite foundation with granular core is proposed for loose alluvial deposits and a simple theoretical approach using Winkler type responses, i.e. linear shear stress-displacement relationship for outer surface of composite foundation and linear bearing stress – displacement relationships for steining and base of granular pile, is presented. The parametric study quantifies the effects of length to diameter ratio(L/d_0), inner to outer diameter ratio, modulus ratio (E_{gp} / E_s) and relative steining-granular core stiffness (α_b), Poisson's ratio of soil (υ_s), and relative steining surface to base stiffness ($\alpha_{s,b}$) on the sharing of the applied load by the well steining and the granular core and on the settlement of the foundation.

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NOTATIONS

d = diameter of granular core / inner diameter of the well steining.

d_o = diameter (outer) of the well steining.

d_r = diameter ratio.

D_{gp} = constrained modulus of granular core.

E_s = Young's modulus of soil.

E_{gp} = Young's modulus of granular core.

k = Coefficient of lateral pressure

k_0 = Coefficient of lateral pressure at rest.

k_{gp} = Winkler's subgrade reaction coefficient of granular core.

$k_{s,L}$ = Winkler's subgrade reaction coefficient of granular core at base.

$k_{st,s}$ = Winkler's subgrade reaction coefficient of steining (surface).

$k_{st,b}$ = Winkler's subgrade reaction coefficient of steining (base).

L = length of steining.

l_r = length - diameter ratio.

Q = load applied to composite foundation with granular core.

Q_{st} = load taken (shared) by the well steining of composite foundation.

Q_{gp} = load taken (shared) by the granular core of composite foundation.

q_{st} = load intensity (stress) taken by the well steining of the composite foundation.

q_{gp} = load intensity (stress) taken by the granular core of the composite foundation.

$q_{st,s}$ = load intensity (stress) taken by the steining (surface) of the composite foundation.

$q_{st,b}$ = load intensity (stress) taken by the steining (base) of the composite foundation.

$q_{gp,s}$ = load intensity (stress) taken by the granular core (surface) of the composite foundation.

$q_{gp,b}$ = load intensity (stress) at the granular core (base) of the composite foundation.

R = relative granular core stiffness.

w_{st} = displacement of the steining relative to soil.

$w_{s,L}$ = displacement of of the granular core at base.

α_b = ratio of coefficient of subgrade reaction of steining and granular core.

$\alpha_{s,b}$ = ratio of coefficient of subgrade reaction of steining surface and base.

I_f = settlement influence factor.

τ_{gp} = shearing stress on the surface of the granular core.

τ_i = shearing stress on the inner surface of the steining.

τ_o = shearing stress on the outer surface of the steining.

ν_s = Poisson's ratio of soil.

ν_{gp} = Poisson's ratio of granular core.

ϕ = Angle of friction of granular core

β = ratio of constrained modulus and Young's modulus of granular core.