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Skirted Granular Pile Foundation for High Water Table Areas

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Introduction

Lands affected by fluctuating surface water levels e.g. by tides, floods, etc. are classified as lowlands. Large tracts of flood plains and coastal lands which are below mean sea level exist all over the world notably in the Netherlands, Japan, Bangladesh, India, Thailand, etc. In many of the developing countries, due to heavy population pressure, human settlements are encroaching on to the flood plains of major rivers which are inundated periodically. Some of the lowlands of the world are reviewed and difficulties encountered in their development with respect to geotechnical aspects are presented by Madhav and Miura (1994).

The top soil strata of flood plains and lowlands are often too soft to bear any load. Due to high water table throughout the year, conventional shallow foundations are difficult to construct due to flooding of the foundation pit. The chances of erosion of the foundation soil during floods is more. Conventional de-watering techniques are very expensive especially for small and medium size projects. Use of deep foundations such as piles & caissons (well foundations) which overcome these difficulties are also expensive. Several ground improvement methods such as granular piles, etc. are possible to overcome this problem.

Ground improvement with granular piles (GP) / stone columns / sand compaction piles (SCP) is considered as one of the most versatile and cost effective alternative. This technique has been used in many difficult foundation sites throughout the world to increase the bearing capacity, to reduce settlement, to increase the rate of consolidation and also to improve the resistance to liquefaction (Alamgir et al. 1994). This ground improvement technique has already proven its applicability in various geotechnical engineering projects throughout the world (Greenwood 1970, Bergado et al. 1991, Madhav and Miura 1994, Ranjan and Rao 1994, Madhav and Nagpure 1995, Poorooshasb and Meyerhof 1997). However, necessary equipments are not easily available locally.

To circumvent the problems mentioned above, it was decided to develop a suitable and economic foundation for floodplain and lowlands, using locally

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available material and technology. The concept of skirted granular pile along with well steining is developed for the proposed foundation.

Literature Review

Granular piles / stone columns are cost effective technique for construction in difficult foundation sites especially in developing countries like India (Ranjan, 1989). These piles improve the performance of foundations on soft ground both by reducing the settlement to an acceptable level and by increasing the load carrying capacity to a desired level. In addition, granular piles densify the in-situ soil, drain rapidly the generated pore pressures, accelerate consolidation and minimize post-installation settlement. Vibro-compaction and vibro-replacement methods are widely used for stone column / granular pile installation in non-cohesive and cohesive soils respectively (Thornburn and McVicar 1960, Baumann and Baur 1974). The capacity of stone column is controlled by the undrained shear strength and in-situ lateral stresses in the soil, and the angle of internal friction of the granular column material (Hughes and Withers 1974, Hughes et al. 1975). Priebe (1976) proposed the reduced stress method for the estimation of reduction in settlement due to ground improved with stone columns. Aboshi et al. (1979) proposed the equilibrium method which is based on the concept that the vertical stress concentration on the stone column gives a reduced average stress on the soft soil. Barksdale and Bachus (1983) introduced the concept of equivalent parameters for composite ground. Ranjan (1989) and Ranjan and Rao (1994) used the analogy of expansion of cylindrical cavity (Vesic 1972) and the concept of equivalent coefficient of volume compressibility in homogeneous, isotropic and infinite soil mass to estimate the ultimate bearing capacity and settlement of ground treated with granular piles. Van Impe and Madhav (1992) derived expressions to predict the effect of dilatancy of the granular pile material on the settlement behaviour of stone column reinforced ground. The densified stone column material is considered to be at the yield condition and hence, dilating. Poorooshasb and Meyerhof (1997) examine the influences of column spacing, the weak soil properties, etc. on the load-carrying capacity of stone column foundation. Based on these studies, the factors that affect the performance of stone column foundation are identified as spacing of columns (or the area ratio) and the degree of compaction of the material in the columns.

Improved load capacity and substantial decrease in settlement can be achieved by resorting to skirted granular pile foundations (Narahari and Rao, 1979). This concept has further been supported by Broms (1981) as a new shallow foundation method, where ‘Skirt’ is renamed as “Tubular Elements”, which can primarily be used in granular soil, where heavy concentration of loads occur. Weak sub-soils supporting column foundation reinforced with granular piles / stone columns individually or collectively skirted (Rao and Bhandhari, 1979) are capable of supporting very high loads and provide significant reduction in overall settlement (Ranjan and Rao, 1983 and 1989). Behaviour of skirted granular pile has been studied and various parameters affecting its performance and application have been reported by Rao and Bhandari (1979), Rao and Sharma (1980), Rao and Ranjan (1980 and 1985).

Granular piles/stone column are a very simple option to improve soft ground or loose sand deposits. However, their load-carrying capacity under isolated foundations is limited by the bulging capacity which can be very low if the effective lateral stresses are small as is the case in high water table areas. Skirted granular piles have been shown to overcome this aspect. An innovative concept of skirted granular pile using locally available pre-cast concrete steining is developed as an appropriate foundation for flood plains with high water table.

Proposed Foundation

A new short rigid composite foundation consisting of shallow open caisson with granular core inside is being proposed for foundations in lands with high water table (Figure 1). It consists of shallow pipe or well steining (outer diameter = 1.0 to 1.5 m, thickness of steining = 100-150 mm and length =1.5m to 4.0 m). It 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 proposed composite foundation. The elastic moduli, Poisson's ratios and angles of shearing resistance of the soil and the granular core are E_s, ν_s, ϕ' and E_{gp}, ν_{gp}, ϕ_{gp} respectively. A vertical load, Q, is applied at the top of the proposed foundation.

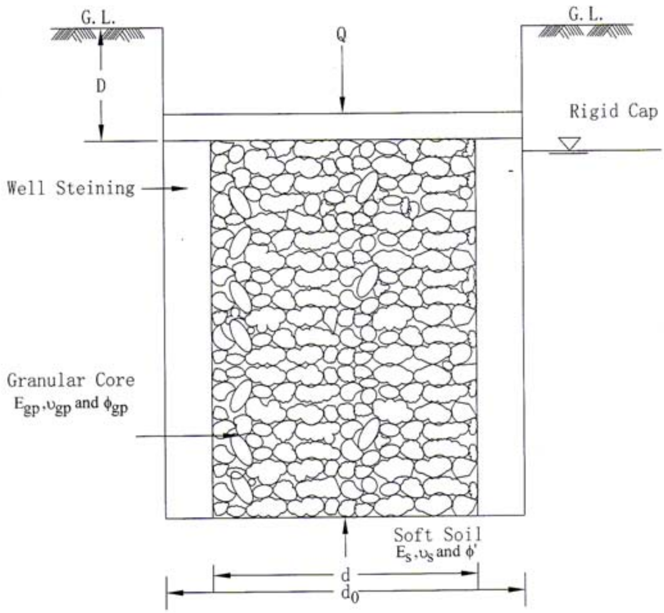

Fig. 1 Proposed Innovative Foundation

The proposed composite rigid caisson with granular core functions similar to a short pipe pile except that the granular infill is much stronger and stiffer than the original ground. Hence, it carries and transfers part of the applied load. The steining is relatively incompressible and hence settles more than the granular core. Therefore, the outer and inner surfaces of the pipe or caisson resist the applied load by positive shaft resistances. The granular infill would therefore be subjected to down-drag or negative skin resistance because of which larger loads are transferred through its base. Granular material, if confined, deforms one-dimensionally with stiffness increasing with confining stress.

Analysis Based on Linear Winkler Approach

Analysis of an axially loaded caisson comprises the evaluation of

> The structural capacity of the caisson to transmit the load;

> The capacity of soil to resist the load;

> The vertical displacement or settlement of caisson under the applied loads.

By far, the first two aspects have been paid more attention. However, the settlements govern the design (Poulos and Davis 1980).

The applied load, Q, is shared at the top (Figure 2) by the well steining, $Q_{st,}$ and the granular core, Q_{gp}. The relative proportions of loads transferred to the steining, Q_{st}, and to the core Q_{gp}, are controlled by the stiffness, geometry and interfacial shear stresses of the component materials. The vertical force equilibrium for the composite foundation with granular core inside is (Figure 2a)

$$Q = Q_{st} + Q_{gp} \tag{1}$$

or

$$= Q_{st,s} + Q_{st,b} + Q_{gp,L} \tag{2}$$

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. The forces and stresses on the steining and the granular core are depicted in Figures. 2b and 2c respectively. Expressing the forces in terms of stresses

$$Q=\left[\left(\frac{\pi}{4}\right)d_0{}^2\right] q = \pi\, d_0 L\, \tau + [\, \pi(d_0{}^2 - d^2) / 4]\, q_{st,b} + \pi(d^2 / 4)\, q_{gp,L} \tag{3}$$

simplifying

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

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

The granular core not only gets loaded from the top cap, Q_{gp}, but also from the down-drag stresses, τ_{gp}. The vertical equilibrium of an element of the granular core (Figure 3), neglecting its weight, is

$$(\sigma_z + \Delta\sigma_z) (\pi/4)\, d^2 - \sigma_z\ (\pi/4)\, d^2 - \tau_{gp} (\pi\, d\ )\, dz= 0 \tag{5}$$

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

where σ_z and $(\sigma_z + \Delta\sigma_z)$ are the normal stresses on the top and bottom planes respectively and τ_{gp} , the shear stress.

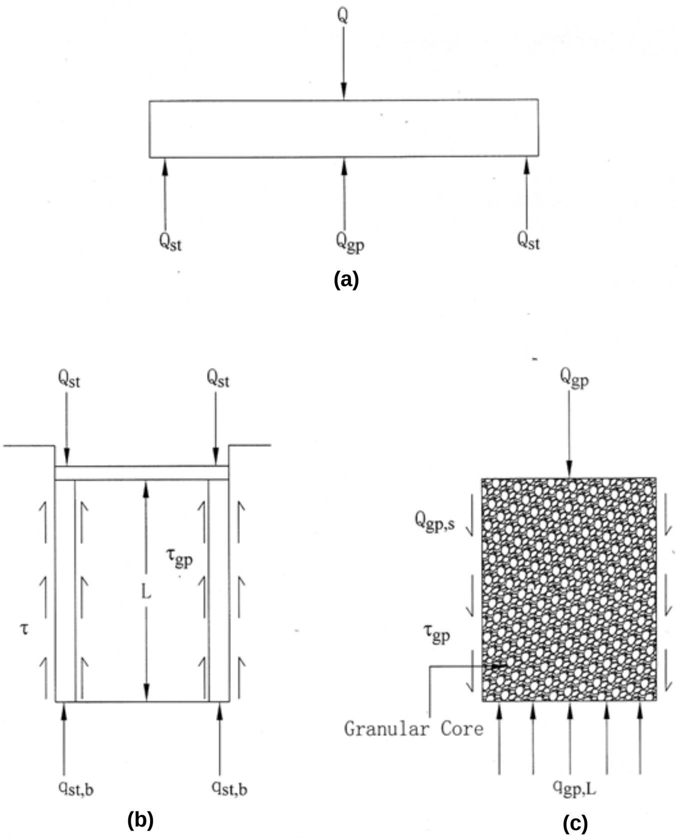

Fig. 2 Forces and Stresses on (a) Cap (b) Well Steining and (c) Granular Core

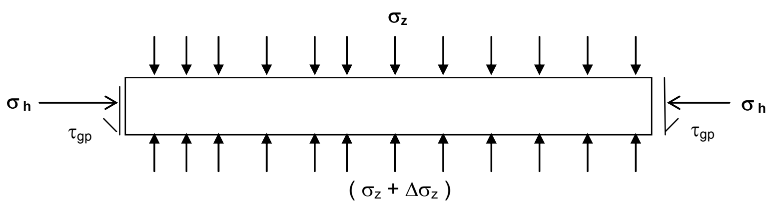

Fig. 3 Stresses Acting on an Element of Granular Core

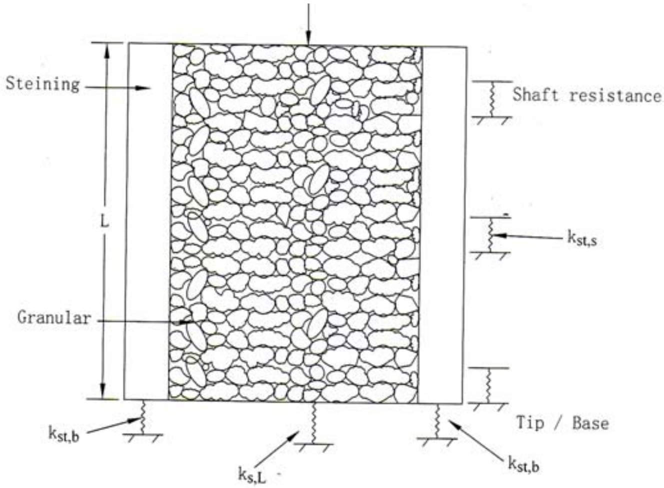

Fig. 4 Winkler Representation of Soil Response to Load on Composite Foundation

Assuming full mobilization of shaft resistance between granular core and inner surface of the caisson (steining), i.e. $\tau_{gp} = \sigma_h.\tan\delta = K.\sigma_z.\tan\delta$

where $\sigma_h = K.\sigma_z$ is the horizontal stress, K being the coefficient of lateral earth pressure and δ, the wall friction angle. Substituting for τ_{gp} in Eqn. 6, one gets

$$\frac{d\sigma_z}{dz} = \left(\frac{4}{d}\right) K\sigma_z \tan\delta \ dz \tag{7}$$

The above equation is integrated as

$$\sigma_z = c_o.\exp(c_1 z) \tag{8}$$

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

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

and

$$\sigma_z = q_{gp} \exp(c_1 z) \tag{9}$$

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

$$q_{gp,L} = q_{gp}.R_{gp} \tag{10}$$

where $R_{gp} = \exp(c_1 L)$.

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

$$w_{s,L} = \sigma_z \big|_{z=L} \left(\frac{d\,(1-\nu_s^2)\,I_f}{E_s} \right) \tag{11}$$

or

$$w_{s,L} = q_{gp,L} \left(\frac{d\,(1-\nu^2)\,I_f}{E_s} \right) = [\, q_{gp,\,L} / k_{s,L} \,] \tag{12}$$

where I_f = an influence factor, and $k_{s,L,}$ modulus of subgrade reaction of the soil below the granular core = $[E_s/d\ (1 - \nu_s^2)\ I_f]$. Substituting for $q_{gp,L}$ from Eqn. 10 into Eqn. 12, one gets

$$w_{s,L} = [\, q_{gp,}\, R_{gp} / k_{s,L} \,] \tag{13}$$

The granular core is under $K \geq K_0$ condition and its compression, Δw_{gp}, is evaluated by integrating the one dimensional compression equation for an element as

$$\Delta\, w_{gp} = \int_0^L \epsilon_z\, dz \tag{14}$$

$$\text{Or}\ \ \Delta\, w_{gp} = \int_0^L \left(\frac{\sigma_z}{D_{gp}} \right) dz$$

Substituting for σ_z from Eqn. 9, one gets

$$\Delta\, w_{gp} = \int_0^L \left(\frac{q_{gp}\ R_{gp}}{D_{gp}} \right) dz$$

$$\Delta\, w_{gp} = \int_0^L \left(\frac{q_{gp}}{D_{gp}} \right) \exp\!\left(\left(\frac{4}{d} \right) K\ \tan\delta\ z \right) dz$$

$$= \left(\frac{q_{gp}}{D_{gp}} \right) \left[\frac{\exp(4K\tan\delta) \left(\frac{z}{d_0} \right) \left(\frac{d}{d_0} \right)^{-1}}{4 \left(\frac{K}{d_0} \right) \tan\delta \left(\frac{d}{d_0} \right)^{-1}} \right]_0^L$$

$$\Delta\, w_{gp} = (q_{gp} / D_{gp})\ [\, (d/d_{0)}\ d_o\ (\, R_{gp} - 1) / t \,] \tag{15}$$

where

$R_{gp} = \exp\,(t\ (L/d_0) / (d/d_0)\ )$, $t = 4\ K \tan\delta$, $D_{gp,}$ - the constrained modulus $= [E_{gp}\ (1-\nu_{gp}) / (1+\nu_{gp})(1-2\ \nu_{gp})] = \beta\ E_{gp}$ 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 the base, and is obtained as

$$w_{gp,0} = w_{s,L} + \Delta w_{gp} \tag{16}$$

$$= q_{gp}\left(\frac{R_{gp}}{k_{s,L}}\right) + \left[\left(\frac{q_{gp}}{D_{gp}\, t}\right)\left(d_r\, d_0\, (R_{gp} - 1)\right)\right] = q_{gp} f_1 \tag{17}$$

where

$$f_1 = \left(\frac{R_{gp}}{k_{s,L}}\right) + \left[\left(\frac{1}{D_{gp}\, t}\right)\left(d_r\, d_0\, (R_{gp} - 1)\right)\right] \tag{18}$$

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

$$w_{st} = w_{gp,0} \tag{19}$$

As per Scott (1981), the shear stress on the outer surface of steining, τ, is related to the displacement, w_{st}, (Figure 4), as

$$\tau = k_{st,s}.w_{st} = k_{st,s}\, w_{gp,0} \tag{20}$$

and

$$q_{st,b} = k_{st,b}..w_{st} = k_{st,b}\, w_{gp,0} \tag{21}$$

where the spring constant, $k_{st,s}$ for the shaft resistance is related to base stiffness $k_{st,b}$ (Scott, 1981) as $k_{st,s} = \alpha_\tau k_{st,b}$ where α_τ 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 another constant of proportionality. Substituting for all the terms in Eqn. 4, one gets

$$q = 4.(L/d_0)\, k_{sts}\, w_{st} + [\,(1 - (d^2/d_0^2)]\, k_{stb}\, w_{st} + (d^2 / d_0^2)\, q_{gpL} \tag{22}$$

or

$$q = w_{st}\, [4\ (L/d_0)\, k_{sts} + [\,(1 - (d^2/d_0^2))\, k_{stb}\,] + (d^2 / d_0^2)\, q_{gpL} \tag{23}$$

or

$$q = w_{st}\, f_2 + q_{gp}\, f_3 \tag{24}$$

where

$$f_2 = 4\left(\frac{L}{d_0}\right) k_{sts} + \left[1 - \left(\frac{d^2}{d_0^{\,2}}\right)\right] k_{stb} \quad \text{and} \quad f_3 = \left(\frac{d}{d_0}\right)^2 R_{gp}$$

Substituting for the value of w_{st} from Eqns. 17 & 19, one gets or

$$q = q_{gp}\,(f_1 f_2 + f_3) \tag{25}$$

where

$$F = f_1 f_2 + f_3 = \left(\frac{R_{gp}}{k_{sL}} + \frac{d(R_{gp}-1)}{D_{gp}\ t}\right) x \left(4\ \left(\frac{L}{d_0}\right) k_{sts} + \left[1 - \left(\frac{d}{d_0}\right)^2\right] k_{stb}\right) + \left(\frac{d}{d_0}\right)^2 R_{gp} \tag{26}$$

Substituting the values of Ksts & Kstb in terms of KsL, one gets

$$F = \left(\frac{R_{gp}}{k_{sL}} + \frac{d(R_{gp}-1)}{D_{gp}\ t}\right) \left(4\ \left(\frac{L}{d_0}\right) \alpha_\tau\, \alpha_b\ k_{sL} + \left[1 - \left(\frac{d}{d_0}\right)^2\right] \alpha_b\, k_{sL}\right) + \left(\frac{d}{d_0}\right)^2 R_{gp}$$

$$= \left(4\ \left(\frac{L}{d_0}\right) \alpha_\tau + \left[1 - \left(\frac{d}{d_0}\right)^2\right]\right) \left(\left(\alpha_b R_{gp}\right) + \frac{(R_{gp}-1)\alpha_b}{R\ t}\right) + \left(\frac{d}{d_0}\right)^2 R_{gp} \tag{27}$$

Eqn. 25 may be written as

$$q = q_{gp}\, F \tag{28}$$

or the force transmitted to the granular core is

$$Q_{gp} = (Q / F)\ \ d_r^2 \tag{29}$$

And the fraction of the load transferred to the steining becomes

$$(Q_{st}/Q) = [1 - (Q_{gp}/Q)] \tag{30}$$

where

$F = f_1\, f_2 + f_3$

$= [4\alpha_\tau\, (L/d_0) + (1 - (d/d_0)^2)]\ [\{\alpha_b\, R_{gp} / I_f\} + \{\alpha_b\, (d/d_0)\, (R_{gp} - 1) / Rt\}] + (d/d_0)^2 R_{gp}$

$R = (D_{gp} / k_{s,L}\, d) = \beta^* (E_{gp} / E_s)$, is the relative granular core stiffness

and $\beta^* = (1 - \nu_s^2)\, \beta$.

Combining Eqs. 17 and 19, and normalizing, w_{st}, the settlement of the composite foundation is

$w_{st} = q_{gp}\, f_1$

$$= q_{gp} \left(\frac{R_{gp}}{k_{sL}} + \frac{d(R_{gp}-1)}{D_{gp}\ t}\right)$$

Substituting the value of q_{gp} from Eqn. 28, one may write

$$w_{st} = \left(\frac{q}{F}\right)\left(\frac{R_{gp}}{k_{sL}} + \frac{d(R_{gp}-1)}{D_{gp}\ t}\right)$$

$$= \left(\frac{4.0\,Q}{\pi\, d_0^2\, F}\right)\left(\left(R_{gp}\right) + \frac{(R_{gp}-1)}{t}\left(\frac{d\ k_{sL}}{D_{gp}}\right)\right)$$

$$= \left(\frac{4.0\,Q}{\pi\, d_0^2}\right)\left(\frac{1}{F}\right)\left(\frac{d(1-\nu_s^2}{E_s}\right)\left(\left(R_{gp}\right) + \frac{(R_{gp}-1)}{R\ \ t}\right)$$

$$\frac{w_{st}\ d_0\ E_s}{Q} = \left(\frac{1}{F}\right)\left(\frac{4.0\, d_r\,(1-\nu_s^2)}{E_s}\right)\left(\left(R_{gp}\right) + \frac{(R_{gp}-1)}{R\ \ t}\right) \tag{31}$$

From Eqn. 13, the settlement of the soil below the granular core, $w_{s,L}$

$$w_{s,L} = \left(\frac{q_{gpL}}{K_{sL}}\right) = \left(\frac{q_{gp}\ R_{gp}}{k_{sL}}\right) = \left(\frac{4.0\,Q}{\pi\, d_0^2}\right)\left(\frac{R_{gp}}{F}\right)\left(\frac{d\,(1-\nu_s^2)}{E_s}\right)$$

or

$$\frac{w_{sL}\ d_0\ E_s}{Q} = \left(\frac{R_{gp}}{F}\right)\left(\frac{4.0\left(d/d_0\right)(1-\nu_s^2)}{E_s}\right) \tag{32}$$

Results and Discussion

Results have been obtained for a range of parameters to illustrate their influences on the settlement of the proposed composite foundation with granular core. The typical values of the parameters considered in this study are given in Table 1.

Table 1 Typical Values of Parameters Considered

Parameters	Assigned value /range
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 base stiffness (α_b)	0.25, 1.0 and 4.0
Relative steining- soil interface / base stiffness ($\alpha_{\square}$)	0.25, 1.0 and 4.0
Diameter ratio, d/d_0	0.65 – 0.95
Length to diameter ratio, L/d_0	0.50 - 3.00
Angle of shearing resistance of granular core, ϕ_{gp}	10^0 - 40^0
Coefficient of lateral pressure, K	K_0 (at rest) – 2.0 K_0

In the following sections, results are presented and discussed for the effects of length to diameter ratio (L/d_0), diameter ratio (d/d_0), Poisson's ratio of soil (ν_s), modular ratio (E_{gp}/E_s), relative steining base stiffness (α_b), relative steining-soil interface stiffness (α_τ), angle of shearing resistance of granular core (ϕ) and the coefficient of lateral pressure (K) on the settlement response of the composite foundation.

The reduction of normalized settlement of the composite foundation ($w_{st}.E_s.d_0/Q$) with increasing length of the composite foundation, l_r ($=L/d_0$) and decreasing values of the diameter ratio, d_r ($=d/d_0$), for ν_s = 0.5, α_b = 1.0, $\alpha_{\square}$ = 0.25 and E_{gp} / E_s = 10.0 is shown in Figure 5. For L/d_0 = 0.5, the normalized settlements of the composite foundation decrease from 0.62 for d/d_0 = 0.95 to 0.43 for d/d_0 = 0.65. Similar trends are observed for all values of L/d_0. The normalized settlement of the composite caisson decreases with increase in L/d_0 for all ratios of (d/d_0), while it increases with increase in d/d_0 for all L/d_0. The shaft surface area becomes larger with increasing length of the proposed foundation, and the settlements decrease as is the case with solid piles (Poulos and Davis 1980).

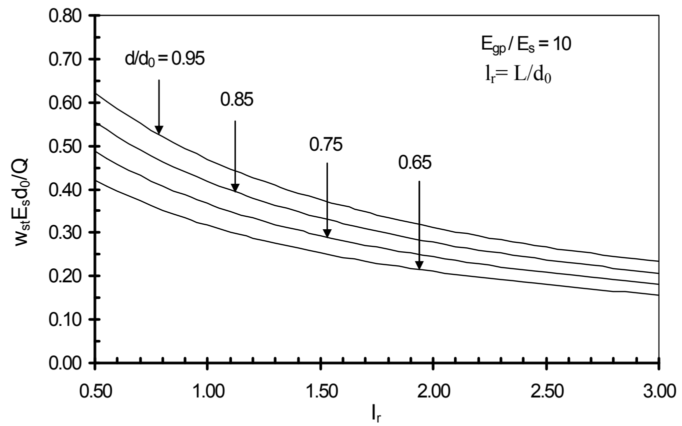

Fig. 5 Variation of Normalized Settlement of the Composite Foundation with L/d_0

The variation of the normalized settlement of the ground beneath the granular core, ($w_{s,L}.E_s.d_0/Q$) with L/d_0 for ν_s = 0.5, α_b = 1.0, α_τ = 0.25 and E_{gp} / E_s = 10 is presented in Figure 6. The normalized settlement beneath the granular core decreases with increase in the L/d_0 and the decrease of d/d_0, For a constant L/d_0 = 0.5, the normalized settlements of the soil beneath the granular core are 0.59, 0.52, 0.45, and 0.4 for d/d_0 = 0.95, 0.85, 0.75 and 0.65 respectively. A decrease in diameter ratio (d/d_0) implies an increase in base area of steining. Since steining is stiffer than the core, it will carry more load leading to reduction in settlement.

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 stiffness 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 variations of the tip settlements of the steining and the granular core with L/d_0 are depicted in Figure 7. The difference, Δw_{gp}, between these two settlement curves is due to the compressibility of the granular core. The difference in the settlements is significant for a thin steining (d/d_0 = 0.95) and negligible for a thick (d/d_0 = 0.65) one. A thin steining undergoes larger settlement compared to a thick one because of smaller base area and base resistance and hence the large difference in settlements of the steining and the core.

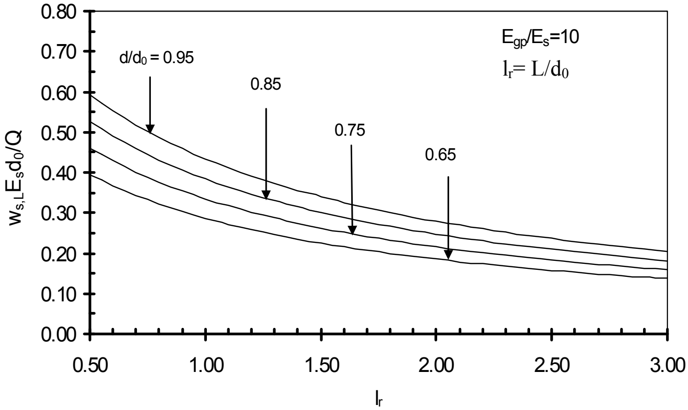

Fig. 6 Variation of Settlement of Granular Core at the Base with (L/d_0) and (d/d_0)

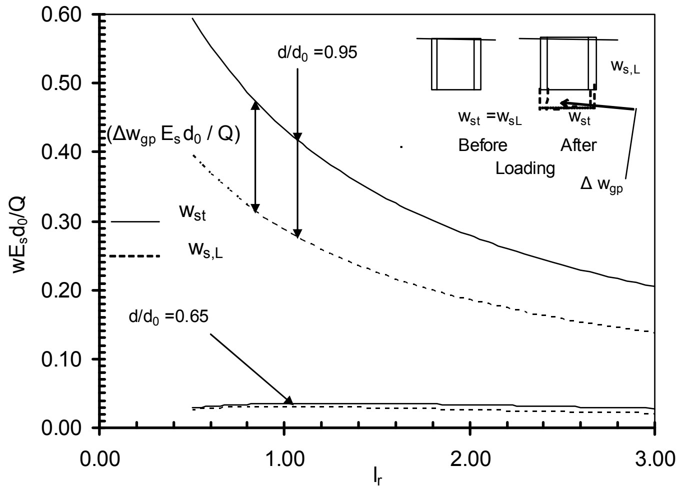

Fig. 7 Variation of Settlement Difference (w E_s d_0 /Q) with (L/d_0) and (d/d_0)

Figure 8 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_b = 1.0, α_τ = 0.25 and d/d_0 =0.65. An increase in modular ratio from 10 to 100, leads to only a small decrease in the settlement of composite foundation due to large percentage of the load applied being carried by steining and low percentage of the same carried by the core. For L/d_0 = 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/d_0. Thus, the effect of modular ratio on the overall settlement of composite foundation is relatively insignificant. Desai and Chandrasekharan (1986) report similar result for short concrete caissons (E_{gp}/E_s = 1.0, d/d_0= 1.0).

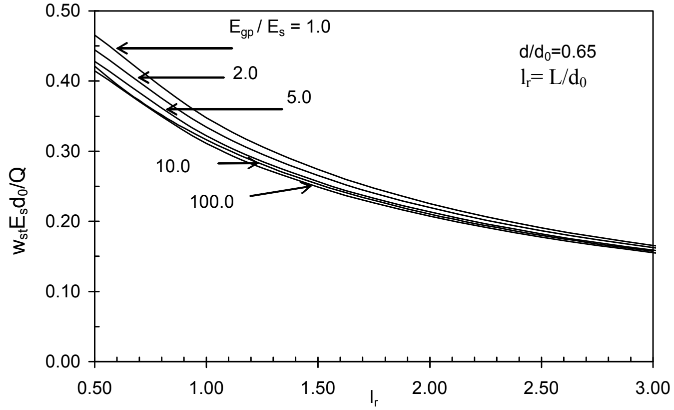

Fig. 10 Effect of Modulus Ratio (E_{gp} / E_s) on Settlement of Composite Foundation

The normalized settlement of the proposed foundation / steining decreases with increase in relative steining base / granular core stiffness (α_b) as shown in Figure 9. For E_{gp}/E_s = 10.0, ν_s = 0.5, α_τ= 0.25, d/d_0 = 0.95 and L/d_0 = 0.50, the normalized settlements of the 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. A similar trend is observed for d/d_0 = 0.65. 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, a reduction in settlement.

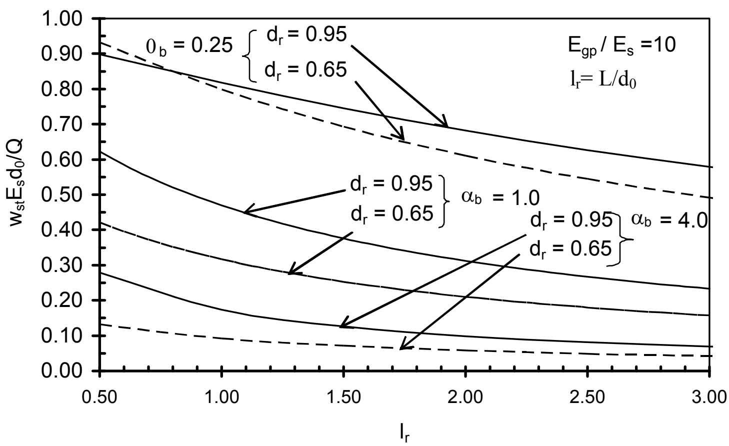

Fig 9 Effect of α_b on Settlement of Steining / Composite Foundation

The normalized settlement of the proposed foundation decreases with increase in relative stiffness (α_τ) of steining-soil interface surface with respect to base as shown in Figure 10. For E_{gp}/E_s = 10.0, ν_s = 0.5, α_b = 1.0, d/d_0 = 0.95 and L/d_0 = 0.50, the normalized settlements of foundation are 0.65, 0.30 & 0.10 at relative steining-soil interface stiffness to base stiffness (α_τ) of 0.25, 1.0 and 4.0 respectively. The trend is similar for d/d_0 = 0.65. Once again, an increase in α_τ reflects a stiffer steining base and hence, to an increase in the load carried by the well steining and consequent reduction in settlement.

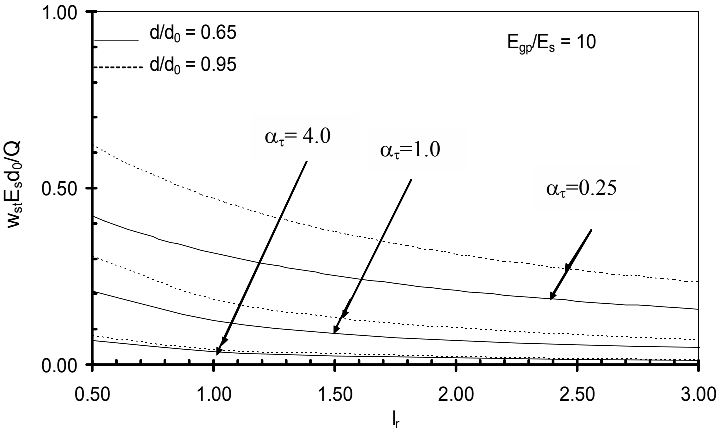

Fig. 10 Effect of α_τ on Settlement of Steining / Composite Foundation

The normalized settlement of composite foundation is almost independent of the angle of shearing resistance of the granular soil (ϕ) for a constant L/d_0 and d/d_0 (Figure 11).

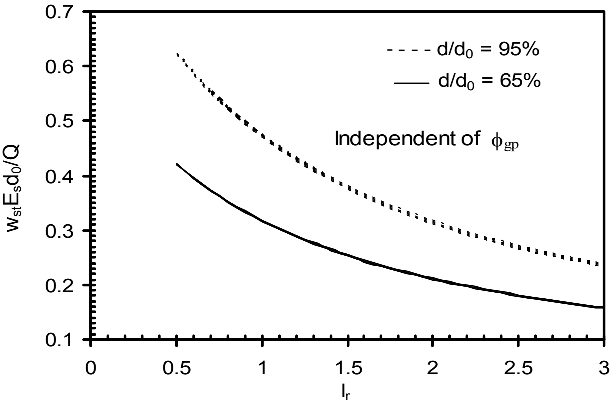

Fig. 11 Effect of Angle of Shearing Resistance ϕ_{gp} on Settlement of Composite Foundation

The settlement of foundation/steining is less for d/d_0 = 65% in comparison with that for d/d_0 = 95% for all L/d_0, and is almost independent of ϕ.

On similar lines, the settlement of the foundation/steining is almost independent of the variation in coefficient of lateral pressure (K) for all d/d_0. Also for a constant L/d_0, the settlement of the steining decreases with decrease in d/d_0 (Figure 12). The effect of K on compression of the granular core is almost negligible for given L/d_0 and d/d_0 ratios.

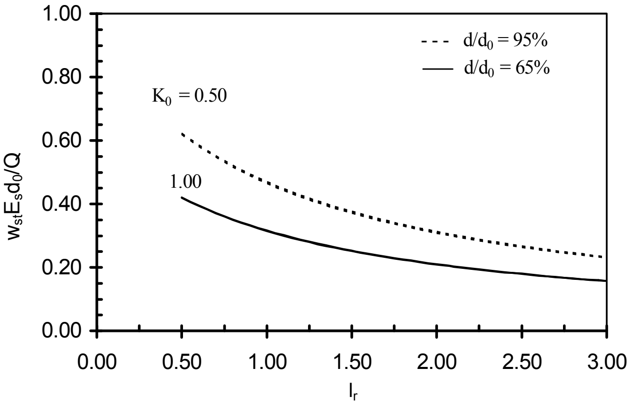

Fig. 12 Effect of Coefficient of Lateral Pressure, (K_0) on Normalized Settlement

Effects of Poisson's ratio of the soil on normalized settlement is depicted in Figure 13. It is evident from the figure that settlement decreases slightly from 0.51 to 0.45 for Poisson's ratio increasing from 0.3 to 0.5 for E_{gp}/E_s = 10.0, α_b = 1.0, (d/d_0) = 0.95 and (L/d_0) = 0.50. Similar trend is observed for (d/d_0) = 0.65. Higher Poisson's ratio, υ_s, corresponds to stiffer condition and to a reduction in settlement.

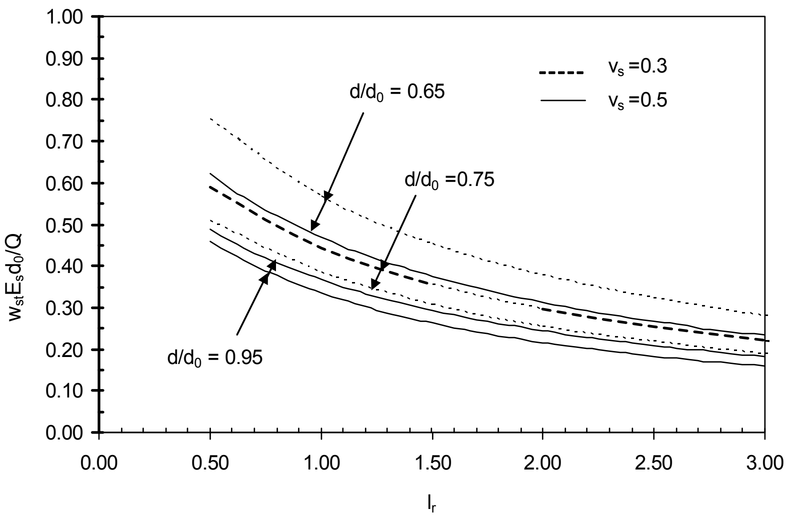

Fig. 13 Effect of Poisson's Ratio of Soil on Settlement of Steining /Composite Foundation

Effect of Poisson's ratio of granular core, ν_{gp}, on normalized settlement is depicted in Figure 14. It is evident from the above figure that Poisson's ratio, ν_{gp}, has almost no influence on the settlement. Alamgir et al. (1994) report a similar result for a granular pile.

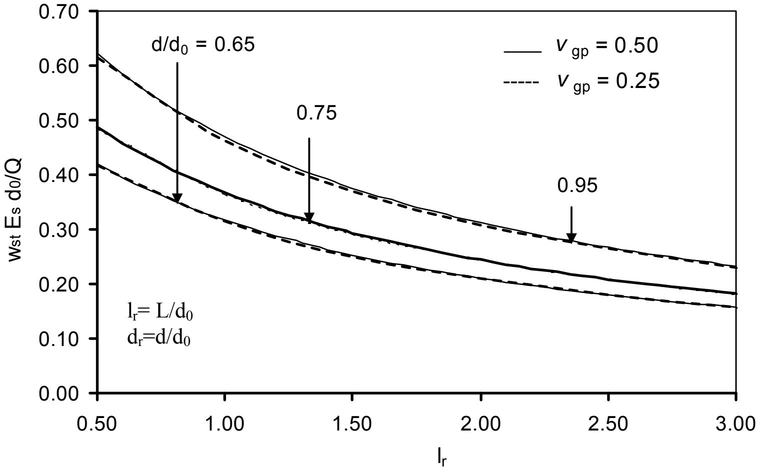

Fig 14 Effect of Poisson's Ratio of Granular Core on Settlement of Steining/Composite Foundation

Experimental Study

The predictive power of the numerical models will be enhanced by comparison of predictions with field measurement or with data from carefully conducted physical modeling (Chandrasekaran, 2000). Full scale field tests do provide valuable and reliable information, but are very expansive. To verify the predicted behaviour of piles, model pile load tests have been normally carried out by various researchers (Cooke and Whitaker, 1961, Alampalli and Peddibotla, 1997, Dickin and Nazir, 1999, Sastry and Meyerhof, 1999). Although many uncertainties are associated with small scale model tests, they are useful for validating the predictions, at least qualitatively. Few tests conducted on small scale models of proposed rigid composite caisson foundations in the laboratory are reported here in.

All model tests were conducted in soil samples prepared within a model testing system. Locally available sub-angular quartz sand was used for all the laboratory model tests since its behaviour is free from time effects, densities are reproducible in the laboratory and due to its easiness in handling. The model testing system consists of a tank of size 1.50 m x 1.50 m x 1.50 m, made with 10.0 mm thick mild steel plates, fixed by bolts and nuts arrangement to the angle iron frame of the same dimensions, loading and data acquisition systems. Two I-girders and a beam are provided in the model testing system for maintaining its stability. The position of hydraulic jack on the top can be adjusted with the help of grips provided on the beam. Displacement sensors (LVDT's) are fixed on beam with the help of four LVDT holders. One side of the tank is made of thick Perspex (plaxi glass) and could be opened by removing the plate of that side, to make it easy to empty the sand. The dimensions of the tank were decided with the consideration of failure surface getting fully mobilised without being restricted by the tank size and from considerations of the volume of soil to be handled to fill the tank each time. Epoxy adhesive was applied along all joints on the inner sides of the tank to prevent any leakage. Loading is applied with the help of a 200 kN capacity hydraulic jack connected to hydraulic pump with a hydraulic hose. Load is measured by a loading cell of 200 kN capacity and four displacement sensors of $\pm$ 20 mm travel were employed to measure the corresponding displacement. Data acquisition system with 16 channel data logger is used to record the load and displacement data (Figure 15).

Fig 15 Photograph Showing Laboratory Testing of Composite Foundation

Models of composite short caisson foundations of inner to outer diameter (d/d_0) ratios of 0.65 and 0.95 and length to outer diameter (L/d_0) ratios of 1.0, 1.5 and 2.0 respectively were cast in the laboratory (Table 2). After casting, they were cured in a water tank. In order to facilitate the sinking of the model caisson foundation, sand was scooped out or excavated and removed from the inside. Additional weights were applied on to the caisson to overcome side friction and expedite the sinking. After being sunk to its final stage, granular material of size 20 mm was placed inside the caisson in layers, each layer being compacted with 25 blows of a 2.6 kg (0.025 kN) rammer dropped from a height of 310 mm above the granular layer (compaction energy = 6.045×10^4 kg-m/m^3) to achieve maximum dry density.

A circular rigid metal plate of diameter equal to the outer diameter, d_0, of the model is placed on top of the composite foundation. A load cell is placed between the rigid metal plate and the hydraulic jack. Loads are applied in small increments and each increment lasting till the rate of settlement was practically zero. Failure was indicated by continuous caisson movement. The typical Schematic Diagram of test set-up and model composite foundation are given in Figure 16.

Table 2 Ultimate Load and Normalized Settlement from Model Tests

S. No.	*L/d_0*	*d/d_0*	*Normalized Settlement, ($w_{st}\, E_s\, d_0/Q$)*
1	2.0	0.65	0.248
2	1.5	0.65	0.287
3	1.0	0.65	0.438
4	2.0	0.95	0.340
5	1.5	0.95	0.432
6	1.0	0.95	0.650

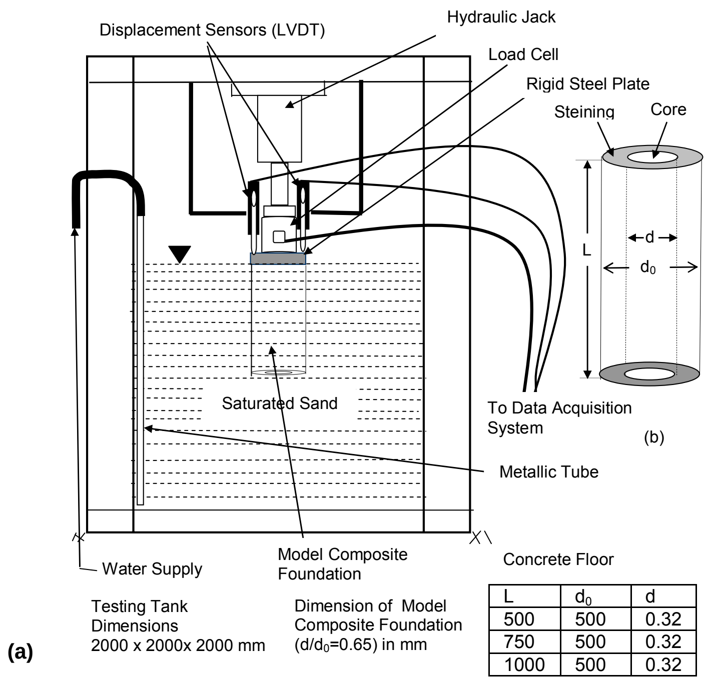

Fig. 16 Schematic Diagram of (a) Test Set-Up (b) Model Composite Foundation

Load-settlement responses of model caisson foundations with different length to diameter ratio, L/d_0, and diameter ratio, d/d_0, have been obtained experimentally. Normalized settlements ($w_{st}.E_s.d_0/Q$) were obtained from the above mentioned data for different L/d_0 and d/d_0 ratios as shown in Table 2. The measured and predicted displacements are plotted for comparison in Figure 17. The agreement is generally good. However, the measured settlements for caissons having L/d_0 ratios greater than 1.5 are less than the predicted ones probably due to the fact that modulus of the soil increases with depth.

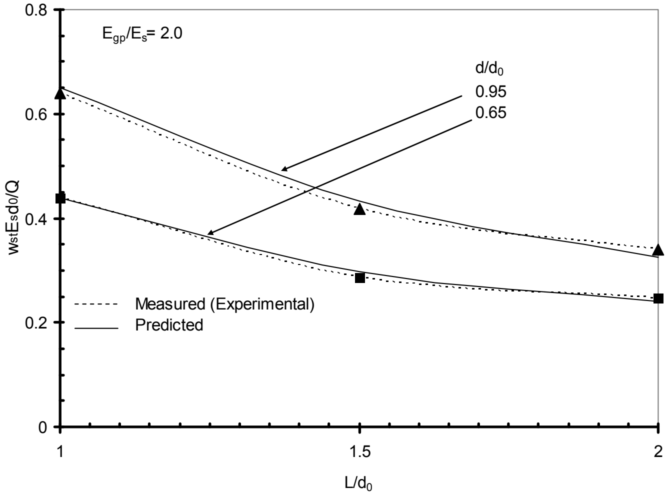

Fig. 17 Measured and Predicted Settlements of Composite Foundation

Concluding Remarks

A new composite foundation with granular core is proposed for loose alluvial deposits with high water table. A simple theoretical approach using Winkler type responses, i.e. linear shear stress- displacement relationship for the outer surface of the composite foundation and linear bearing stress – displacement relationships for the steining and the base of the granular core, 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), relative steining surface to base stiffness (α_τ), Poisson's ratios of soil (υ_s) and of granular core (υ_{gp}) on the sharing of the applied load by the well steining and the granular core as well as on the settlement of the proposed foundation.

The normalized settlement (w_{st} E_s d_0/Q) of the proposed foundation decreases with increase in the length to diameter ratio (L/d_0) at constant diameter ratio (d/d_0). In addition, the normalized settlement of the granular core, ($w_{gp}E_sd_0/Q$), at the base decreases with increase in L/d_0. The load carried by the steining (Q_{st}/Q_{ult}) and the settlement decrease with increase in the modular ratio E_{gp}/E_s. Model tests conducted for the validation of the theoretical data show reasonable agreement between measured and predicted settlements.

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