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Continuum approach for analysis of short composite caisson foundation

Approche de continuum pour l'analyse de la base composée courte de caisson

S.M. Ali Jawaid
Department of Civil Engineering, M. M. M. Engineering College, Gorakhpur – 273 010, India
Madhira R. Madhav
Department of Civil Engineering, Jawaharlal Nehru Technological University, Hyderabad – 500034, India

ABSTRACT
This study is an attempt to analyze the load – settlement behavior of short composite caisson foundation using the continuum approach. The steining considered to be rigid, undergoes rigid body translation and hence is treated as an incompressible cylinder while the core inside is analyzed as a compressible pile. Only compatibility of vertical displacements is considered in this analysis. The soil displacements are calculated at mid points of the outer surfaces of each element, and at the centers of bases of steining and granular core by integrating numerically the Mindlin's solution for a vertical point load inside a semi-infinite medium. A detailed parametric study is carried out to evaluate the relative influence of each parameter on the overall deformation response of the composite foundation. The results encourage the use of the above foundation for alluvial lowlands.

RÉSUMÉ
Cette étude est une tentative d'analyser la charge - comportement de règlement de base composée courte de caisson en utilisant l'approche de continuum. Steining considéré comme rigide, subit la traduction rigide de corps et par conséquent est traité comme cylindre incompressible tandis que l'intérieur de noyau est analysé comme pile compressible. Seulement la compatibilité des déplacements verticaux est considérée dans cette analyse. Les déplacements de sol sont calculés à de mi points des surfaces externes de chaque élément, et aux centres des bases du noyau steining et granulaire en intégrant numériquement la solution du Mindlin pour une charge verticale de point à l'intérieur d'un milieu semi-infinite. Une étude paramétrique détaillée est effectuée pour évaluer l'influence relative de chaque paramètre sur la réponse globale de déformation de la base composée. Les résultats encouragent l'utilisation de la base ci-dessus pour les terres en contre-bas alluviales.

1 INTRODUCTION

Short composite caisson foundation with granular core is being developed for alluvial lowlands (Madhav & Jawaid 2003). It consists of shallow pipes or well steinings (outer diameter, d_0 = 1.0 to 1.5 m, thickness of steining, t = 10 – 15 cm and length, L = 1.5 – 4.0 m) with compacted granular core inside. It is sunk to the desired depth by conventional sinking techniques. Soil within the steining is removed and granular material filled in and compacted in layers to enhance the stiffness and thereby the stability and the load carrying capacity of the proposed composite foundation.

The steining is considered to be rigid, undergoes rigid body translation and hence is analyzed as an incompressible cylinder while the core inside is treated as a compressible pile. The base is considered to be made of rigid circular steining of outer diameter, d_0, and inner diameter, d, and the granular core of diameter d is considered to be compressible but semi-rigid. Shear stresses along the outer surface of the steining and pressures on the steining and granular core bases are the mobilized stresses. Only compatibility of vertical displacements is considered in this analysis. The soil displacements are calculated at mid points of the outer surfaces of each element, and at the centers of bases of steining and granular core by integrating numerically the Mindlin's (1936) solution for a vertical point load inside a semi-infinite medium. Poulos and Davis's (1968) approach for incompressible pile is adopted to evaluate the caisson's displacements.

2 ANALYSIS

A circular composite foundation of length, L, outer diameter of steining, d_0, and diameter of granular core, *d,* is selected as shown in Fig. 1. The moduli and the Poisson's ratios of soil and

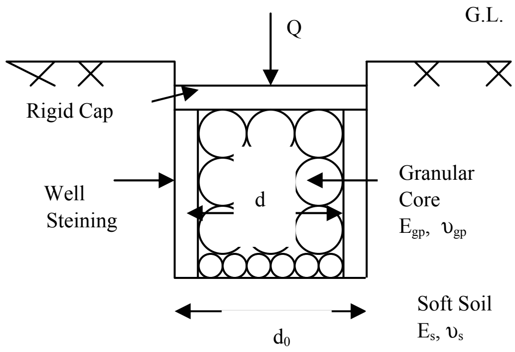

Fig. 1. Short Composite Caisson Foundation with granular core

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.

A solid pile, a plugged caisson or a well foundation transfers 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 on the other hand mobilizes positive skin resistance on both its inner and outer surface since the soil inside the pipe is soft and highly compressible. The proposed composite foundation functions similar to a short pipe pile except that the granular in fill is much stronger and stiffer than the original ground. Hence, it can withstand part of the applied load. 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 its base. Granular material, if confined, deforms one dimensionally and becomes stiffer with increasing confining stress.

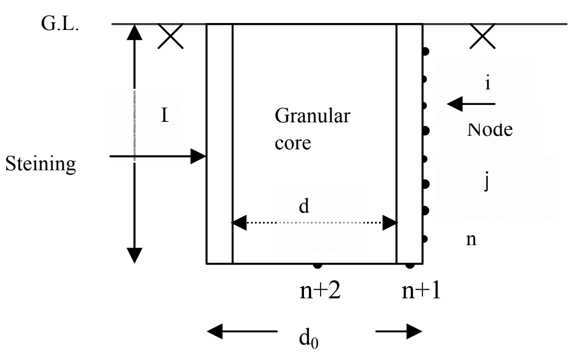

Fig. 2. Discretization Scheme of proposed composite foundation

The periphery of composite foundation is divided into n equal cylindrical element, with a typical element, *j,* being acted upon by a uniformly distributed vertical shear stress, p_j, The base elements considered to be made of rigid circular steining of outer diameter, d_0, and inner diameter, d, and a compressible core (granular) of diameter, d, are represented by nodes *n+1* and *n+2* respectively. The necessary condition for the solution of the problem is that at every point along the composite short caisson, the displacements of the soil adjacent to the caisson must be compatible with those of the caisson itself. It has been reported that in case of piles, consideration of both vertical and radial displacements leads to solutions which are insignificantly different from those obtained by considering only vertical displacements (Mattes, 1969). Therefore, only compatibility of vertical displacements is considered in this analysis. These displacements are calculated at mid points of the outer surfaces of each element, and at the centers of bases of steining and granular core. The detail of discretization is shown in Fig. 2. Referring to the above mentioned figure and assuming downward vertical displacement as positive, the displacement at *i* due to shear stress p_j on element j may be expressed as

$$_s\rho_{ij} = (d_0/E_s)\, I_{ij}\, q_j \quad \ldots\ldots (1)$$

where I_{ij} = the influence coefficient for the displacement at i due to shear stress q_j on element *j*.

The displacement, $_s\rho_i$ at node, *i* due to shear stresses on all n elements of the periphery of the composite foundation, plus the normal stresses on the base (steining & granular core), is

$$_s\rho_i = (d_0/E_s)\left\{ \sum_{j=1}^{n} I_{ij}\, q_j + I_{i\,n+1}\, q_{stb} + I_{i\,n+2}\, q_{gpb} \right\} \ldots\ldots (2)$$

where $I_{i\ n+1}$ & $I_{i\ n+2}$ are the influence coefficients for displacements at *i* due to a vertical stresses on the base of rigid steining and granular core base respectively. The coefficients are obtained by double integration of the Mindlin's solution.

Similarly, the displacement, $_s\rho_{n+1}$, of the soil beneath the base of steining may be expressed as

$$_s\rho_{n+1} = (d_0/E_s)\left\{ \sum_{j=1}^{n} I_{n+1\,j}\, q_j + I_{n+1\,n+1}\, q_{stb} + I_{n+1\,n+2}\, q_{gpb} \right\} \ldots (3)$$

The displacement, $_s\rho_{n+2}$, of the soil beneath the base of granular core may be expressed as

$$_s\rho_{n+2} = (d_0/E_s)\left\{ \sum_{j=1}^{n} I_{n+2\,j}\, q_j + I_{n+2\,n+1}\, q_{stb} + I_{n+2\,n+2}\, q_{gpb} \right\} \ldots (4)$$

Thus, the displacement of all elements on the steining may be conveniently grouped in matrix form as

$$[_s p] = (d_0/E_s)\, [_s I]\, [q] = w_{st}\, [1] \ldots\ldots (5)$$

where

$\{_s\rho\}$ = soil displacement vector

$$[q] = \text{pile stress vector} = \begin{pmatrix} \tau_1 \\ \tau_2 \\ . \\ . \\ . \\ q_{st,b} \\ q_{gp,b} \end{pmatrix} \quad \text{and}$$

$$[_s I] = \begin{pmatrix} a_{11} & a_{12} & \ldots & a_{1n} & a_{1gpb} \\ a_{21} & a_{22} & \ldots & a_{2n} & a_{2gpb} \\ \ldots & & & & \\ \ldots & & & & \\ \ldots & & & & \\ a_{n+1\,1} & a_{n+1\,2} & \ldots & a_{n+1\,n} & a_{n+1gpb} \end{pmatrix} \quad (7)$$

$[_s I]$ is (n+1) by (n+1) matrix of soil displacement influence factor, all elements of this matrix were obtained by double integration of Mindlin solution. Details of the relevant integration are given elsewhere (Poulos & Davis, 1968).

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 vertical equilibrium of an element of the granular core (Fig.3), neglecting its weight, is

$$(\sigma_z + \Delta\sigma_z)(\pi/4)\, d^2 - \sigma_z\,(\pi/4)\, d^2 - \tau_{gp}\,(\pi\, d\ \Delta\sigma_z) = 0 \quad (8)$$

$$\text{or} \quad (d\sigma_z / dz) - (4/d)\,\tau_{gp} = 0 \quad (9)$$

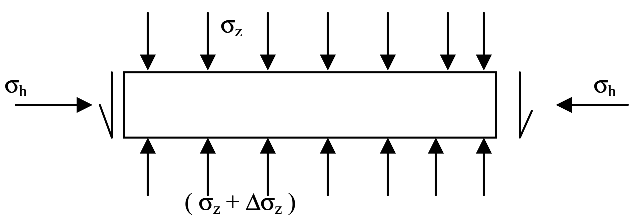

Fig. 3. Stresses acting on an element of granular core

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

$$\sigma_z = c_o \exp(c_1 z) \quad (10)$$

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, $\sigma_{z,L}$, to the soil below i.e. at z = L becomes

$$\sigma_{z,L} = q_{gp}\ d_1 \qquad (11)$$

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

$$w_{st,L} = \sigma_{z,L}\ [\ d\ (1-\nu_s^2)\ C/\ E_s\ ] = [\ q_{gp}\ d_1\ /\ (k_{s,L}\ I_f\ )] \qquad (12)$$

where I_f = an influence factor (Fox 1948), 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

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

or $\Delta w_{gp} = (q_{gp}\ /\ D_{gp})\ [(d/d_0)\ d_0\ (d1 - 1)\ /\ t]$

or $\Delta w_{gp} = (q_{gp}\ d_0/\ D_{gp})\ [(d/d_0)\ (d1 - 1)\ /\ t]$ (13)

where d_1 = exp (t (L/d_0) / (d / d_0)), t = 4 k tanδ, the constrained modulus $D_{gp} = [E_{gp}\ (1 - \nu_{gp})\ /\ (1+ \nu_{gp})\ (1- 2\ \nu_{gp})] = \beta\ E_{gp}$ and $\beta = (1 - \nu_{gp})\ /\ [(1+ \nu_{gp})\ (1- 2\ \nu_{gp})]$

Equation 13 may be rewritten as

or $\Delta w_{gp} = (q_{gp}\ d_0/\ E_{gp})\ C_{gp}$ (14)

where $C_{gp} = [(d/d_0)\ (d1 - 1)\ /\ \beta\ t]$

Now, the displacement of n +2 element may be conveniently write in matrix form as

$[_s\rho] = (d_0/E_s)\ [_sI]\ [q] = w_{st}[1] - \Delta w_{gp}$ ………………. .(15)

$[_s\rho] = (d_0/E_s)\ [_sI]\ [q] = w_{st}[1] - (q_{gp}\ d_0/\ E_{gp})\ C_{gp}$ …......(16)

The total load carried by the proposed foundation may be obtained

$Q = \int q\ dA$

$= (\pi\ dL/n)\sum \tau_i + q_{st}\ A_{st} + q_{gp}\ A_{gp}$ (17)

3 RESULTS AND ANALYSIS

The main objective of this analysis is to evaluate the vertical displacement of the composite granular core and to obtain the stress distribution along its periphery and base. Composite foundation with is discretized in to 8-10 elements for the purpose of numerical integration. Each element is further subdivided into 150 cylindrical sub-elements. Each cylindrical sub-element is divided into 150 angular subdivisions. The base of composite foundation is divided into 200 and 150 annular and angular sub-divisions respectively.

A parametric study is carried out in order to bring out the effects of length to diameter ratio, L/d_0, modular ratio, E_{gp}/E_s, and the diameter ratio, d/d_0. Following are the ranges of parameters considered.

Table 4.1 Typical values of parameters considered

S. No.	Parameter	Range/Assign Values
1.	Length to diameter ratio, L/d_0	1.0 - 3.0
2	Diameter ratio, d/d_0	0.65 - 0.95
3	Modular ratio, E_{gp}/E_s	1- 100
4	Poisson's ratio of soil, ν_s	0.50
5.	Poisson's ratio of granular core, ν_{gp}	0.25
6	Friction angle of soil, φ	30°

Poisson's ratios of soil (ν_s) and granular core (ν_{gp}) are reported to have negligible effect on load carrying capacity of the foundation as well as on its settlement (Alamgir, 1996, Madhav & Jawaid, 2003). Hence their variations are not considered in this analysis.

Fig. 4 shows that when the diameter ratio (d/d_0) < 0.95, only steining base ($Q_{st,\ b}$) and periphery ($Q_{st,s}$) carry the entire load. But for diameter ratio (d/d_0) ≥ 95%, the granular core base ($Q_{gp,b}$) and steining periphery ($p_{st,s}$) carry most of the load. This is due to the fact that as the thickness of the steining decreases, the load carried by its base decreases. The percent load carried by steining base ($Q_{st,b}$) decreases with increase in length ratio (L/d_0) as well. At the same time percent load carried by the steining surface ($T_{st,s}$) (Fig. 5) increases for a given diameter ratio (d/d_0). It is due to the fact that the surface area of the steining increases with increase in L/d_0, leading to an increase in the peripheral shear. Similar type of behaviour is reported by Poulos and Davis (1968) for the case of incompressible piles. Their results are also shown in Fig. 4 for ready reference. Also, for a given length to diameter ratio (L/d_0), the load carried by steining base increases with decrease in d/d_0 i.e. with increasing steining thickness. The percent loads taken by steining base for d/d_0 of 0.65 and 0.85 are approximately 74% and 44% respectively for L/d_0 = 1.0. It is due to an increase in base area of the steining and a corresponding decrease in the cross-sectional area of granular core. Similar trend was reported by Desai and Chandrasekharan (1985) for short concrete caissons (E_{gp}/E_s = 1.0, d/d_0 =1.0).

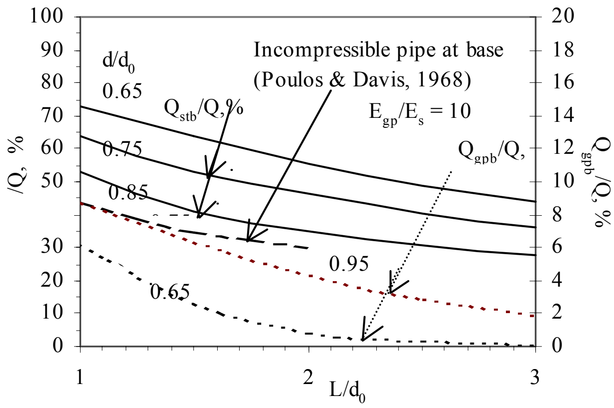

Fig. 4. Effect of L/d_0 and d/d_0 on percent load carried by steining base and core.

The typical distribution of shear stress along the steining surface is shown in Fig. 6 for L/d_0=1.0 to 3.0 and d/d_0=0.65. The shear stress decreases gradually from the top and reaches the minimum near the centre of the composite caisson. It again increases gradually up to z/L=0.8 and then decreases and shows the minimum value near the base. Similar results were reported for slender piles (Poulos and Davis, 1968).

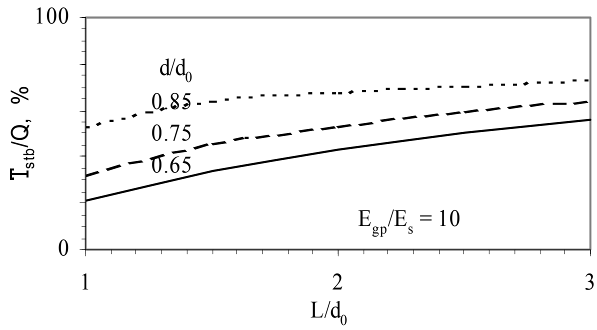

Fig. 5. Effect of L/d_0 and d/d_0 on the percent load carried by steining surface.

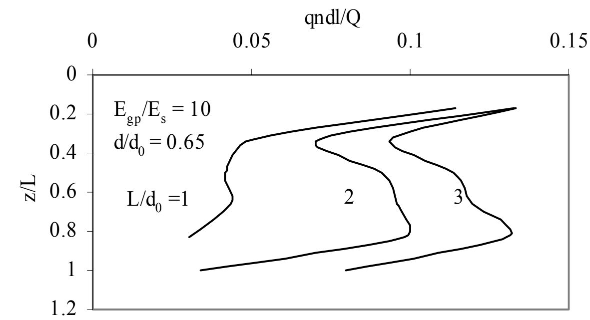

Fig. 6. Distribution of side shear along the steining surface

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 loads carried by steining base and steining surface for d/d_0=0.65 are depicted in Fig. 7. The percentage load carried by steining base and steining surface decrease with increase in modular ratio, E_{gp}/E_s. A higher modular ratio, E_{gp}/E_s reflects stiffer granular core and hence, less load is carried by the steining. The decrease is significant up to length to diameter ratio (L/d_0) of 2.50. Further, for $L/d_0 \geq$ 2.50, the percentage load carried by the steining decreases only marginally.

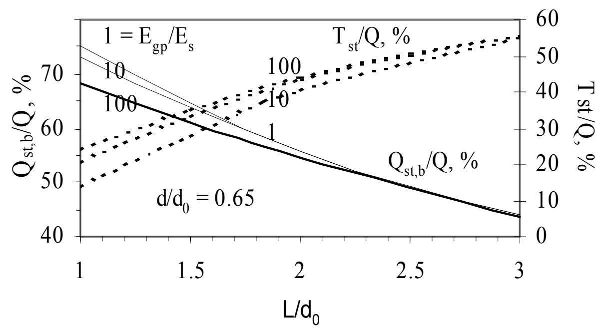

Fig. 7. Effect of modular ratio on percent load carried by steing base and surface.

The variation of normalized settlement of composite foundation (w_{st}) with L/d_0 for E_{gp} / E_s = 10.0 is shown in Fig. 8. For L/d_0 = 1.0, the normalized settlements of the composite foundation are 0.36, 0.34 and 0.33 for d/d_0 = 0.85, 0.75 and 0.65 respectively. Similar results are observed for all values of L/d_0. The normalized settlement of the composite foundation 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.

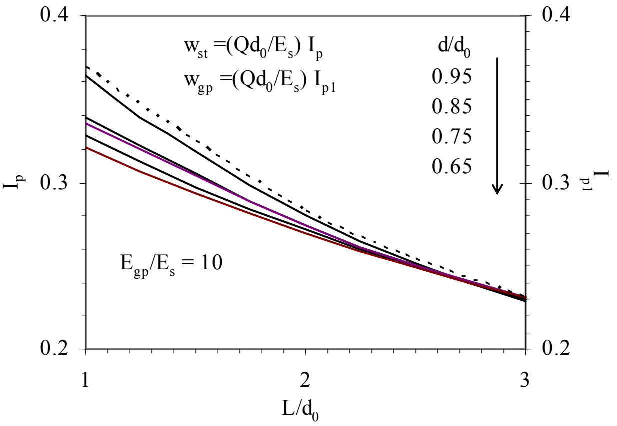

Fig.8. Settlement variation with L/d_0

Fig. 9 shows the effect of modular ratio (E_{gp}/E_s) on normalized settlement of composite foundation (w_{st}), for d/d_0 =0.65. For L/d_0 = 1.0, the normalized settlements of the composite foundation are 0.34, 0.32, and 0.31 at (Egp/Es) = 1.0, 10.0 and 100.0 respectively.

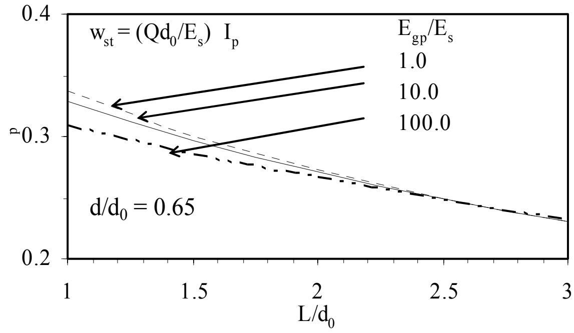

Fig. 9. Influence of modular ratio (E_{gp}/E_s) on settlement

4 SUMMARY AND CONCLUSIONS

A new short composite caisson foundation with granular core is proposed for alluvial lowlands and the load – settlement behavior of the same is analyzed using continuum approach (Mindlin, 1936). The parametric study quantifies the effects of length-diameter ratio (L/d_0), diameter ratio (d/d_0), modulus ratio (E_{gp} / E_s) and friction angle (ϕ) on the sharing of the applied load by the well steining and the granular core and on the settlement. The conclusions of practical significance drawn from this theoretical study are:

1. The amount of load transferred to the steining base of a single short composite caisson foundation decreases with increase in length-diameter ratio (L/d_0) and diameter ratio (d/d_0).
2. As the L/d_0 ratio increases to more than 95%, the load transferred to the base of short composite caisson foundation is only borne by steining.
3. The load carried by steining surface increases with increase in modular ratio (E_{gp}/E_s) at a constant diameter ratio (d/d_0).
4. The settlement of short composite caisson foundation decreases with increase in length – diameter ratio (L/d_0).

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