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Bearing capacity of a circular foundation on layered sand–clay media

Jyant Kumar*, Manash Chakraborty

Indian Institute of Science, Bangalore 560012, India

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Abstract

The bearing capacity of a circular footing lying over fully cohesive strata, with an overlaying sand layer, is computed using the axisymmetric lower bound limit analysis with finite elements and linear optimization. The effects of the thickness and the internal friction angle of the sand are examined for different combinations of $c_u/(\gamma b)$ and q, where c_u = the undrained shear strength of the cohesive strata, γ = the unit weight of either layer, b = the footing radius, and q = the surcharge pressure. The results are given in the form of a ratio (η) of the bearing capacity with an overlaying sand layer to that for a footing lying directly over clayey strata. An overlaying medium dense to dense sand layer considerably improves the bearing capacity. The improvement continuously increases with decreases in $c_u/(\gamma b)$ and increases in ϕ and $q/(\gamma b)$. A certain optimum thickness of the sand layer exists beyond which no further improvement occurs. This optimum thickness increases with an increase in ϕ and q and with a decrease in $c_u/(\gamma b)$. Failure patterns are also drawn to examine the inclusion of the sand layer.

Keywords: Clays; Failure; Finite element method; Foundations; Limit analysis; Optimization; Sand

1. Introduction

The bearing capacity solutions available in literature are generally meant for homogeneous soil deposits (Terzaghi, 1943; Meyerhof, 1963; Hansen, 1970; Vesic, 1973). In some cases, foundations need to be constructed over soft clay deposits for which the bearing capacity can be improved significantly by providing either stone columns or a layer of medium dense to dense sand below the footing base (Terzaghi and Peck, 1948; Hughes and Withers, 1974; Hanna and Meyerhof, 1980). In such cases, it is necessary to assess the improvement in bearing capacity as well as the reduction in settlement with an overlaying sand layer. The aim of the present research is to determine the ultimate bearing capacity with an overlaying sand layer for a circular footing which lies over fully cohesive strata. With reference to existing studies, Terzaghi and Peck (1948) determined the bearing capacity of a strip footing overlying clayey strata with an overlay of a sand layer with an assumption that the sand mass spreads the footing load over a larger area and eventually, the shear failure occurs within clay strata. Using the limit equilibrium approach, Meyerhof (1974) and Hanna and Meyerhof (1980) proposed simplified expressions for finding the ultimate bearing capacity of strip and circular footings with a sand layer overlying the clayey strata. A punching shear failure mechanism was assumed in the sand layer, and a truncated pyramid, encompassing the sand mass along with the footing base, is pushed into the lower clayey strata wherein eventually a general shear failure occurs. By assuming a failure mechanism comprised of a series of triangular blocks, Georgiadis and Michalopoulos (1985) evaluated the bearing capacity of a strip footing placed over a combination of cohesive and non-cohesive strata. Oda and Win (1990) conducted small-scale model strip footing tests

*Corresponding author.
E-mail addresses: jkumar@civil.iisc.ernet.in (J. Kumar), manasrcr@civil.iisc.ernet.in (M. Chakraborty).
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by varying the thicknesses of both the sand layer and the clayey stratum. Craig and Chua (1990) performed centrifuge model tests on spudcan footings with an overlaying stronger sand stratum lying over weaker clayey strata. By assuming the rigid block collapse mechanism, Michalowski and Shi (1995) evaluated the bearing capacity of a strip footing placed over a layer of granular soil media underlain by clayey strata. By applying FLAC and the finite element program, Burd and Frydman (1997) computed the bearing capacity of a strip footing placed on layered sand–clay media. By conducting model tests, the load-deformation response of a strip footing placed on a sand bed overlying clayey strata, was observed by Kenny and Andrawes (1997). Okamura et al. (1997) conducted centrifuge model tests for different footings placed on dense sand overlying soft clay. Okamura et al. (1998) proposed a limit equilibrium method for estimating the bearing capacity of a sand layer overlying a deep clayey deposit. Using the finite element limit analysis, Shiau et al. (2003) bracketed the bearing capacity for a strip footing placed on a two-layer soil media. A modified failure mechanism, based on the upper-bound theorem of the limit analysis, was assumed by Huang and Qin (2009) for determining the bearing capacity of a rigid strip footing placed over two-layer soil media. By applying Plaxis, Ornek et al. (2012) examined the scale effects for circular footings placed on natural clay deposits stabilized with compacted granular layers. By performing centrifuge model tests, Lee et al. (2013a) studied the behavior of flat circular and spudcan footings placed on sand overlying a clayey stratum. Lee et al. (2013b) developed a method with an assumption of the conical collapse mechanism by incorporating the dilatancy angle (ψ) of sand. It may be noted that most of the existing approaches for this type of footing problem over a two-layer soil mass are generally based on the assumption of the failure mechanism. The present research does not make any such kind of approximation.

Using the axisymmetric lower bound limit analysis in combination with finite elements and the linear optimization technique, the bearing capacity of a circular footing lying over a sandy layer, which is underlain by fully cohesive strata, is determined. The effects of the thickness of the sand layer and its internal friction angle on the results have been examined for different normalized values of undrained cohesion for clayey strata. Failure patterns have also been drawn for a number of cases. The results from the analysis have been compared with the available theoretical and experimental data from the literature.

2. Definition of the problem

A circular footing of radius b is placed on a sand layer which lies over homogenous clayey strata. The thickness of the sand layer is h. The ground is horizontal and is subjected to uniform surcharge pressure (q). The soil layers are assumed to be perfectly plastic and to obey an associated flow rule and the Mohr–Coulomb yield criterion. The footing is subjected to vertical downward load (Q) without any eccentricity. It is to evaluate the average limit pressure (p_u) as defined by the following expression:

$$p_u = \frac{Q_u}{\pi b^2} \tag{1}$$

Here Q_u is the magnitude of the collapse load. The value of $p_u/(\gamma b)$ becomes a function of the following non-dimensional parameters:

$$\frac{p_u}{(\gamma b)} = f\left(\frac{h}{b}, \frac{c_u}{(\gamma b)}, \frac{q}{(\gamma b)}, \varphi\right) \tag{2}$$

where ϕ defines the internal friction angle of the sand layer and c_u denotes the undrained shear strength of the clay. The unit weights (γ) of both the sand layer and the clayey strata are assumed to be the same.

3. Problem domain, boundary conditions, and mesh

The problem remains symmetrical for the vertical axis passing through the center of the footing. By keeping the axis of symmetry as one of the boundaries, a planar domain in the r–z plane was employed. The thickness of the clayey stratum (h') and the horizontal extent of the domain measured from the footing edge (L) were chosen such that an extension of the size of the domain, beyond that chosen, does not affect the magnitude of the collapse load. For such a domain, the yielded elements generally do not approach any of the chosen boundaries. Considering these aspects, L and h' were kept within the range of $5.5b$ to $22.5b$ and $4.35b$ to $17.75b$, respectively. The chosen problem domain along with the governing stress boundary conditions are depicted in Fig. 1. Along the central axis (MN) of the domain, the shear stress is zero. Along the ground surface (PQ), $\tau_{rz}=0$ and $\sigma_z=-q$. No stress boundary conditions need to be explicitly specified along lines ON or OQ. The roughness angle along the footing–soil interface is specified to equal δ. Along the interface of the footing–sand surface, the following inequality condition has been imposed: $\left|\tau_{rz}\right| \leq (c\cot\phi-\sigma_z)\tan\delta$. Since the footing–soil interface is generally rough, δ has been taken as being equal to ϕ. Three-node triangular elements are used to discretize the problem domain. To simulate a sudden change in the directions of the principal stresses at the edge of the footing, the sizes of the elements are gradually decreased and approach the footing edge. Typical meshes for two different values of h/b, corresponding to $\phi=30^\circ$ and $\phi=35^\circ$, are illustrated in Fig. 2(a) and (b), respectively. Parameters E, N, N_i, and D_c refer to the total number of elements, nodes, nodes along the footing–soil interface, and discontinuities, respectively.

4. Analysis

While performing a lower bound limit analysis, it is necessary to construct a statically admissible stress field so that, in the domain, the equilibrium conditions are satisfied, the normal and shear stresses remain continuous along the chosen stress discontinuities, the prescribed stress boundary conditions