Effect of Confinement on the Bearing Capacity and Settlement of Spread Foundations

Tahsin Toma Sabbagh, Ihsan Al-Abboodi, Ali Al-Jazaairry

Abstract**—**Allowable-bearing capacity is the competency of soil to safely carries the pressure from the superstructure without experiencing a shear failure with accompanying excessive settlements. Ensuring a safe bearing pressure with respect to failure does not tolerate settlement of the foundation will be within acceptable limits. Therefore, settlement analysis should always be performed since most structures are settlement sensitive. When visualising the movement of a soil wedge in the bearing capacity criterion, both vertically and horizontally, it becomes clear that by confining the soil surrounding the foundation, both the bearing capacity and settlement values improve. In this study, two sizes of spread foundation were considered; (2×4) m and (3×5) m. These represent two real problem case studies of an existing building. The foundations were analysed in terms of dimension as well as position with respect to a confining wall (i.e., sheet piles on both sides). Assuming B is the least foundation dimension, the study comprised the analyses of three distances; (0.1 B), (0.5 B), and (0.75 B) between the sheet piles and foundations alongside three depths of confinement (0.5 B), (1 B), and (1.5 B). Nonlinear three-dimensional finite element analysis (ANSYS) was adopted to perform an analytical investigation on the behaviour of the two foundations contained by the case study. Results showed that confinement of foundations reduced the overall stresses near the foundation by 65% and reduced the vertical displacement by 90%. Moreover, the most effective distance between the confinement wall and the foundation was found to be 0.5 B.

Keywords**—**Bearing capacity, cohesionless soils, spread footings, soil confinement, soil modelling.

I. Introduction

ANY foundation when designed should satisfy two essential requirements; first, it must have at least a certain specified safety against bearing capacity failure, and second, the settlement under working load must not exceed the tolerable limits for the superstructure.

In any specific case, one of these criteria will determine the necessary dimensions of the foundation, but it may be difficult to say in advance, which one it will be. Therefore, it is usually necessary to investigate both. The study of ultimate bearing capacity has the purpose of determining the load under which a foundation with given shape, dimensions, and depth sinks indefinitely into the soil. The study of the limit of deformation has the purpose of determining load movements of the structure are at the limit of what is still allowable for the stability.

Fig. 1 shows the pattern of particle motion at failure for a shallow foundation. This provides the basis for understanding the development of failure in granular soil. When the soil fails, the particles move downward and move in the horizontal direction at the same time. Hence, the soil is pushed out from beneath the footing and the surface of the surrounding soil heaves.

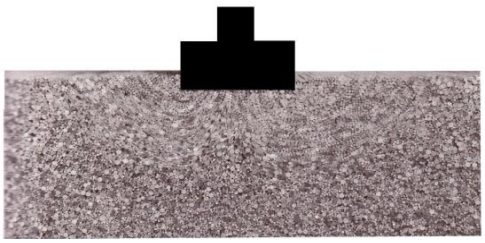

Fig. 1 Failure Zones under Footing (after [1])

By observing the behaviour of shallow foundations subjected to vertical loads, it is a well-established fact that the bearing capacity failure occurs usually as a shear failure of the soil supporting the footing. The problem formulated, as shown in Fig. 2, has been solved by Al-Aghbari [3] using the theory of plasticity.

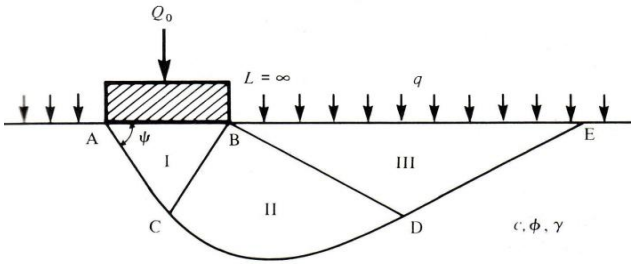

Fig. 2 Failure Wedge (after [2])

The basic solution available indicated that the failure pattern should consist of three zones; (I, II, III), where zone (I) is an active Rankin zone, which pushed the radial Prandtl zone (II) sideways and the passive Rankin zone (III) in an upward direction. The lower boundary (ACDE) of the displaced soil mass is composed of two straight lines AC and DE, inclined at $45+\phi/2$ and $45-\phi/2$ respectively, to the horizontal. The shape of the connected curve CD depends on the angle ϕ and on the ratio $\gamma B/q$.

When $\gamma B/q$ approaches 0 (weightless soil) the curve becomes a logarithmic spiral, which for $\gamma = 0$ degenerates into a circle. In the general case ($\gamma B \neq 0$), the curve lies between a spiral and circle as long as ($\phi \neq 0$). For frictionless soil ($\phi = 0$), the curve is always a circle. All these finding have been

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confirmed experimentally by Vesic [4].

A closed analytical solution of this problem has not yet been found and probably will not be found, except for special cases. For weightless soil ($\gamma = 0$), Prandtl and Reissner have found that:

$$q_o = CN_c + qN_q \quad (1)$$

where N_c and N_q are dimensionless bearing capacity factors defined by:

$$N_q = e^{(\pi \tan\phi)} \tan^2 (\pi/4 + \phi/2) \quad (2)$$

$$N_c = (N_q - 1) \cot\phi \quad (3)$$

For cohesionless soil without overburden ($C = 0$, $q = 0$), it can be shown that:

$$q_o = \tfrac{1}{2} \gamma B N_\gamma \quad (4)$$

where N_γ varies sharply with the dilatancy angle (ψ), Vesic [4] showed that the value of N_γ can be approximated with an error on the safe side (not exceeding 10% for $15 < \phi < 45$ and not exceeding 5% for $20 < \phi < 40$ by the analytical expression:

$$N_\gamma = 2(N_q + 1) \tan\phi \quad (5)$$

For all intermediate cases, where $C \neq 0$, $q \neq 0$, $\gamma \neq 0$ (1) and (4) are combined into:

$$q_o = CN_c + qN_q + \tfrac{1}{2} \gamma B N_\gamma \quad (6)$$

This equation is known as Busiman-Terzaghi equation. This superposition is not strictly correct; however, it leads to errors which are on the safe side, not exceeding 17-20% for $\phi = 30^o$ to 40^o, while equal 0 for $\phi = 0^o$

It should be mentioned that in the literature there exists a great variety of proposed solutions to this problem. While, the variations in N_c and N_q values proposed remain relatively insignificant, the differences in N_γ values coming primarily from the sharp variation of N_γ with the dilatancy angle (ψ), which are substantial, ranging from about one-third to double the values from (6) [5].

II. Previous Studies

The behaviour of axially loaded skirted shallow foundations resting on sand was studied by Hisham [6], using numerical analysis. The effects of foundation size, shear strength of sand, and confinement depth on bearing capacity and settlement of skirted foundations were investigated. The results revealed that skirted foundations exhibit bearing capacity and settlement values that are close to those of pier foundations of the same width and depth. Moreover, the enhancement in bearing capacity of shallow foundation increases with the increase of confinement depth and decreasing relative density of sand. Settlement reduction factor of more than 70% was noticed in the case where skirt depth to foundation width of 2.0 was reached.

Shabana and Joseph [7] presented results for square footings with and without confinement on uniform soil condition showing that the bearing capacity can be improved by a factor of 1.08 to 1.64 when confinement was provided with depth to width of footing ratio of 0.25 to 1.0.

Azzam and Farouk [8] studied laboratory model tests and performed a numerical study on the behaviour of a strip footing with structural skirts adjacent to a sand slope with different skirt depths, different locations of the skirted footing relative to the slope crest and the slope inclination. They concluded that stabilizing the earth slope using structural skirts with adequate depth in the vicinity of strip footing adjacent to slope crest has a significant effect in improving the soil bearing capacity.

Nazir and Azzam [9] investigated a circular type of footing resting on sand pile-partially replaced for two cases: with and without surrounding skirts. Their results showed an improvement of the bearing capacity and a remarkable decrease in vertical settlement.

El-Sawwaf [10] conducted tests of model footings resting on sandy soil to study the confinement effect on the behaviour of such footings. Testing parameters under study included depth of soil and cell dimensions (height, diameter). The results revealed that the bearing capacity of circular footing can be appreciably increased by soil confinement.

Cerato [11] carried-out model footing tests on a finite layer of granular soil. The bearing capacity of model footings resting on the surface of a finite layer of granular (cohesionless) soil was investigated. Results of the model scale footing tests showed that the modified bearing capacity factor, N_γ, is dependent on both relative density (Dr) and relative layer thickness (H/B) but appears to be independent of footing shape.

Anderson [12] studied the collapse of saturated soil due to reduction in confinement. The potential for collapse due to a reduction in confinement is evaluated for two very different soils: uniformly graded sand and an undisturbed clayey colluvium soil. Soil specimens were consolidated anisotropically and subjected to constant-shear-drained (CSD) tests. During the CSD test, the effective confining pressure is gradually reduced while the shear stress is held constant. It was depicted from the study that only for very loose specimens collapse was observed before the failure envelope or steady state is reached. In general, specimens subjected to the CSD stress path behaved differently compared to specimens of similar initial density and confining pressure subjected to typical compression stress paths. This implies that predicting the collapse potential of the stress path is of great importance and hence the potential flow failure of such soils.

III. The Case Study

As a case study, the adopted building for this research is an existing building (Al-Samirrai complex in Baghdad), consists of two floors and has a spread type of foundation resting on a sandy clay soil. The thickness and depth of the spread foundations are 0.6 m each, and the foundation is enduring a uniformly distributed load of (75 kN/m^2). The subsoil