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BEHAVIOR OF CIRCULAR FOOTINGS ON COHESIVE SOILS CONFINED WITH A SHEAR RESISTANCE WALL

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ABSTRACT: In geotechnical applications, soil with a low shear strength exhibits a low bearing capacity and excessive settlement. In this paper, to mitigate the effects of such problems, a confining cylindrical wall is utilized to prevent the development of shear stresses in the soil. The behavior of circular footings confining with a cylindrical wall was investigated through a series of experiments carried out in a laboratory on small-scale models of foundations. The bearing capacity and settlement of foundations were studied in relation to the impact that wall depth and thickness had on these characteristics. According to the results, an increase in wall depth and thickness leads to an increase in the carrying capacity of shallow foundations and a reduction in a settlement. In addition, a theoretical model for determining the ultimate bearing capacity of such foundations is presented. The results of the physical modeling were presented as normalized curves in order to validate the developed theoretical model. Comparing the theoretical results to those of the current experimental investigation shows a good agreement.

Keywords: Cohesive soil, Bearing capacity, Settlement, Shear resistance wall

1. INTRODUCTION

It is common practice to construct shallow foundations in order to sustain a structure by transmitting loads of the structure to the subsurface-bearing soil. Several approaches for soil enhancement have been implemented in order to enhance soil properties. The confinement of soil to shallow depths may have a substantial impact on soil-bearing capacity.

The soil replacement approach is the most commonly used of numerous ground improvement techniques to improve the bearing capacity of footings resting on weak foundations. However, replacing the subsoil will not be deemed practical for large construction projects.

Several techniques have been employed for the improvement of the bearing capacity of shallow foundations. In this approach, planar reinforcement was used in various studies to improve the bearing capacity of foundations in clay [1−4]. Under these circumstances, the primary source of the reinforcing effect is the adhesive resistance that develops at the soil-reinforcement interface.

Furthermore, two methods for enhancing in situ ground conditions are sand compaction piles and stone columns. These methods were utilized to increase the bearing capacity of clay and reduce foundation settlement [5−8].

Bhattacharya and Kumar [9] presented a method for increasing the bearing capacity of foundations on soft clays considering the undrained conditions by incorporating a single vertical granular trench beneath the footing. Also, Bhattacharya et al. [10] conducted an experimental study in which reinforced granular trenches were used to increase the carrying capacity of footings in clay soil.

Recent studies [11, 12] used soil-cement reinforcement to increase the bearing capacity of foundations and reduce settlement. It was suggested that the layer of soil and cement function as a single component at the same depth as the reinforcement base. Other investigations used skirting foundations to improve the bearing capacity of the soil. The bearing capacity of skirted foundations on sand is primarily determined by the interaction of the skirt and the sand [13−14]. An increase in the angle of interface friction improves soil-skirt interaction and, consequently, the bearing capacity of the foundation [15]. In contrast, the foundations on cohesive soil do not exhibit this type of behavior.

This paper presents a method that is rather inexpensive for improving the bearing capacity and compressibility behavior of circular footings resting on cohesive soils by incorporating a cylindrical wall into the foundation system to resist the shear stresses generated in the soil. The mechanism of the present method depends on the fact that the shear resistance wall collapses under loads greater than those required to cause soil failure. In addition, a theoretical model is derived for the proposed type of foundation is presented.

2. RESEARCH SIGNIFICANCE

This study presents a novel method for resisting the shear stresses that develop in silty clay using a cylindrical vertical element of a shear-resistant wall. The present method can be used as a substitute for deep foundations to increase the bearing capacity and reduce the settlement of shallow foundations.

Moreover, a theoretical model for determining the bearing capacity of circular footings on silty clay confining with a shear resistance wall is presented.

3. MODEL TESTS

The experiment was conducted on miniature models (1 g) of circular footing encircled by a cylindrical wall. According to ASTM D1194 [16], four series of stress-controlled loading tests were performed to scrutinize the effect of the shear resistance walls on the behavior of circular footings on cohesive soil.

3.1 Test Materials

Experiments for the study were conducted using silty clay soil. Table 1 summarizes the physical and shear strength properties of soil.

Table 1 Physical and shear strength properties of soil

Property	Value
Liquid limit, L.L [%]	75
Plastic limit, P.L [%]	28
Specific Gravity, *Gs*	2.8
Cohesion, *c* [kPa]	47
Angle of internal friction, ϕ [°]	18

Sand with a specific gravity of 2.65 is used in the composition of shear-resistant walls. The particle size distribution of the sand was determined by dry sieving analysis. The sand was classified by the Unified Soil Classification System (USCS) as SW (well-graded sand). The effective size (D_{10}), uniformity coefficient (C_u), and coefficient of curvature (C_c) of sand are respectively 0.281 mm, 6.37, and 1.0. The grain size distribution of the clay soil and sand is depicted in Fig. 2.

3.2 Shear Resistance Wall

In this research, the foundations are surrounded by a cylindrical sand-cement wall of varying depths and thicknesses. Mixing the components of sand and cement in a dry state and placing them in a trench with a particular density. Following the completion of the wall construction procedure, the radial drainage path is formed.

In the subsequent phase, the hydration process in cement begins due to the transfer of water, and the creation of the cementitious product is facilitated by the presence of sand containing a high percentage of silicate minerals; with time, the shear resistance wall's strength improves.

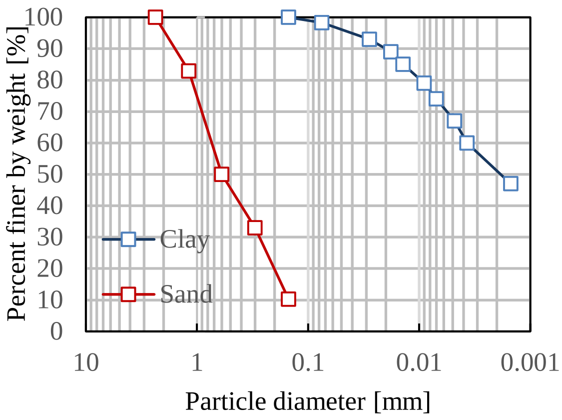

Fig. 2 Grain size distribution of clay and sand

3.3 Experimental Setup and Test Program

Experiments on models were carried out in a testing container with dimensions of 600 mm in length, 300 mm in width, and 350 mm in depth. The reconstitute soil used in the experiments had been prepared to have an in-situ undrained shear strength, c_u, of 47 kPa. In accordance with the dry density of soil (16.9 kN/m^3), the clay was mixed thoroughly with a water content of 21.3%. The soil is classified as low plasticity clay (CH) according to the USCS.

Subsequently, the soil was compacted in 30 mm thick layers in the test container until the desired height of the soil bed was reached. The shear resistant wall was constructed artificially using sand mixed with varying percentages of cement (in percentage of the dry sand). Before mix the components and compaction of the sand-cement layers, the soil around the footing was dug in a cylindrical shape according to the specified dimensions of the shear resistance wall. Each layer of sand-cement mixture was manually compacted with a specified dry unit weight of 15.3 kN/m^3 and allowed to cure.

The present method is vastly more effective than other methods, in terms of saving time and effort, for enhancing the bearing capacity of foundations, such as horizontal layer soil reinforcement [18–20] in which the whole site is excavated.

A rigid footing with a diameter, *D*, of 60 mm and a thickness of 25 mm was placed on the clay surface. According to Hu [21], the affected zone in the clay beneath a circular foundation is contained within approximately 3*D*, where the boundary effect may restrict the active area. In the present investigation, however, the affected zone was less than the test container's perimeter.

A compression test machine attached to the top of a frame applied the load to the footing. The container test is fastened directly to the ground. During the test program, the load was increased steadily until the soil had experienced a shear failure in case of unconfined footing, while for shear resistance foundation the test continued until failure occurs in the cylindrical wall as described in Section 5. 2.

In this regard, each load increment was preserved until the settlement of foundation attained a stable state. Also, two dial gauges were installed on opposing sides of the footing to measure the settlement of the foundation. Figure 3 illustrates the geometry of the soil, the model footing, and the shear resistant wall.

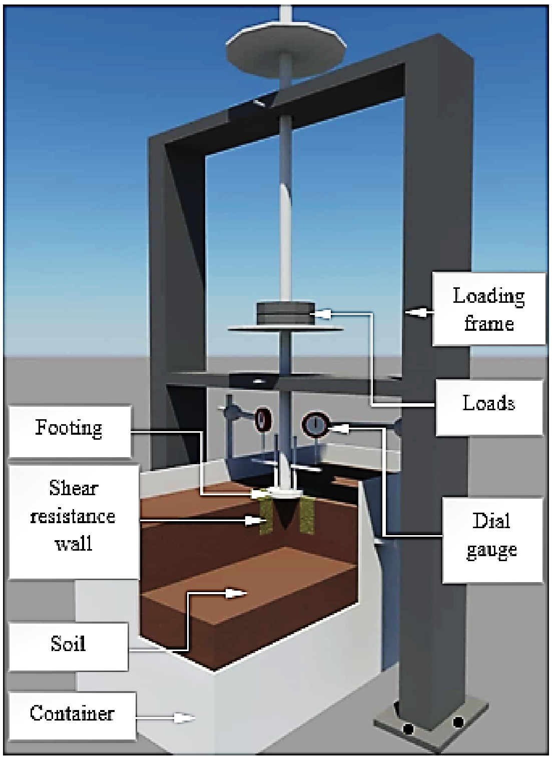

Fig. 3 Setup of the experimental model

The undrained capacity tests (Series A−D) on circular model footing were conducted. During the initial stages of the testing program, the behavior of a surface foundation model is investigated. After that, each series of tests was conducted to determine the effect of a single parameter on the behavior of the shear resistant foundation, while keeping all other variables constant.

The studied variables are the shear resistance wall depth (d), shear resistance wall thickness (t), cement percent (C) and curing time (T) as detailed in Table 2. Fig. 4 illustrates the schematic view of the shear resistance foundation. The tests were conducted at least twice to validate the reliability of the test results.

Table 2 Model tests with different parameters

Test Series	Constant parameters	Variable parameters
A	d/D = 0.5; t/D = 0.5; T^a = 7	C^b = 5, 7, 9
B	d/D = 0.5; t/D = 0.5; C = 7	T = 7, 28
C	t/D = 0.5; T = 7; C = 7	d/D = 0.5, 1.0, 1.5
D	d/D = 0.5; T = 7; C = 7	t/D = 1/3, 1/2, 2/3

[a] *T = Curing Time (day)*
[b] *C = Cement percent (%)*

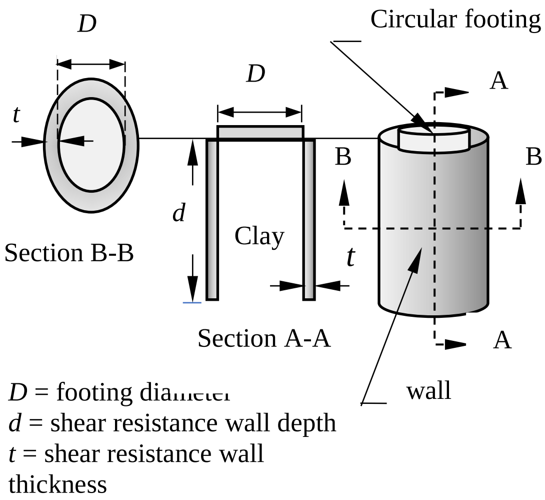

Fig. 4 Schematic view of footing-shear resistance wall

4. MODEL TEST RESULTS

In this section, the results of pressure-settlement relationship for the unconfined footing and footing confined with a shear resistance wall are presented. The behavior of footing confined with the shear resistance wall in silty clay was investigated by analyzing the effects of various parameters on the performance of foundation and then determining the effect of the geometry and strength of the shear resistant wall on the bearing capacity of foundation. The relationships between pressure-settlement for various depths and thicknesses of shear resistance wall and other experimental model test parameters are depicted in Figs. 5−8.

The normalized pressure-settlement curves are represented by a nondimensional factors: S/D which is defined as the ratio of settlement, S, to the diameter of the footing, D. Another factor known as q/c_u is defined as the ratio of pressure, q, to the initial undrained shear strength, c_u of the soil.

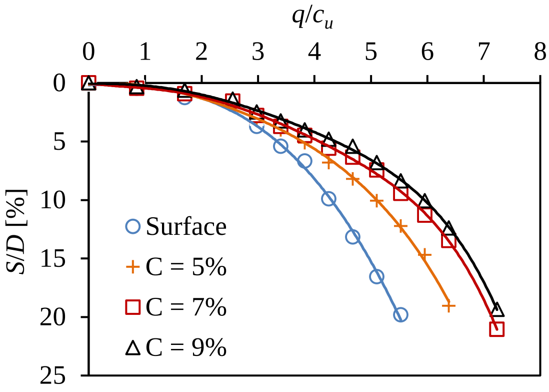

Fig. 5 Normalized pressure-settlement curves (Series A)

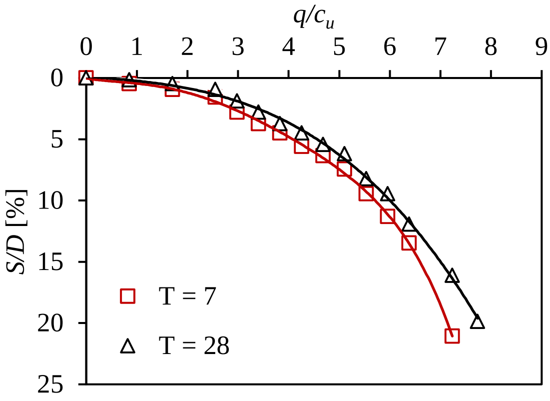

Fig. 6 Normalized pressure-settlement curves (Series B)

5. ANALYSIS AND DISCUSSION

The relationship between load and settlement is determined, as well as the ultimate bearing capacity and settlement of unconfined footings and footings confining with a shear resistance wall.

The improvement in bearing capacity due to the shear resistant wall is indicated by the ratio of bearing capacities, q_{sh}/q_{su}. This ratio represents the ratio of the ultimate load capacity of footing confining with a shear resistant wall to the ultimate load capacity of the surface foundation (footing without confinement). As failure mechanism, the bearing capacity ratio, q_{sh}/q_{su} is determined at the ultimate S/D ratio (S/D = 10%).

To analyze the behavior of shear resistance wall foundations in terms of settlement, the settlement ratio of shear resistance foundation, S_{sh} to the foundation surface, S_{su} at a level of applied stress equal to 50% of the ultimate bearing capacity of the surface foundation was considered. The applied stress level of $0.5q_{su}$ for comparison of settlement values represents the settlement at working stress levels between one-third and one-half the ultimate bearing capacity used in the practical applications. [22].

The shear resistance wall considerably increases the bearing capacity of the foundation, as shown in Figs. 7 and 8. In addition, the failure mechanism for surface foundations changes from punching shear failure to local shear failure when the footing is confined with a shear resistance wall.

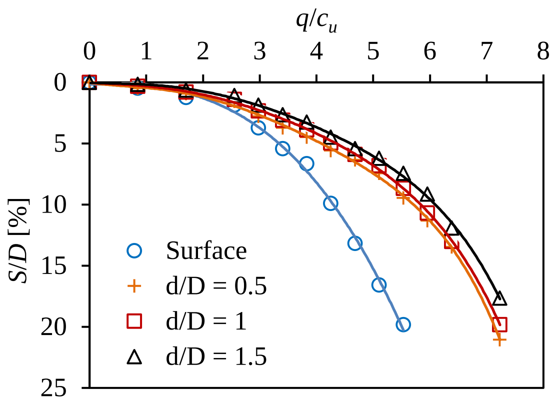

Fig. 7 Normalized pressure-settlement curves (Series C)

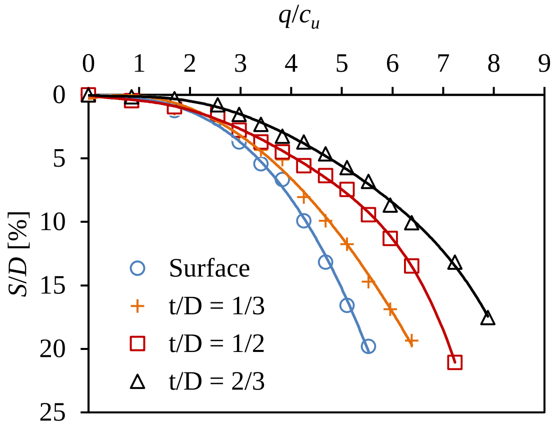

Fig. 8 Normalized pressure-settlement curves (Series D)

In general, it was observed during test program, the installation of the confining wall improves the bearing capacity and rigidity of the foundation.

5.1 Effect of Cement Percent

Sand that has been cemented has a more brittle characteristic. As the confining pressure exerted on the cemented sand increases, its formerly brittle behavior transforms into a more ductile one [23]. This behavior led to the mechanism that resulted in the ability of shear wall foundations to resist shear stresses induced by external loads imposed on the foundation. This behavior can be enhanced by an increase in the percentage of cement and curing time. Fig. 9 demonstrates that the ratio of bearing capacity, q_{sh}/q_{su}, increases as the percentage of cement increases.

5.2 Effect of Shear Resistance Wall Depth

Experiments were conducted on models with three different depth ratios (d/D): 0.5, 1, and 1.5 in order to study and evaluate the effect of shear resistance wall depth on the foundation behavior in terms of bearing capacity and settlement. Fig. 10 illustrates the variations of bearing capacity ratio (q_{sh}/q_{su}) with depth ratio (d/D). The results indicate that the ultimate bearing capacity of shear resistance foundations increases as the depth of shear resistance wall increases. However, the presence of a shear resistance wall around a footing enhanced the bearing capacity by 32%, 35%, and 42%, respectively, for d/D values of 0.5, 1.0, and 1.5.

From the trend of Fig. 10, it seems that the values of improvement for L/D of 1.5 and 1.0 are slightly higher than the value of L/D = 0.5. Therefore, it can be stated that increasing the depth of the wall for a specific value of d/D = 0.5 leads to an improvement in the bearing capacity of the soil beyond the increase in bearing capacity is not substantial. This behavior can be explained by the fact that the maximum strain in the soil occurs at a depth of 0.5 D.

The following is a conceivable explanation for the increase in bearing capacity of the footing: the plastic state of surface failure is developed initially at one edge of the footing and then proceeds to the shear resistance wall. At this stage, the shear resistance wall prevents soil volume expansion, following that an increase in the lateral earth pressure applied on the wall. The failure occurs when the shear stresses in the active zone are greater than the shear strength of the wall as shown in Fig. 11.

From Fig. 12, that depicts the variation of settlement ratio with normalized depth (d/D), it is feasible to observe that there is a significant decrease in the settlement of the footing. This can be explained as follows: when the footing is loaded, the confinement by the cylindrical wall acts against the lateral displacements of soil particles beneath the footing and detains the soil. This results in a significant reduction in the vertical settlement that occurs. Therefore, it is possible that using shear resistance foundations to reduce settlement can be more effective than simply expanding the size of the surface foundations.

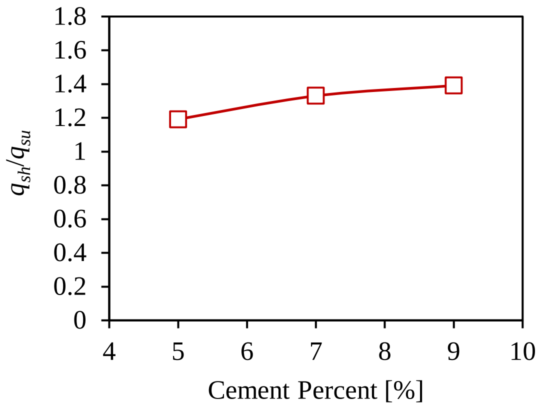

Fig. 9 Variation of bearing capacity ratio with cement percent

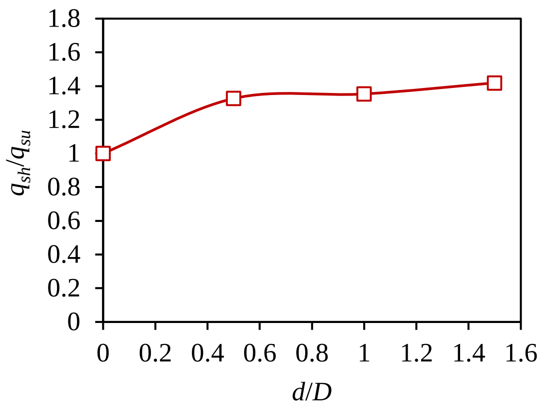

Fig. 10 Variation of bearing capacity ratio with depth ratio

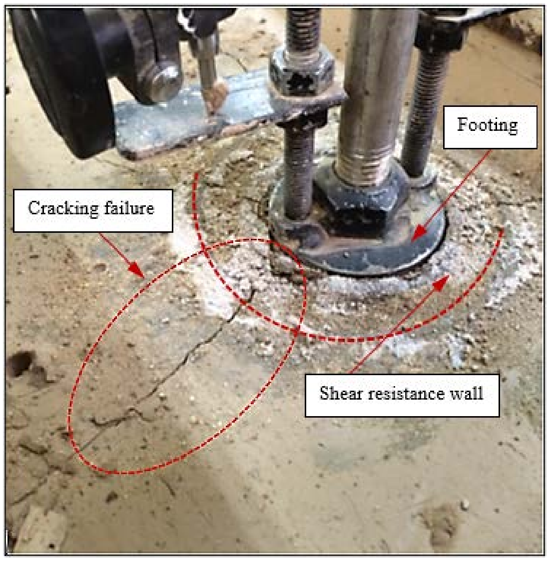

Fig. 11 Failure of shear resistance foundation

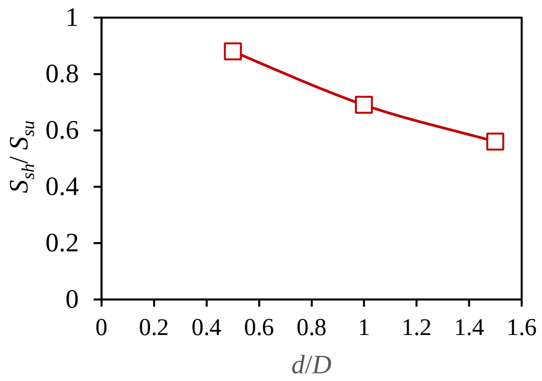

Fig. 12 Variation of settlement ratio with depth ratio

5.3 Effect of Shear Resistance Wall Thickness

The behavior of shear resistance foundations is similar to that of deep foundations such as retaining wall in which the bearing load increases due to the shear resistance. Fig. 13 illustrates that the bearing capacity increases by 10%, 32%, and 45%, respectively, for *t*/*D* values of 1/3, 1/2, and 2/3, respectively compared to surface foundation.

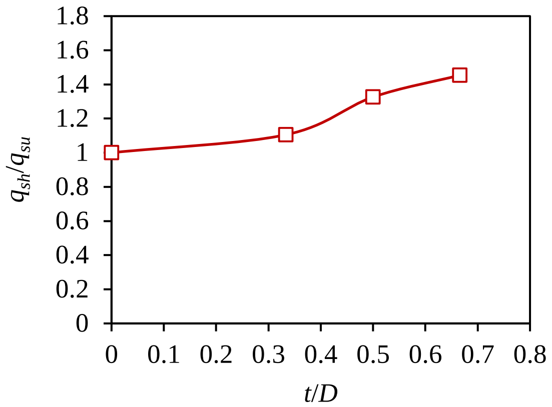

Fig. 13 Variation of bearing capacity ratio with thickness ratio

6. SCALE EFFECT

The physical model utilized in this investigation is miniature, whereas the problem experienced in the field is a prototype footing. The small-scale physical modeling at 1 g is extensively used to study foundation behavior and other simulations due to the problems and difficulties associated with full-scale.

However, in the literature, the scale effects of models in experiments were taken into account to extend the results from small-scale tests to a large-scale design [24]. Therefore, to achieve a more accurate qualitative representation and overcome the scale effect, the normalization of the results was used.

The undrained shear strength, c_u, or the initial effective geostatic stress, p_i', can been used to normalize stress parameters [25, 26].

In the present study, the initial undrained strength, c_{ui}, is used for the normalization because the model test results were compared with the results of the analytical solution, which uses c_u as foundation parameter. Furthermore, according to Ornek et al. [27], the effect of footing size in clay soil can be neglected when the settlement is represented in terms of a non-dimensional settlement (S/*D*). However, the results of the present investigation are defined as a non-dimensional settlement, which eliminates the scale effect from the test results.

7. ANALYTICAL SOLUTION

Prandtl [28] proposed a failure mechanism for shallow foundations under pressure that involves kinematic soil collapse. As shown in Fig. 13, Prandtl's failure pattern consists of three distinct regions: (1) an active zone of soil wedges (region I); (2) a passive zone of soil wedges (region III); and (3) a zone of logarithmic spiral transitions (region II).

The superposition principle is utilized to calculate the bearing capacity of the foundation, which includes soil parameters such as cohesion, *c*, and soil unit weight, The analytical solution for bearing capacity presented by Prandtl is

$$q_u = c\, N_c \tag{1}$$

Keverling Buisman [29] extend Eq. (1) by incorporating the weight of the soil, γ. Terzaghi [30] provided the solution for the ultimate bearing capacity of footing resting on the surface as

$$q_u = c\, N_c s_c + \frac{1}{2}\,\gamma\, D\, N_\gamma s_\gamma \tag{2}$$

The shape factors s_c and s_γ have been proposed for conversion the values of N_c and N_γ from plane strain to axisymmetric.

The present study proposed a failure mechanism for the shear resistance foundations through two zones as shown in Fig. 14

Zone 1: An active zone where a plastic state of surface failure initially developed at one edge of the footing with an active wedge angle, β, and then proceeds to the wall.

Zone 2: The transition zone of the Prandtl failure surface was omitted and compensated for it through a shear resistance wall, which can be thought of as a retaining wall, with an active lateral thrust P_a from the zone 1 pushing against passive resistance P_p. The friction between the soil and wall, δ, decreases the lateral earth pressure (Liu, 2014); therefore, it is neglected in the present analysis for simplicity (i.e., δ = 0).

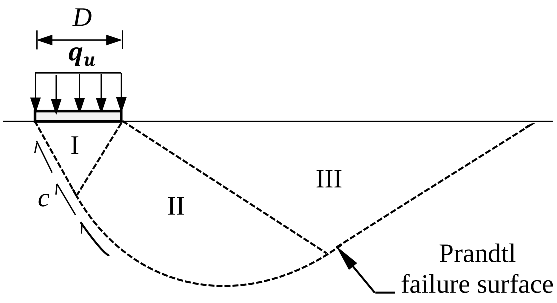

Fig. 13 Prandtl failure surface

In the literature, numerous researchers [30- 32] and others suggested solutions for bearing capacity factor, N_γ, each based on a different proposed failure mechanism.

The bearing capacity factor, N_γ, was determined in this study using the equilibrium of the two Coulomb wedges as shown in Fig. 14.

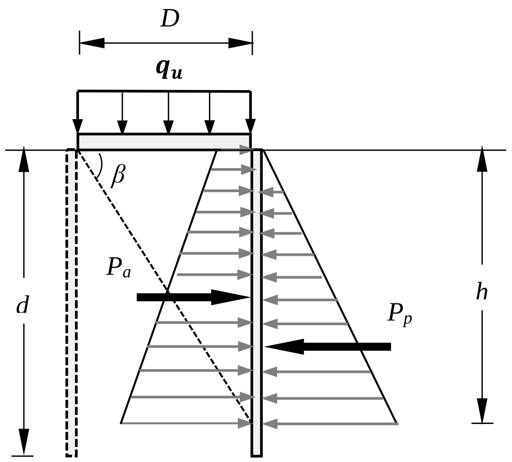

Fig. 14 Assumed failure surface and Coulomb mechanism for shear resistance foundations (δ = 0)

In present analysis, the following assumptions have been considered:

First: The tangential stress induced by the axisymmetric earth pressure was ignored due to the high stiffness of the wall.

Second: The active wedge depth, h, represents the height of the lateral earth pressure acting on the wall.

By considering only the weight of the soil beneath the footing (c_u = 0)

$$P_a = q_u\, hK_a + \frac{1}{2}\gamma h^2 K_a \tag{3}$$

$$P_p = \frac{1}{2}\gamma h^2 K_p \tag{4}$$

From the stability; $P_a = P_p$, then

$$q_u = hK_a + \frac{1}{2}\gamma h^2 K_a = \frac{1}{2}\gamma h^2 K_p \tag{5a}$$

$$q_u = \ hK_a + \frac{1}{2}\gamma h^2 K_a = \frac{1}{2}\gamma h^2 K_p \tag{5b}$$

$$q_u\, D\, tan\, \beta = \frac{1}{2}\gamma (D \tan\beta)^2 \left(\frac{K_p}{K_a} - 1\right) \tag{5c}$$

By simplifying, yield

$$q_u = \frac{1}{2}\gamma\, D \tan\beta \left(\frac{K_p}{K_a} - 1\right) \tag{5d}$$

$$q_u = \frac{1}{2}\gamma\, DN_\gamma \tag{5e}$$

where: $N_\gamma = \tan\beta \left(\frac{K_p}{K_a} - 1\right)$

When the friction between the soil and the shear resistance wall is ignored (δ = 0), the earth pressure coefficients are [33]

$$K_p = \frac{1 + sin\,\phi}{1 - sin\,\phi} \tag{6}$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} \tag{7}$$

The critical angle, β of rupture surface without wall friction is [34]

$$\beta = \phi + tan^{-1}\{[tan\,\phi(tan\,\phi + cot\,\phi)]^{1/2} - tan\,\phi\} \tag{8}$$

8. COMPARISON OF THEORTICAL AND EXPERIMNTAL RESULTS

To validate the present theoretical model, the experimental and theoretical results are compared. In the present prediction, the dimensions of the shear resistance wall are specified using the values as follows: the diameter of the footing is 3.0 m, the depth ratio, d/D, and thickness ratio, t/D, are both 0.50.

Moreover, the percentage of cement is 7% and the curing time is seven days. The soil cohesion and angle of internal friction are 47 kN/m^2 and 18°, respectively. For local shear failure, the modified undrained shear strength value, c_u^* is calculated as c_u^*= 2 cu/3 [35]. The bearing capacity factor, N_c= 5.7.

Fig. 15 depicts the pressure versus settlement ratio curve obtained from the experiment model test. The ultimate bearing capacity of 250 kN as predicted by the theoretical model for footing diameter, D of 3.0 m, is also shown in Fig. 15.

The observed difference between the theoretical prediction of ultimate bearing capacity and the measured bearing capacity is 8% (i.e., the theoretical model is conservative).

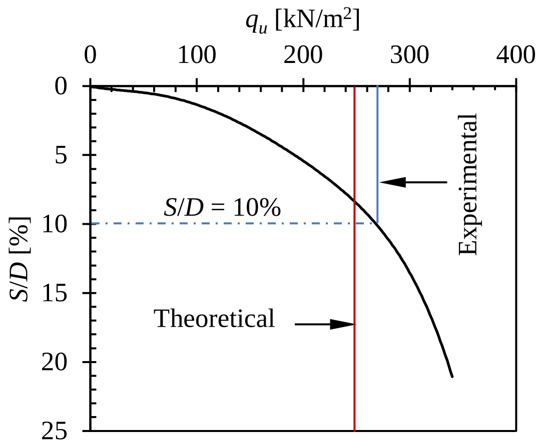

Fig. 15 Ultimate bearing capacity of shear resistance foundation (experimental vs. theoretical)

9. CONCLUSION

This research aims to investigate the influence of soil confinement with a shear resistant wall on the behavior of shallow foundations considering only 1g model tests of circular footings on silty clay. Normalized curves are derived from experimental results which can be used to validate the theortical formulation. Furthermore, the present work developed a theortical model for determining the bearing capacity of shear resistant foundations.

Based on the experimental results and theortical analysis, the following conclusions can be drawn:

1. Soil confinement with a shear resistance wall can improve the performance of circular footings on soil with a low shear strength. It was found that the ultimate capacity increased by a factor of 1.40 compared to the unconfined foundation.

2. The soil confinement with a shear resistance wall can be utilized to reduce the settlement of the foundation by a factor of 0.45.

3. For the depth of shear resistance wall relative to the diameter of the footing, the foundation behaves as a shallow foundation, and the failure occurs as a crack in the surrounding wall.

4. The performance of shear resistance wall is dependent on the depth of wall to diameter of footing ratio, d/D. the optimum value of wall depth, d is 0.5D.

5. Increasing the thickness of the shear resistance wall, results in increasing in the stiffness of the wall; therefore, it can be resisting the shear stresses developed in the soil.

The optimum vale of wall thickness to diameter of footing ratio, t/D is 0.5.

6. The theoretical model predented in this study to determine the bearing capacity of shear resistant foundation is more realistic with the recommended optimum wall geometry,

In conclusions, it is recommended that future research investigate the effect of shear resistance walls on the behavior of square and rectangular footings on cohesive soil as well as the shallow foundation on soft cly.

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