Ultimate Bearing Capacity of Closely Spaced Strip Foundations

BRAJA M. DAS AND SAID LARBI-CHERIF

Laboratory model test results for the ultimate bearing capacity of two closely spaced, rough, shallow, strip foundations resting on sand are presented. The experimental results are compared with the existing theory. The ultimate bearing capacity of foundations increases with the decrease of the center-to-center spacing; however, the magnitude of increase is considerably smaller than that predicted by theory.

Numerous theoretical and experimental studies of the ultimate bearing capacity of isolated foundations in homogeneous and layered soils are available in the literature. However, studies of the change in the load-bearing capacity of foundations when they are closely spaced are relatively scarce. This change may be important in some foundation design problems such as rest area buildings and mass transit stations. Two questions might arise during the design: (a) How does the ultimate bearing capacity change when two shallow foundations are placed rather close to each other? (b) What is the settlement of the foundations at ultimate load as compared with that obtained when they are far apart? Stuart (1) proposed a theoretical solution for the ultimate bearing capacity of two closely spaced, rough, shallow, strip foundations on sand (see Figure 1) as

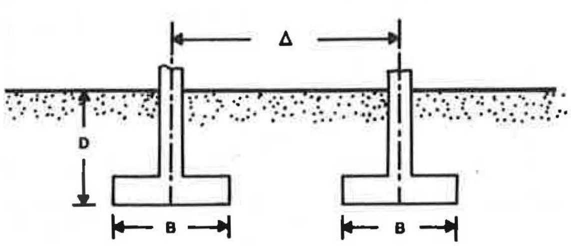
$$q_u = qN_q\alpha_q + \tfrac{1}{2}\gamma BN_\gamma\alpha_\gamma \quad (1)$$

where

q_u = ultimate bearing capacity of each foundation,
γ = unit weight of soil,
B = width of foundation,
q = γD,
D = depth of foundation,
N_q, N_γ = bearing capacity factors, and
α_q, α_γ = efficiency factors.

The variation of α_γ and α_q with the center-to-center spacing (Δ) of foundations is shown in Figures 2 and 3. From these figures, it may be seen that for a given soil friction angle (ϕ) the two closely spaced foundations behave as one from Δ/B = 1 up to a given value. This is because, at small values of Δ/B, the soil located in between the two foundations forms an inverted arch that travels down as a unit with the foundation as the load is applied. At a value of Δ/B = 1, the zone or arching disappears, and the system behaves as a single unit of foundation having a width of 2B.

Figure 1. Definition of parameters for the ultimate bearing capacity of two closely spaced, shallow foundations as given by Equation 1.



In this paper some laboratory model test results are compared with the theoretical efficiency factors. Also, based on the laboratory results, some discussion of foundation settlement at ultimate load is presented. The tests were conducted in the Soil

Figure 2. Variation of efficiency factor α_γ with center-to-center spacing of foundations [theoretical curves after Stuart (1)].

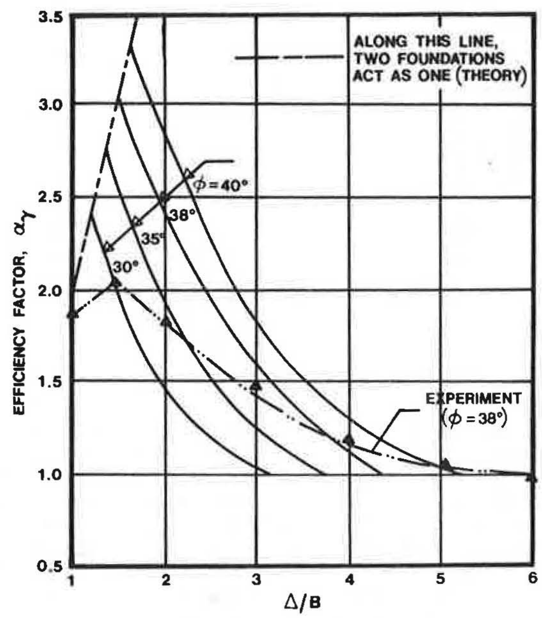
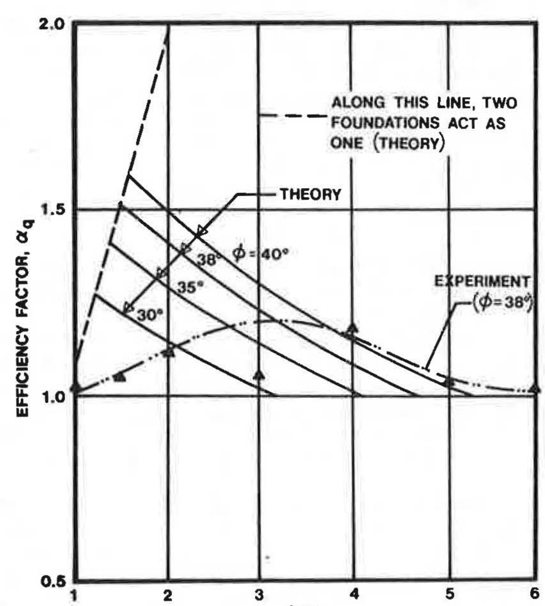


Figure 3. Variation of efficiency factor α_q with center-to-center spacing of foundations [theoretical curves after Stuart (1)].

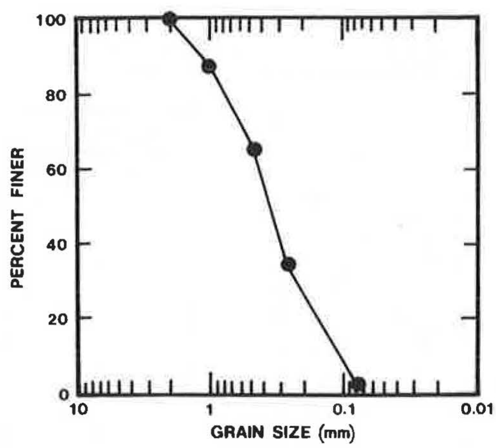


Mechanics Laboratory of the University of Texas at El Paso.

LABORATORY MODEL TESTS

Laboratory model tests were conducted in a box having a length, width, and height of 1.524 m, 0.305 m, and 0.914 m, respectively. The sand used for the tests had 100 percent passing No. 10 sieve, 87 percent passing No. 20 sieve, and 2 percent passing No. 200 sieve. The grain-size distribution of the sand is shown in Figure 4. To conduct the model tests,

Figure 4. Grain-size distribution of sand used for model tests.



sand was poured in 50.8-mm layers in the test box and compacted to a unit weight of 15.88 kN/m³, which gave a relative density of compaction of about 54 percent. The angle of friction at this unit weight of compaction was determined from direct shear tests to be equal to 38 degrees. This relative density of compaction was chosen for two primary reasons, one of which is the reproducibility in the model test box. The second reason is to check the opinion of some investigators, such as Vesic (2), that interference effect on ultimate bearing capacity of shallow foundations may be practically negligible when the foundations are located on loose soil (i.e., local shear failure condition).

Two model steel foundations measuring 50.8 mm x 304.8 mm were used for the tests. The ends of the model foundations and the sides of the box were polished to make them as smooth as possible to keep friction resistance to a minimum. Sandpaper was glued to the bottoms of these model foundations to make them rough. The gradation of the sand on the sandpaper was similar to that used for the tests. Each model foundation was rigidly attached to a steel shaft having a diameter of 19 mm. For tests on a single foundation, load to the foundation was applied through this shaft by means of a hydraulic jack. To conduct tests on two closely spaced foundations, the steel shafts were rigidly attached to a horizontal steel crossbar 50.8 mm x 25.4 mm in cross section. Load to the foundation was applied at the center of the crossbar. The loads on the foundation and the corresponding foundation settlements were measured by a proving ring and a dial gauge, respectively.

Model tests were conducted for D/B ratios of zero and one, with center-to-center foundation spacing-to-width ratios (ΔB) of 1, 1.5, 2, 3, 4, 5, and 6. Tests on single isolated foundations ($\Delta/B = \infty$) were also conducted for D/B ratios of zero and one. After each test the sand in the test box was completely emptied and recompacted in layers before the next test was begun.

In all tests the nature of variation of load versus displacement plots was similar to the type obtained for local shear failure in soils. A typical load versus settlement plot obtained from the laboratory tests is shown in Figure 5. The ultimate load at failure for each test was determined by using the criteria proposed by Vesic (2). This means that the ultimate load in a load-settlement plot is defined as the point where the slope of the load-settlement curve first reaches zero or a steady minimum value. Figure 6 shows the variation of the ultimate failure load (q_u) for all tests conducted.

For single isolated foundations the bearing capacity factors N_γ and N_q can be determined as follows.

$$N_\gamma = [q_{u(\Delta/B = \infty,\ D/B = 0)}]/0.5\gamma B \tag{2}$$

$$N_q = [q_{u(\Delta/B = \infty,\ D/B = 1)} - q_{u(\Delta/B = \infty,\ D/B = 0)}]/\gamma D \tag{3}$$

Figure 5. Typical load-settlement curves from laboratory tests.

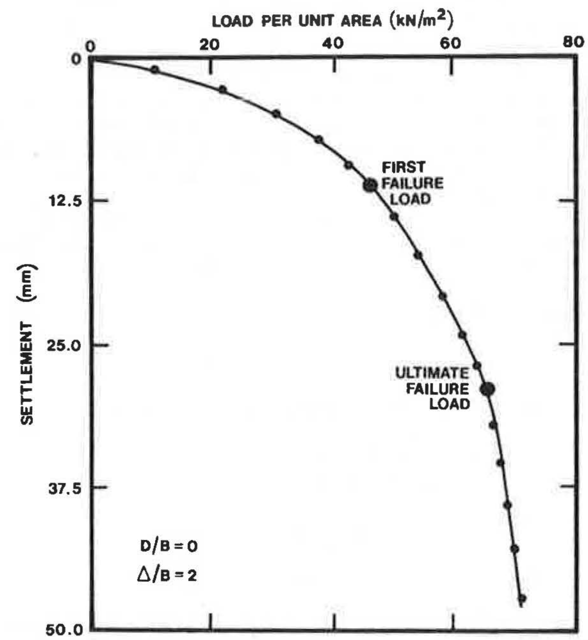
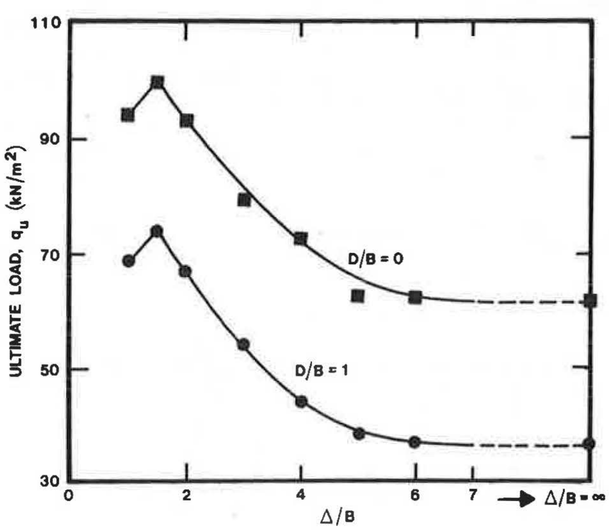


Figure 6. Variation of ultimate bearing capacity obtained from laboratory model tests.



Based on Equations 2 and 3 the experimental values of N_γ and N_q were determined to be 91 and 31, respectively. This value of N_γ = 91 is in reasonable agreement with the theoretical values of Terzaghi (3) and Caquot and Kerisel (4) for a soil friction angle of 38 degrees. However, a comparison shows that this experimental value of N_q is in better agreement with the theory given by Vesic (5) and can be expressed as

$$N_q = (e^{3.8\phi \tan\phi}) \tan^2(45 + \phi/2) \quad (4)$$

According to Equation 4, for ϕ = 38°, N_q = 30.1. This compares well with the experimental results of N_q = 31.

The efficiency factor α_γ for the closely spaced foundations can be determined as

$$\alpha_\gamma = (\tfrac{1}{2}\gamma BN_\gamma\alpha_\gamma)/(\tfrac{1}{2}\gamma BN_\gamma) = [q_{u(\Delta/B,\, D/B = 0)}] / [q_{u(\Delta/B = \infty,\, D/B = 0)}] \quad (5)$$

In a similar manner the efficiency factor α_q can be determined from the following equation.

$$\alpha_q = (qN_q\alpha_q)/(qN_q) = [q_{u(\Delta/B, D/B = 1)} - q_{u(\Delta/B,\, D/B = 0)}]$$

$$\div [q_{u(\Delta/B = \infty,\, D/B = 1)} - q_{u(\Delta/B = \infty,\, D/B = 0)}] \quad (6)$$

By using these equations and the experimental values given in Figure 6, the experimental variations of the efficiency factors have been calculated. Figures 2 and 3 also show the variation of α_γ and α_q. Comparison of the experimental results with the theory shows that, although the nature of variation of the experimental α_γ and α_q is approximately similar to that predicted by theory, their magnitudes are considerably smaller than that predicted by theory for Δ/B < about 3. Note that the value of α_q reached a maximum of about 1.2 and, for all practical purposes, may be assumed to be equal to one. The discrepancy between the theoretical and experimental values of α_γ and α_q may be due to the assumption of ideal rigid-plastic behavior of soil. Also, the theory neglects the self-weight of soil (6).

Figure 7 shows a nondimensional plot of the foundation settlement (S) at ultimate load against the center-to-center spacing of the foundation. Note that, although the ultimate bearing capacity of model foundations increased with the reduction of center-to-center spacing (Δ), it was accompanied by an increased foundation settlement, particularly for the range of Δ/B < about 4.5. This is potentially important for practical design problems where settlement normally controls.

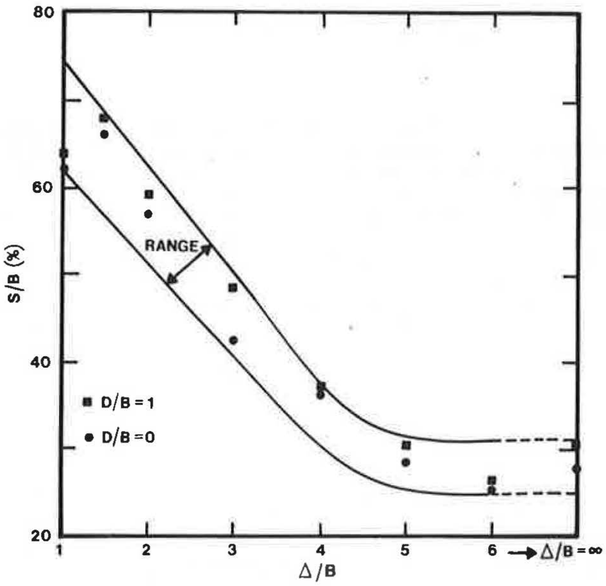
CONCLUSION

Laboratory model test results for the ultimate bearing capacity of two closely spaced, rough, strip foundations on sand have been presented. The experimental results have been compared with the theory presented by Stuart (1). Based on this comparison, the following conclusions can be drawn.

The ultimate bearing capacity of two closely spaced foundations increases as their center-to-center spacing decreases.

The efficiency factors α_γ and α_q generally vary only in the patterns predicted by Stuart's theory for Δ/B > about 4. For Δ/B < about 4, the magnitudes of

Figure 7. Nondimensional plot of foundation settlement (S/B) against the center-to-center spacing of foundations (Δ/B).



α_γ and α_q are considerably smaller than those predicted by theory.

In soils where local shear failure occurs, the value of the efficiency factor α_q is practically equal to one for all center-to-center spacings.

In these tests, the settlement of foundations at ultimate load increased with the decrease of Δ/B. The settlement is expressed by $(S/B)_{\Delta/B = 1} \approx 2(S/B)_{\Delta/B = 4.5}$.

REFERENCES

1. J.G. Stuart. Interference Between Foundation With Special Reference to Surface Footings in Sand. Geotechnique, Vol. 12, No. 1, 1962, pp. 15-22.
2. A.S. Vesic. Analysis of Ultimate Loads on Shallow Foundations. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM1, 1973, pp. 45-73.
3. K. Terzaghi. Theoretical Soil Mechanics. Wiley, New York, 1943.
4. A. Caquot and J. Kerisel. Sur le termes de surface dans le calcul des foundations en milieu pulverulent. Proc., Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, Switzerland, Vol. 1, 1953, pp. 336-337.
5. A.S. Vesic. Bearing Capacity of Deep Foundations in Sand. HRB, Highway Research Record 39, 1963, pp. 112-153.
6. J.M. West and J.G. Stuart. Oblique Loading Resulting From Interference Between Surface Footings on Sand. Proc., Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Canada, Vol. 2, 1965, pp. 214-217.

Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.