Bearing Capacity of Shallow Foundation on a Granular Trench in Clay

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SUMMARY The existing theories and modifications thereof for the ultimate bearing capacity of a strip foundation on a granular trench made in a weak clay have been described. These theoretical values have been compared with the results of laboratory model tests.

1 INTRODUCTION

Design and construction of shallow foundations on soft clay layers is rather difficult due to the low bearing capacity of the soil associated with the large magnitude of elastic and consolidation settlements. Under these circumstances, various types of soil improvement techniques are used. Some of these techniques may include chemical stabilization of clay, preloading, and construction of stone columns. If the preceding techniques are not satisfactory, the structure is constructed on pile foundations or drilled shafts.

The use and analysis of stone columns have been examined by several investigators in the past (e.g., Barksdale and Bachus, 1983; Hughes and Withers, 1974; Hughes, Withers, and Greenwood, 1975). The construction of a stone column generally consists of water jetting a vibrofloat into the soft clay to prepare a circular hole. The hole is then filled with imported granular material. This granular material is compacted in the circular hole as the vibrator is withdrawn.

The present paper relates to the determination of the ultimate bearing capacity of a shallow strip foundation which is supported by a granular trench made in a soft saturated or near-saturated clay soil. The granular trench is actually a plane strain variation of the stone column approach described in the preceding paragraph. Figure 1 shows the geometric parameters of a strip foundation on a granular trench. It needs to be pointed out that the width of the trench (B) for this study has been kept equal to the width of the foundation (B).

At the present time, published studies (experimental and/or theoretical) relating to the bearing capacity of a strip foundation on a granular trench in clay are relatively limited. The purpose of the present paper is to present the existing theories and modifications thereof on this subject. These will be verified by laboratory model tests, and the shortcomings of the theories will be highlighted.

2 THEORETICAL PREDICTION OF ULTIMATE BEARING CAPACITY

Madhav and Vitkar (1978) have analyzed the ultimate bearing capacity of a shallow strip foundation of width B resting on a granular trench of width W and depth L (Figure 2) by using the upper bound limit analysis theorem presented by Drucker and Prager (1952) and Chen (1975). At ultimate load, a general shear failure mechanism in

Figure 1 Strip foundation on a granular trench constructed in a saturated clay

Figure 2 General shear failure in soil under a strip foundation as assumed by Madhav and Vitkar (1978)
soil is assumed. As shown in Figure 2, the failure consists of four parts. They are:

(a) Rankine active zone ABC, (b) mixed transition zones AGI and BCD, (c) transition zones AJD and BDE, and (d) passive zones AJD and BEF. The failure surfaces CD and CG are arcs of a logarithmic spiral based on the friction angle of the trench material (θ). Similarly, DE and GI are arcs of a logarithmic spiral based on the friction angle of the clay (in this case, θ\text{clay}=0). The theoretical ultimate bearing capacity thus determined can be expressed as

\[ q_u = c_u N_u + (\gamma_c D_f) N_q + \frac{\gamma_c}{2} B N_r \]  

(1)

in which \( c_u \) = undrained shear strength of clay; \( \gamma_c \) = unit weight of clay; \( D_f \) = depth of foundation; \( N_u, N_q, N_r \) = bearing capacity factors = f(\( \phi, \theta, R/B \)) \( \gamma_c \) = friction angle of the granular material in the trench. Figure 3 shows the variation of the interpolated values of \( N_u, N_q, \) and \( N_r \) for the case of \( W/B = 1 \).

\[ q_u = \left( \frac{1 + \sin \phi}{1 - \sin \phi} \right) \left( \gamma_c D_f + 2c_u \right) \]  

(2)

However, \( \sigma_{III} = \sigma_1 \). So

\[ \sigma_1 = q_u = K_p \sigma_{III} = K_p \sigma_1 \]

where \( K_p = \) Rankine passive earth pressure coefficient

\[ 1 + \sin \phi = \frac{1}{1 - \sin \phi} \]

So

\[ q_u = \left( \frac{1 + \sin \phi}{1 - \sin \phi} \right) \left( \gamma_c D_f + 2c_u \right) \]  

(3)

The third possible mode of failure, although remote, is the condition where the granular trench will act as a strip pile. For such a case

\[ q_u = \frac{1}{B} \left\{ (Q_p) + (Q_s) \right\} \]  

(4)

where \( Q_p = \) point bearing capacity per unit length of the foundation, and \( Q_s = \) adhesion between the trench material and clay per unit length of the foundation. However, \( Q_p \) can be approximated as (Skempton, 1951)

\[ Q_p = B \left\{ (5c_u \{1 + 0.2(L/B)\}) \right\} \]  

(5a)

and

\[ Q_s = 2c_u \]  

(5b)

Combining Equations (4), (5a), and (5b)

\[ q_u = 5c_u \{1 + 0.2(L/B)\} + 2(L/B) c_u \]  

(6)

3 EXPERIMENTAL VERIFICATION

3.1 Laboratory Model Tests

In order to verify the validity of the theories stated in Section 2, a number of small-scale laboratory model tests were performed.

For conducting the tests, a large amount of a clayey soil was collected from the field. The soil was well pulverized in the laboratory. Figure 5 shows a grain-size distribution of the soil based on a wash sieve analysis. The liquid and plastic limits of the clay were 31% and 18%, respectively. A poorly graded, highly angular sand was used as the trench material, the grain-size distribution of which is also shown in Figure 5. A predetermined amount of water was added to and thoroughly mixed with the clay. The moist soil was then transferred to
several plastic bags that were tightly sealed. These bags were stored in a moist curing room for about a week before use.

The model foundation was a steel plate with dimensions of 304.8 mm (length) \times 101.6 mm (width) \times 9.5 mm (thickness). The inside dimensions of the model test box were 91.4 mm (length) \times 304.8 mm (width) \times 609.6 mm (height). The sides of the box were heavily braced to avoid lateral yielding. The inside of the box was smoothed as much as possible with sandpaper to minimize friction during the tests.

For conducting the model tests only on sand or clay (without the granular trench), the soil was compacted in the model test box to the desired height in layers of 50.8 mm to 76.2 mm. The compaction was done in sections by using a flat-bottomed rammer. After compaction, the model foundation was placed centrally in the box on top of the compacted soil layers. The load on the model foundation was applied by a hydraulic jack. The load and corresponding settlement were recorded by a proving ring and dial gauge, respectively.

In order to conduct model tests in clay with a granular trench, the clay was first compacted in the test box in 50.8 mm layers to a desired height. A wooden mold having outside dimensions of 304.8 mm (length) \times 101.6 mm (width) \times 508 mm (height) was then centrally placed in the test box on the compacted clay layer. The clay was then compacted in the test box to the desired unit weight in lifts of 50.8 mm to 76.2 mm. Following compaction of the clay, the inserted wooden mold was carefully removed, thereby creating a trench in the clay having a width equal to the width of the model foundation. Measured amounts of sand were then placed in the trench and compacted in lifts of 50.8 mm by a flat-bottomed rammer. The model foundation was then placed on the granular trench, and the bearing capacity test was conducted. This means that all tests were for surface foundation conditions (i.e., \( D_f = 0 \)). At the end of each test, some surface cracks in the clay were observed close to the foundation. The sand in the trench was carefully removed, and the post-failure shape of the trench was observed. In all cases, a lateral bulging of the trench was clearly noticed. The bulging was generally noticeable at depths of about 0.58 to 0.6 m. The sequence of the model tests, unit weight of compacted soils, and the shear strength parameters of the soil are shown in Table I.

### 3.2 Model Tests

The plot of the load per unit area (q) on the model foundation and the corresponding settlement for tests on uniform clay and sand (Test Series I, II, III, and IV) are shown in Figure 6. A general shear type of failure for the tests on sand was observed showing a peak value of \( q = q_u \). For tests on clay only, the nature of the plots of q vs. foundation settlement were typical of local shear type of failure in soil.

For surface foundation conditions (i.e., \( D_f = 0 \))

\[
q_u = c_u N_c \quad \text{(in clay with } \phi = 0) \tag{7}
\]

\[
q_u = \frac{1}{2} g N_b N_y \quad \text{(sand)} \tag{8}
\]

where \( N_c \) and \( N_b \) are bearing capacity factors. Using proper experimental values of \( q_u \) in Equation (7) yields a value of \( N_c = 5.05 \) for Test No. 1. This value is in good agreement with the experimental value of 4.92 for this test.
with the theoretical value of $N_c=5.14$ (Prandtl, 1921). In a similar manner, using the experimental values of $q_u$ from Test Nos. 2, 3, and 4 in Equation (8) yields values of $N_r$ to be 203.3, 128, and 77.5, respectively. These values of $N_r$ are slightly higher than those provided theoretically by Cagout and Kerisel (1953).

The nature of the load per unit area ($q$) vs. settlement relationships obtained for tests in Series IV, V, VI, and VII (Test Nos. 5 to 28) were of similar type as observed for Test No. 1 shown in Figure 6. The ultimate loads obtained from these tests have been plotted against L/B in Figure 7, from which the following observations can be made:

![Figure 7 Variation of $q_u$ with L/B](image)
a. For given values of $q_g$ and $c_u$, the magnitude of $q_u$ generally increases in a linear manner with $L/B$ up to a maximum and remains constant thereafter. This maximum value will be referred to hereafter as $q_u(\text{max})$.

b. The critical value of $L/B$ beyond which the magnitude of $q_u$ does not increase is about 2.58 to 3.8.

3.3 Comparison of Model Test Results with the Theoretical Prediction

In order to compare the present laboratory experimental results with the theories given in Section 2, the ultimate bearing capacities calculated by using Equation (1) and Figure 3 along with proper values of $y$ and B have been determined and are shown in Figures 7a, 7b, and 7c. In a similar manner the magnitudes of the theoretical $q_u$ as obtained from Equations (3) and (6) have also been plotted in this figure. Based on the comparison of the theoretical and experimental values, it appears that Equation (1) predicts values of $q_u(\text{max})$ on the unsafe side. The magnitude of $q_u$ varies between 38% and 42% of $q_u(\text{max-expt.})/q_u(\text{Eq.1})$. On the other hand, Equation (3) gives a reasonable prediction of the experimental $q_u(\text{max})$. The magnitude of $q_u(\text{Eq.3})-q_u(\text{max-expt.})/q_u(\text{Eq.3})$ for the present tests varied between ±2% to ±4%. Equation (6) consistently yielded higher values of $q_u$ as compared to the experimental values. This was, of course, expected; primarily due to the fact that the granular trench is not rigid enough to act as a pile.

4 SETTLEMENT AT ULTIMATE LOAD

Figure 8 shows a nondimensional plot of the foundation settlement at ultimate load with $L/B$. For any given test series, the settlement at ultimate load decreases with $L/B$ and reaches a minimum value at $L/B=3$. This minimum value of settlement was close to that observed for tests on sand alone.

5 CONCLUSIONS

A number of model test results for determination of the ultimate bearing capacity of a strip foundation supported by a granular trench made in a soft clay soil have been presented. For the present tests the width of the granular trench was kept equal to the width of the foundation. Based on the present tests, the following conclusions can be drawn:

1. The ultimate bearing capacity of the foundation increases with $L/B$ up to a maximum value of $L/B=3$. For $L/B>3$, the ultimate bearing capacity practically remains constant.

2. The settlement of the foundation at ultimate load decreases with $L/B$, reaching a constant minimum value at $L/B=3$.

3. The theory of Madhav and Vithar (1978) predicts values of $q_u(\text{max})$ which are on the unsafe side. This is primarily because of the theoretical assumption of general shear failure in soil which was not the case as observed from the present tests. The theory for obtaining the ultimate bearing capacity based on the bulging mode of failure is in good agreement with the experimental results.

6 REFERENCES


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