Flat Dilatometer method for estimating bearing capacity of shallow foundations on sand

Alan J. Lutenegger
University of Massachusetts, Amherst, Massachusetts, USA

Michael T. Adams
Federal Highway Administration, McLean, Virginia, USA

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ABSTRACT: A design method is presented for estimating the ultimate bearing capacity of shallow foundations on granular soils using the results from the Dilatometer Test. The method is developed using results obtained from prototype-scale footing load tests performed on compacted sand at the FHWA Turner-Fairbank Highway Research Center and full-scale footing load tests performed on natural sand at the National Geotechnical Experimentation Site at Texas A& M University. The method uses the DMT lift-off and 1mm expansion pressures directly and is similar to the empirical design approach currently in use with the prebored Menard Pressuremeter test. The method incorporates an empirical bearing capacity factor which, much like the Pressuremeter method, is shown to be related to the embedment ratio (D/B) of the footing.

1 INTRODUCTION

Estimating the ultimate bearing capacity of shallow foundations on granular soils is a routine exercise performed by practicing geotechnical engineers throughout the world. Engineers need to evaluate the ultimate bearing capacity in order to insure that a sufficient factor of safety is provided against bearing capacity under the proposed design allowable pressure. The bearing capacity and settlement behavior of shallow foundations are uniquely interrelated. That is, at higher factors of safety, footings experience smaller settlements.

The ultimate bearing capacity of footings on sands may be evaluated using traditional bearing capacity equations (e.g., Terzaghi, Meyerhof, Hansen, etc.) in which superposition of terms is assumed and bearing capacity factors are evaluated as a function of the internal friction angle of the soil or by empirical equations using the results obtained from different in situ tests. Alternatively, empirical allowable bearing capacity charts may be used which provide a limit on settlement. The use of traditional bearing capacity equations is an indirect design approach that requires an estimate of the internal friction angle of the soil, often obtained from empirical correlations to penetration tests such as the SPT or CPT.

This paper presents an alternative direct design method for determining the ultimate bearing capacity of shallow foundations resting on granular soils using results obtained from Dilatometer tests. The method uses the Dilatometer lift-off and 1 mm expansion pressure readings directly without any additional interpretation of test results and is developed based on the observed ultimate bearing capacity of Prototype-Scale and Full-Scale footing load tests performed on concrete footings on compacted and natural sand.

2 DETERMINING BEARING CAPACITY FROM IN SITU TESTS

Engineers have a number of options for estimating the ultimate bearing capacity of shallow foundations using the results obtained from in situ tests. This approach is attractive for granular soils since it is difficult to obtain undisturbed samples for laboratory testing. The more common methods rely on the results of penetration tests, such as the Standard Penetration Test (SPT), the Cone Penetration Test (CPT) or Dynamic Drive Cone Tests (DCPT). For the current study, the design methods based on the pressure expansion curve of the Pressuremeter Test and the tip resistance from the Cone Penetration Test are most applicable.

3.1 *Bearing Capacity of Footings from the Pressuremeter*

Menard (1963) had suggested that the ultimate bearing capacity of shallow foundations, q_{ult}, could be evaluated from the results of prebored pressuremeter tests as:

$$q_{ult} = K\, P^*_L + \sigma_{vo} \tag{1}$$

where P^*_L is defined as the net limit pressure which equals P_L - σ_{Ho}, where P_L is equal to the PMT limit pressure extrapolated from the actual test data and σ_{Ho} equals the in situ total horizontal stress at the test depth, K equals an empirical bearing capacity factor that depends on soil type, soil stiffness, and equivalent footing embedment ratio, H_e/B, and σ_{vo} is the total vertical stress at the base of the foundation. In many cases, the expansion of the pressuremeter in sands does not give a limiting pressure and therefore, the value of P_L may be interpreted by a graphical extrapolation procedure as described in ASTM Test Method D4719 or by other means. The value of σ_{Ho} is often taken directly from the PMT curve as P_o or alternatively from an estimate of the in situ lateral stress ratio, K_o, and soil unit weight. A potential drawback to this technique is that a reliable estimate of K_o is needed.

A detailed design procedure using Equation 1 is described by Baguelin et al. (1978) and Briaud (1992). It should be noted that for this method the recommended value of P^*_L for use in design is taken at depths between 1.5 B below and 1.5 B above the base of the footing. Charts for choosing appropriate values of K are provided by Menard (1963) Baguelin et al. (1978) and Briaud (1992). There is only a slight increase in K with increasing footing embedment within the range of H_e /B from 0 to 1.

2.2 *Bearing Capacity of Footings from the Cone Penetration Test*

Meyerhof (1956; 1965) suggested that the ultimate bearing capacity of shallow foundations on granular soils could be estimated from the CPT tip resistance, q_c. Charts for estimating the allowable bearing capacity of shallow foundations from q_c and taking into account the relative footing embedment have been presented in the Canadian Foundation Engineering Manual (1975; 1985; 1992). In general this approach assumes that q_{ult} is directly related to q_c and is supported by Briaud and Jeanjean (1994) Tand et al. (1995) and Eslaamizaad and Robertson (1996) as:

$$q_{ult} = Kq_c \tag{2}$$

The factor K is dependent on the relative footing embedment D/B. For square footings and D/B in the range of 0 to 1, the factor K varies from about 0.22 to 0.30, depending on the sand density.

3 INVESTIGATION

The principal focus of the work presented in this paper was to investigate the use of the Dilatometer test for estimating the ultimate bearing capacity of shallow foundations on sands. Results from a number of Prototype-Scale footing load tests performed on compacted sand in conjunction with the Shallow Foundations Research Program at the Federal Highway Administration were used. Additional footing load test results available from Full-Scale footings performed on a natural sand deposit at Texas A&M University for the Federal Highway Administration were also used to supplement the Prototype-Scale tests.

3.1 *Prototype-Scale Footing Tests*

Prototype-Scale footing load tests were conducted at the Federal Highway Administration Turner-Fairbank Highway Research Center at McLean, Virginia. Tests were performed in a 3.5 m x 7.1 m x 6.5 m deep test pit on compacted sand beds prepared at different relative densities. Sand placement in the test pit was by 0.3 m loose lifts using a vibratory plate compactor to achieve the required relative density. In place density tests were performed using a nuclear moisture-density gauge at several locations around the pit for each lift to verify the density achieved with each pit fill. The sand used for the testing was uniform fine mortar sand having a mean grain size of 0.75 mm and a uniformity coefficient of 2.6. There is a small amount of fines present in this material, generally less than 5%. Minimum unit weight is 1.41 Mg/m^3 and maximum unit weight is 1.70 Mg/m^3. Tests were conducted on sand beds with relative densities ranging from 13.1% to 75.0%. Load tests were performed with the sand in a moist (M) condition (i.e., as compacted with no water table present), and with the water table located at the surface (S).

Footings were constructed of reinforced concrete and had widths ranging from 0.30 m to 1.22 m. Footings were placed at different depths in the sand to provide varying embedment ratios (D/B) ranging from 0 to 1. Incremental load tests were performed on each footing using a hydraulic ram loading system with the central vertical load measured using an electronic load cell and the vertical displacement measured at the four corners of the footing using LVDT's. Data from each of the load tests were recorded automatically on a data acquisition system as the test progressed. All but two of the footings tested in the facility were square. A summary of the square footing tests performed at the FHWA facility is presented in Table 1.

Table 1. Prototype-scale footing tests.

Series	D_r (%)	Moisture	Width (m)	D/B
90	13.1	M	0.30	0
			0.46	0
			0.61	0
			0.91	0
95	38.8	M	0.30	0
			0.46	0
			0.61	0
			0.91	0
95GA1	46.0	S	0.30	0
			0.61	0
			0.91	0
95GA2	42.4	S	0.30	1
			0.61	1
			0.91	1
95GA3	38.8	M	0.30	1
			0.61	1
			0.91	1
95SD1	35.2	M	0.61	0
			0.61	0.25
			0.61	0.5
			0.61	1
			0.91	1
95SD2	38.8	M	0.61	0
			0.91	0.5
95SD3	38.8	M	0.61	0
			1.22	0.5
95SD4	38.8	M	0.30	0
			0.30	0.5
			0.61	0
			0.61	1
			1.22	0
97SD1	54.5	M	0.30	0.5
			0.61	0
			0.61	0.25
			0.61	0.5
			0.61	1
			0.91	0.5
100SD1	75.0	M	0.30	0.5
			0.61	0
			0.61	0.25
			0.61	0.5
			0.61	1
			0.91	0.5

The Dilatometer test provides a measure of the lift-off and 1 mm expansion pressure of a flexible, circular diaphragm on the face of a flat blade after quasi-static penetration into the soil. Dilatometer tests were performed in each of the test pit fills at FHWA using the procedure recommended by Schmertmann (1986). Two DMT profiles were performed in each pit fill at intervals of 0.3 m beginning alternatively at a depth of 0.3 m and 0.45 m at two locations and were continued to a depth of 4 m below the sand surface.

3.2 *Full-Scale Footing Tests*

In order to provide a comparison between the Prototype-Scale footing load tests performed at FHWA on compacted sand and Full-Scale production size footings placed on a natural sand, test results from the footing load tests performed at the National Geotechnical Experimentation Site at Texas A&M University for the ASCE Specialty Conference Settlement '94 were also used. The sand at this site is a natural deposit which can be described as medium dense fine silty sand. Grain-size and other characteristics of this sand are given by Gibbens and Briaud (1994). The in situ relative density of the sand was estimated to be on the order of 55% based on the results of Standard Penetration and Cone Penetration Tests. Footing load test results and DMT test data for this site are reported by Briaud and Gibbens (1994). All footings tested in this field program were square and ranged in size from 1 m to 3 m. The embedment ratio (D/B) ranged from 0.27 to 0.70. A summary of these footing tests is given in Table 2.

Table 2. Full-scale footing tests.

Footing No.	Width (m)	Depth (m)	D/B
1	3.0	0.8	0.27
2	1.5	0.8	0.53
3	3.0	0.9	0.30
4	2.5	0.8	0.32
5	1.0	0.7	0.70

3.3 *Determining Ultimate Bearing Capacity*

In order to develop a bearing capacity design method, it was important to determine the ultimate bearing capacity from each of the load tests in a consistent manner. In the absence of a well-defined plunging failure which identifies the ultimate capacity, there are a number of methods that can be used to interpret either the "allowable" or the ultimate bearing capacity of foundations from footing load

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tests. In many cases, an “allowable” bearing pressure is used to design footings, where the footing stress corresponding to a limiting absolute settlement value, e.g., 25.4 mm, is used to define the “allowable” bearing capacity. This approach typically is used with any one of a number of design charts.

When actual footing load test data are available, the ultimate bearing pressure may be interpreted using one of the following approaches: 1) choosing the footing stress corresponding to a limiting *relative* settlement value, e.g., s/B = 10% (Briaud and Jeanjean 1994); 2) choosing the footing stress corresponding to a marked change in the settlement, e.g., the intersection of the initial and final tangent slope of the stress vs. settlement curve (Trautman and Kulhawy 1988); 3) manipulating the stress vs. settlement data and then selecting the footing stress corresponding to an intersection point e.g., log stress vs. log settlement (DeBeer 1970); or 4) choosing a reasonable model to fit the stress vs. settlement data and extrapolating to the asymptotic value corresponding to an upper limit of stress, e.g., hyperbolic model (Chin 1983; Wrench and Nowatzki 1986; Ghionna et al. 1991; Wiseman and Zeitlan 1994; Thomas 1994). Each of these interpretation methods may give a different value of bearing capacity and therefore in the development of a design method it is important to select a single interpretation approach in order to be consistent.

In this study the ultimate bearing capacity for all footings (Prototype-Scale and Full-Scale) was determined as the stress producing a relative displacement of 10% of the footing width, hereafter referred to as the 0.1B Method.

4 PROPOSED DESIGN METHOD

Using the results of the footing load tests and the Dilatometer tests performed, an approach similar to that used with the Pressuremeter was investigated for using the DMT results to estimate ultimate bearing capacity as:

$$q_{ult} = N_D\,(P_1 - P_o) + \sigma_{vo} \qquad (3)$$

In this case, P_o represents the DMT lift-off pressure and P_1 represents the DMT 1mm expansion pressure taken directly from the DMT test results. Since the DMT blade is of fixed dimensions, the use of P_o and P_1 represent pressure values that are repeatable from any DMT equipment and which are not subject to arbitrary graphical interpretation. The value of N_D is a DMT “bearing capacity factor” that should depend only on soil stiffness and the geometry of the loading and is analogous to the factors K used in the PMT and CPT design methods and given in Equations 1 and 2.

In sands, it has been well documented that the pressure-expansion curve of the DMT membrane closely follows a linear shape as the membrane is expanded from P_o to P_1 (Campanella and Robertson 1991; Bellotti et al 1997). The slope of the curve is dependent on OCR and relative density. Therefore, the pressure difference P_1 - P_o represents a measure of the stiffness of the soil and was used by Marchetti (1980) to define the “Dilatometer Modulus”, E_D. The value of P_o is related to the initial in situ horizontal stress, but also reflects the influence of stress history and relative density, all of which influence bearing capacity of shallow foundations on granular soil. Therefore, the analogy between the PMT approach and the DMT approach is very strong. In Equations 1 and 3, the vertical stress at the base of the foundation typically represents a relatively small contribution to the bearing capacity for D/B in the range of 0 to 1 and therefore a reasonable estimate of soil unit weight is be considered adequate.

Houlsby and Wroth (1989) showed that in clean sands, the thrust required to advance the DMT blade was related to the lift-off pressure P_o. This has been confirmed by others (e.g., Campanella and Robertson; Bellotti et al. 1994). Additionally, it has been shown that the DMT thrust also relates to the 1 mm expansion pressure P_1 (Campanella and Robertson 1991). It has also been shown that the DMT thrust and the tip resistance from a CPT are strongly correlated in the same sand deposit (Campanella and Robertson 1991). Therefore, it is intuitive that a correlation may be established between the CPT q_c and the DMT pressure difference (P_1 - P_o). This means that it should be expected that if q_c may be related to q_{ult} (i.e., Equation 2) then (P_1 - P_o) may also similarly be related to q_{ult}.

The DMT and the PMT are in situ tests that measure soil response principally in the horizontal direction. One may question the use of such tests to provide useful results for predicting the response of vertically loaded foundations. The bearing capacity of square and circular footings can actually be modeled as a spherical cavity expansion in soil, which obtains a large degree of expansion resistance from the horizontal support of the soil immediately under the footing. This is also consistent with basic Rankine theory for bearing capacity of shallow foundations.

5 RESULTS

Since q_{ult} was determined for each of the footing load tests and σ_{vo} may be calculated from total unit weight that was measured during each pit fill, values of N_D were back calculated for all of the tests by rearranging Equation 3 to solve for N_D. In this procedure, since footing tests represent embedment ratios less than 1, the DMT results within a zone between the base of the footing and a depth of 1.5B below the footing were used. A comparison using the DMT results in a zone of 2B above and 2B below the footing indicated no significant change in the results. Typical DMT results obtained on compacted sand pit fills at FHWA showed that the values P_o and P_1 increased with depth as would be expected in a uniform sand of constant Relative Density.

5.1 *Prototype Footing Tests*

The results of interpreted ultimate bearing capacity from the footing load tests in Table 1 are given in Table 3 along with back calculated values of N_D. The variation in N_D with relative footing embedment from the prototype-scale footing tests are shown in Figure 1. It can be seen that N_D increases slightly with increasing D/B as is expected and is similar in magnitude to values of K suggested for the PMT.

Table3. Results of prototype-scale tests.

Series	D_r (%)	Moisture	Q_{ult} (kPa)	N_D
90	13.1	M	121	0.89
			138	0.90
			180	1.16
			197	1.14
95	38.8	M	245	0.63
			260	1.29
			300	1.53
			380	1.52
95GA1	46.0	S	65	0.63
			87	0.70
			140	0.88
95GA2	42.4	S	197	1.72
			350	2.11
			490	2.27
95GA3	38.8	M	480	1.25
			655	1.23
			770	1.34
95SD1	35.2	M	240	1.19
			345	1.16
			405	1.35
			525	1.57
			280	0.88
95SD2	38.8	M	237	0.58
			448	0.97
95SD3	38.8	M	230	0.89
			620	1.52
95SD4	38.8	M	280	0.92
			400	1.13
			355	0.79
			785	1.36
			580	1.04
97SD1	54.5	M	755	2.29
			508	0.77
			800	1.16
			1110	1.50
			1320	1.48
			1350	1.47
100SD1	75.0	M	1510	2.06
			1000	1.10
			1175	1.22
			1160	1.10
			1350	1.01
			2325	1.68

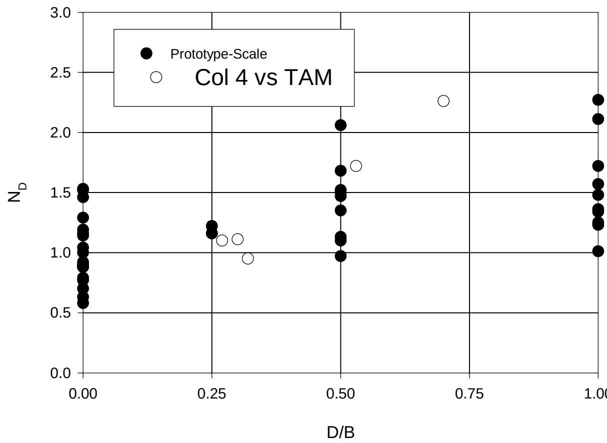

Figure 1. Test results.

The test data of Figure 1 suggest a more or less linear increase in N_D with increasing embedment over the range of D/B from 0 to 1. Beyond an embedment ratio of 1 it is likely that an increase in N_D occurs at a much lower rate and becomes negligible beyond D/B greater than about 4. This would be consistent with observations of PMT results and other general bearing capacity observations as a transition from shallow to deep behavior occurs and bearing capacity increases. The results shown in Figure 1 also suggest that the value of N_D is generally independent of footing size for a given D/B, at least in the range of footings included in this study (B = 0.3 m to 3.0 m).

For the same size footing and footing embedment and for similar same water table conditions, the results indicate that the value of N_D is independent of the relative density of the sand. Variations in the relative density and other soil conditions e.g., water table, appear to be automatically reflected in the DMT results through P_o and P_1. The influence of footing size is accounted for by using the DMT results over an appropriate zone of influence for indi-

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vidual footings. The observed variation in bearing capacity factors at a given D/B value indicated in Figure 1 is likely the result of variations in the DMT results and variations in interpreting the load test results. It should be noted that the scatter indicated in Figure 1 for any given value of D/B is similar to the observed scatter in K values reported for the PMT.

Prototype-Scale tests were also performed on two rectangular footings having length/width ratios (L/B) equal to 2 and 4 to provide a comparison with results obtained from square footings having the same width. The results of these tests indicated that the back calculated values of N_D were less than for square footings of the same width and embedment, and on average represented values of N_D on the order of 70% of the value for a square footing. This is also consistent with the PMT design procedure and with general bearing capacity theory. Therefore, the bearing capacity factors in Figure 1 are recommended for use with square footings only and an adjustment factor of 0.7 should be applied for use with rectangular footings.

5.2 *Full-Scale Footing Tests*

The results of the full-scale footing tests conducted at Texas A&M are given in Table 4 and are also shown on Figure 1. These results fall within the band of test results obtained from the prototype-scale tests and confirm that the value of N_D depends primarily on relative embedment. The results indicated in Figure 1 are also intuitively reasonable.

Table 4. Results of Full-Scale Footing Tests.

Footing No.	q_{ult} (kPa)	N_D
1	1820	1.10
2	1560	1.72
3	1210	1.11
4	1280	0.95
5	1060	2.26

In a uniform, normally consolidated sand deposit with a constant relative density, one would expect the values of P_o and P_1 to increase linearly with depth, but with P_1 increasing at a faster rate. This would produce a higher modulus with increasing depth because of the effect of increasing confining pressure. This would in turn produce higher N_D values for larger D/B ratios for a constant footing width B. Since the ultimate bearing capacity factors obtained using Equation 3 and presented in Figure 1 are based on defining the ultimate bearing capacity as 10% of the footing width, there is no provision for settlement limitations in the design procedure presented.

Using a global factor of safety of 3, which is common in routine shallow foundation design practice, the recommended approach gave "allowable" footing bearing stresses which all produced settlements of less than 25.4 mm. Therefore, the authors suggest that provisionally, a factor of safety of 3 be applied to this procedure to obtain an allowable bearing capacity. As always, the permissible settlement criteria must be checked to provide an adequate foundation design since a fixed settlement criterion of 25.4 mm represents different *relative* displacement for different size footings. At the present time, no recommended design curve for evaluating N_D is given in Figure1. A conservative approach would be to use the lower bound data for a given D/B.

One could argue that an alternative approach to the one presented could be to correlate q_{ult} to the DMT Modulus, E_D, however, this is less direct than the approach presented and implies a certain level of confidence in the use of the Modulus value.

6 CONCLUSIONS

An empirical design procedure for estimating the ultimate bearing capacity of shallow foundations on granular soils based on the results obtained from the Flat Dilatometer test has been presented. The proposed method is simple to use and similar to a procedure that has previously been suggested and used with Pressuremeter results. The procedure makes use of the two pressure readings routinely obtained from the Dilatometer test and requires no additional interpretation of test results. Unlike the PMT method, no estimate of K_o is required.

An empirical bearing capacity factor, N_D, is introduced. Bearing capacity factors for use with this method have been presented for square footings for different values of the footing embedment ratio, D/B. An adjustment factor of 0.7 is suggested for use with rectangular footings. The value of N_D may be dependent on the method used in interpreting the ultimate bearing capacity, which in the present study was the stress producing a relative settlement of 10% of the footing width.

In sands, the use of the Dilatometer allows a more rapid testing approach than the Pressuremeter test, allows for more test data to be obtained within the cavity zone of interest, does not usually require a borehole, and requires less time for data reduction. The proposed method may provide a more cost effective direct design method for shallow foundations and would also be more attractive than using the results of the SPT, which can be subject to large variations.

Additionally work is currently underway to determine if this approach may be extended to other soil types and to determine if other variables can be identified which influence the value of N_D.

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