Estimation of Settlement of Footings on Sand — A Critical Reappraisal

by

G. Ramasamy*

Introduction

The allowable bearing pressure of footings on sand, in general, is governed by settlement considerations rather than bearing capacity considerations. The bearing capacity increases as the width of footing is increased resulting in an increase in margin of safety against shear failure for a given intensity of loading. However, the increase in width increases the settlement for a given intensity of loading, thus reducing the margin of safety with respect to tolerable settlement. Except when the sand is loose and the footing is narrow and below water table, the limiting settlement is found to govern the allowable bearing pressure. Therefore, a reliable estimation of allowable bearing pressure for design of footings is directly related to the reliability of the methods of estimating settlement.

The settlement of footings on sand is governed by many factors, the most well known among them being *(i)* the relative density of sand, *(ii)* the size of the loaded area, and *(iii)* the position of water table. Other factors such as the state of insitu stresses, capillarity, amount of fines present and gradation of soil though known to significantly influence settlement, cannot be accounted for quantitatively in a settlement computation due to lack of adequate information. In view of this, an accurate estimate of settlement remains a distant dream. The situation was aptly described in the words of Terzaghi (1951) thus : "An accurate forecast of settlement of a single footing supported by natural ground, on the basis of the results of soil tests, would be a full time job for an exceptionally competent research engineer, backed by a sponsor who does not count on costs". The situation has changed very little since then. However, this situation hardly justifies the use of settlement equations in a routine fashion. A lot of useful information on the subject has been published in the last two decades which should be judiciously used along with a chosen method of settlement computation. To help the above process, some useful information available in the literature and some based on the results of Author's research work are discussed under the following headings.

1. Interpretation of load test data
2. Effect of capillarity
3. Effect of overconsolidation
4. Limitations of model tests on prepared sand beds.

* Reader, Civil Engineering Department, University of Roorkee, Roorkee-247 667. India.

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5. Comments on the method proposed by De Beer and Martens (1957) and
6. Computation of modulus of deformation.

Interpretation of Load Test Data

The limitations of the results of load tests are well known and need not be over emphasised. However, when viewed in the light of the complex nature and the limitations of other tools available for solution of the problem, it appears that the results of load tests can serve the purpose satisfactorily provided the interpretation is done with adequate knowledge of available literature on the subject. In view of this, some useful information available in the literature is presented briefly.

Bjerrum and Eggestad (1963) and D'Appolonia et. al. (1968) have reported comparison of observed settlements with those predicted using Terzaghi and Peck (1967) extrapolation relationship given by Eq. 1.

$$S_f = S_p \left[\frac{B_f\,(B_p + 30)}{B_p\,(B_f + 30)} \right]^2 \qquad \ldots(1)$$

in which B_f and B_p refer to width of footing and plate respectively and S_f and S_p refer to settlement of footing and plate respectively. The results of comparison are given in Figure 1. The results reveal that the use of Eq. 1 may lead to under estimation of settlement. It was realized that a single extrapolation formula is inadequate and can not possibly hold good for variety of field conditions one may come across. Terzaghi and Peck (1967) suggest that curve '*a*' in Figure 1 may be valid for situations where sand consists of a small amount of organic content, curve '*b*' for loose sands and curve '*c*' for medium and dense sands.

Parry (1978) has presented S_f/S_p vs B_f/B_p relationships for different ground conditions based on elastic calculations assuming that the modulus of elasticity of soil is linearly proportional to the standard penetration resistance (SPT) value, N (observed N). The assumption that the elastic modulus is proportional to N is questionable in view of the fact that the available evidence suggests that the increase in stiffness due to over consolidation or preloading is not reflected in the observed N values (D'Appolonia

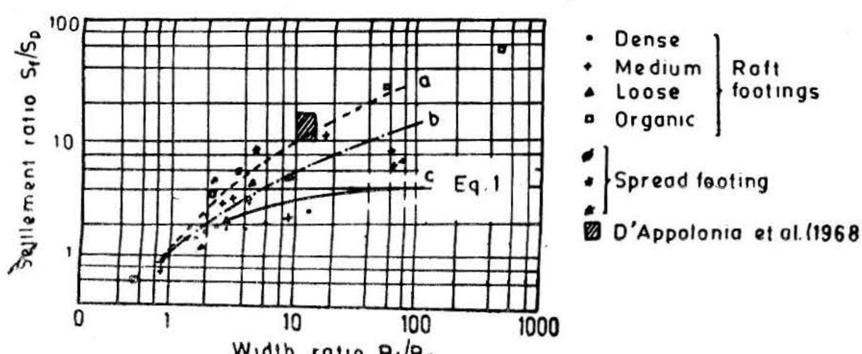


FIGURE 1 **Relationship Between Settlement and Dimension of Area as Derived from Collected Case Records. (After Bjerrum and Eggestad, 1963).**

et. al., 1968). However, the results obtained may be considered valid for variations of soil stiffness similar to assumed variations of N. The assumed variations of N with depth and the corresponding S_f/S_p vs B_f/B_p relationships are shown in Figures 2 and 3. In view of the assumption made, the variations of N with depth represent the assumed variations of stiffness with depth. The results presented in Figures 2 and 3 may be considered to provide information on possible changes in S_f/S_p vs B_f/B_p relationship for different variations of ground stiffness with depth.

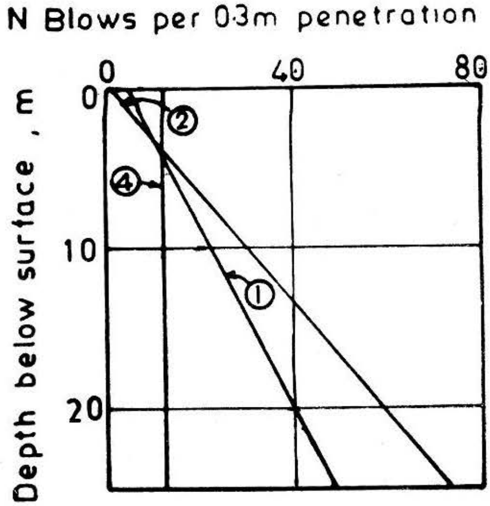


FIGURE 2 Distribution of 'N' Values with Depth (After Parry, 1978)

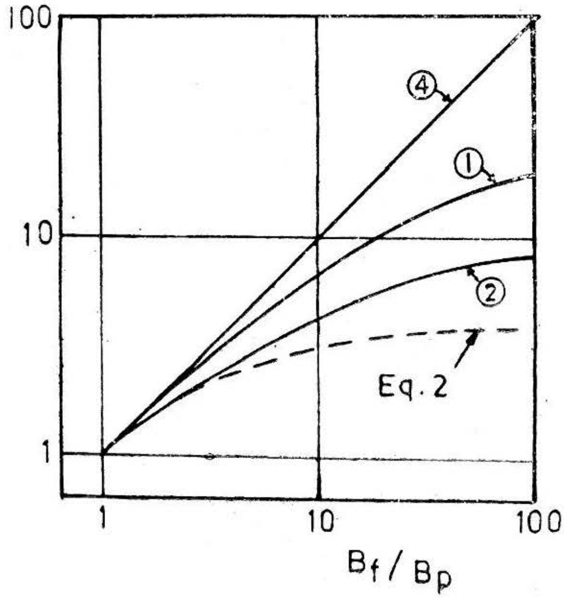


FIGURE 3 Settlement Ratio-as a Function of Width for Soil Conditions Shown in (After Parry, 1978) Figure 2.

Burland et. al. (1977) have compiled a large number of available field measurements of settlements and suggested upper limits for settlement of footings on dense, medium and loose sands. The results are presented in Figure 4. When the results of load test is available, it was suggested that the settlement to pressure ratio, *S/P* for the plate may be plotted in Figure 4 and the settlement of the footing may be extrapolated using the appropriate trend line. The ratio, *S/P* may be obtained from the initial straight line portion of the load-settlement curve.

The results, Figures 1, 2 and 3 reveal that the use of Eq. 1 in a routine fashion would lead to underestimation of settlement in most of the situations. Therefore it is suggested that the settlement of large footings should be extrapolated from the results of load tests judiciously in the light of the above information.

In using the extrapolation relationships for the computation of footing settlement from the results of plate load test data it should be ensured that only the portion of the load-settlement curve within the limits of linear relationship is used for the computations (Rao and Ramasamy, 1980). It appears that this point overlooked by many (Meigh and Nixon, 1961; Prakash and Saran, 1973). It is found that the plate settlement, S_p corresponding to the permissible settlement of footing, S_f normally gets plotted in the failure range of the load-settlement curve (Figure 5). It is unsound to read out the pressure corresponding to this point as the allowable soil pressure. It is recommended that the allowable pressure should be read out on a line joining the origin and the point corresponding to 50 percent of the ultimate load of the plate as shown in Figure 5.

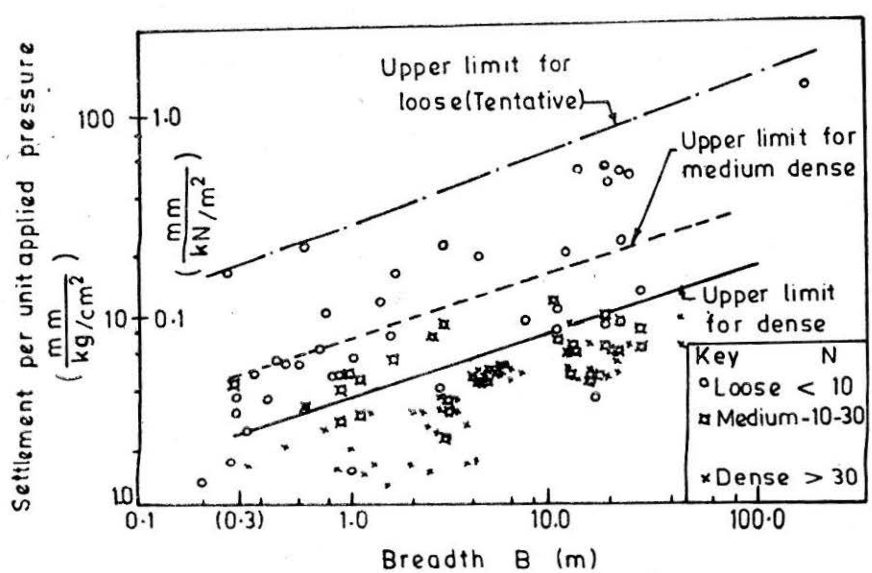


FIGURE 4 Obeserved Settlement of Footings on Sand of Various Relative Densities (After Burland et. al., 1977).

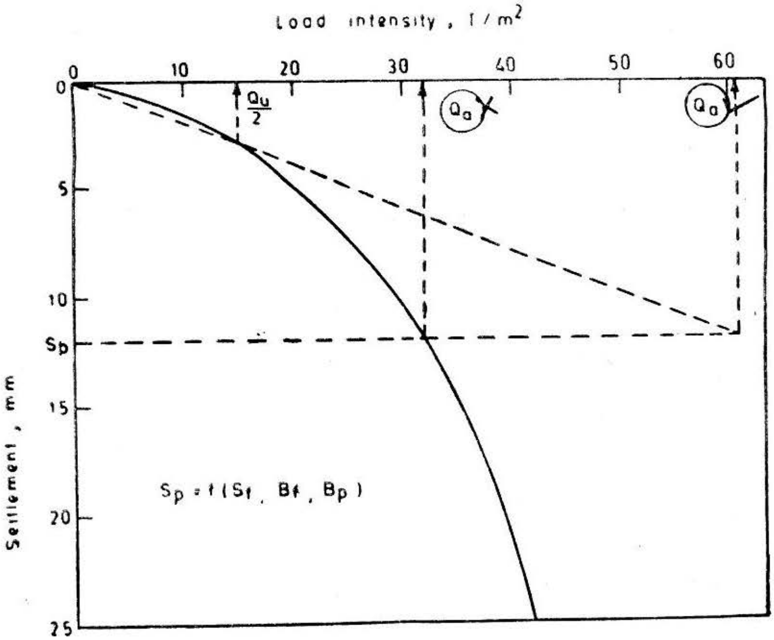


FIGURE 5 Extrapolation of Footing Settlement from Load Tests

Effect of Capillarity

The effect of capillary water in the capillary zone of a sand deposit is to increase the insitu effective vertical stress. This results in an increase in stiffness of the soil. As a consequence, a test plate resting on capillary bed undergoes smaller settlement as compared to a plate resting on dry or submerged bed. When the results of such a load test is used, it leads to under-estimation of footing settlement. Though this fact is known, no literature is available to take into account the effect of capillarity quantitatively in the estimation of footing settlement. Therefore, a simple analytical investigation was carried out and S_f/S_p vs B_f/B_p relationship (Figure 6) were obtained taking into account the effect of capillarity. The details of the investigation are reported elsewhere (Singh, 1981). The charts presented in Figure 6 can be used when the load test is conducted with the test plate resting on capillary bed and it is required to estimate the settlement of a footing resting at the same elevation as of the test plate. It is assumed that the water table would rise up to the bottom of the footing.

The charts presented in Figure 6 should be considered tentative till such time results based on a more refined analysis supported by adequate field and laboratory evidence become available.

The use of the charts presented in Figure 6 requires data on the thickness of capillary zone (or capillary head). Kezdi (1974) reports that the capillary head can be of the order of 2 to 3 m in silty sand. There are other empirical data on capillary head based on grain size (Lambe and Whitman 1969; McCarthy and David, 1977). These can aid a priliminary estimate

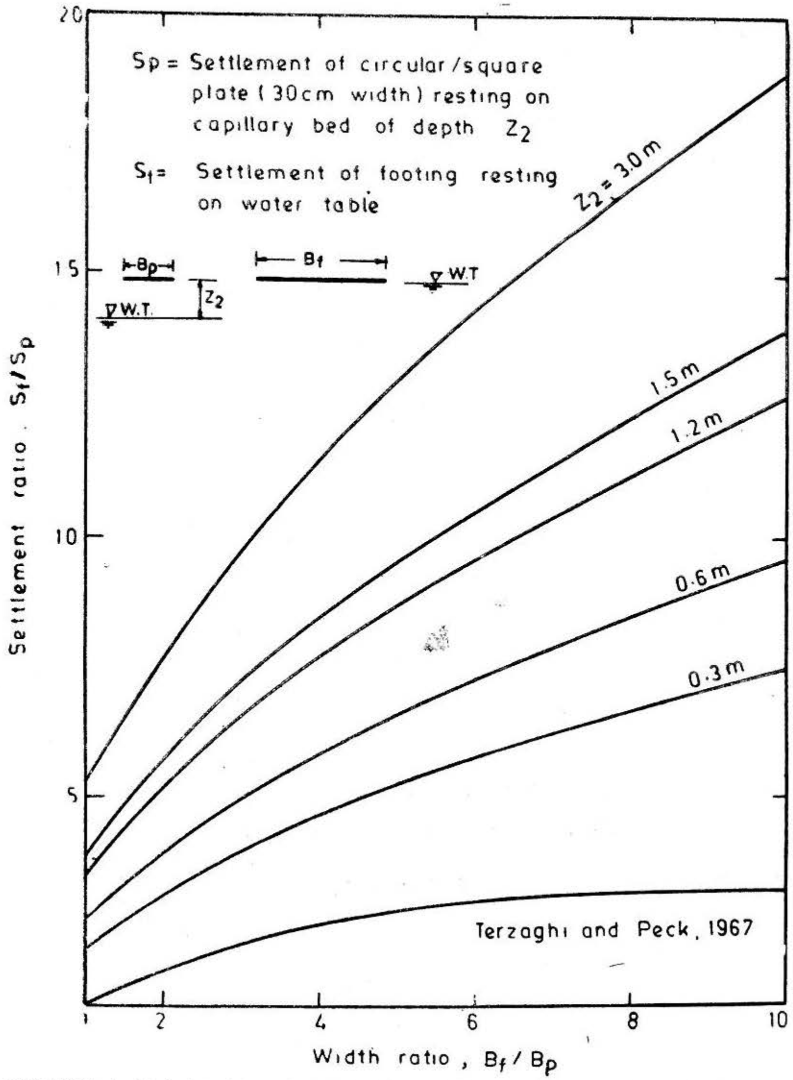


FIGURE 6 Relationship Between S_f/S_p and B_f/B_p for Square/Circular Footing

of possible capillary head for a given site. The capillary head depends on factors other than grain size such as the nature of the fines and other impurities present in soil and the manner in which the capillary equilibrium is achieved (Lambe, 1951; Kezdi, 1974). Therefore, wherever it is suspected that the test plate is resting on capillary bed, the capillary head should be determined from the data on variation of water content with depth upto water table. Samples may be collected from ground level upto the water table for water content determination. The degree of saturation may be calcutated and the same may be plotted as a function of depth as shown in Figure 7. The point 'A' in Figure 7 where the curve tends to be vertical is the elevation upto which the capillary water exists (Lambe, 1951). Therefore, the distance from point '*A*' to the free water surface may be taken as the capillary head.

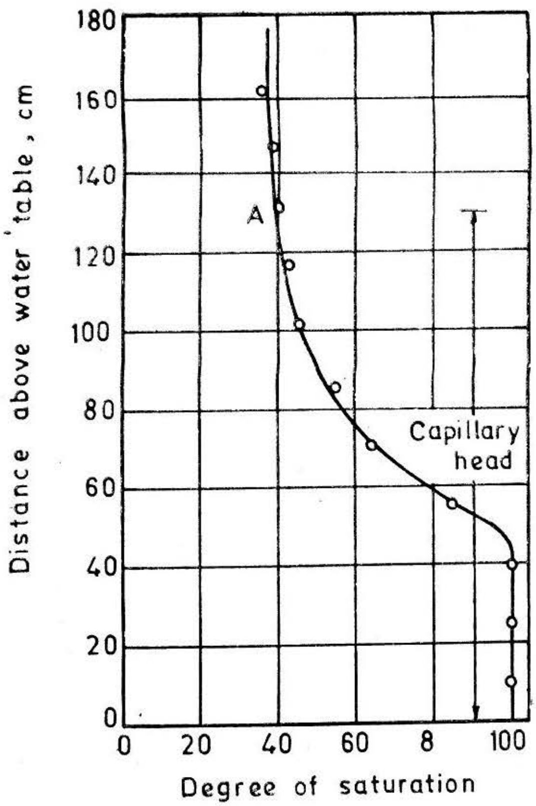


FIGURE 7 Determination of Capillarry Head.

Effect of overconsolidation

The stiffness of sand deposits is generally assumed to be mainly a function of relative density. However stress history affects the stiffness of sand and this appears to have not received due attention. The sand deposits could be overconsolidated and the settlement behaviour of footings could be significantly influenced by the overconsolidation pressure. It is suggested in the literature that one of the possible reasons for the observed settlement being more than that predicted using Eq. 1 is possibly due to the effect of overconsolidation. However, there is no quantitative information on the effect of overconsolidation on footing settlement or on S_f/S_p vs B_f/B_p relationship. The possible effect of overconsolidation on S_f/S_p vs B_f/B_p is discussed below.

In the case of overconsolidated or preloaded sand beds the insitu confining stress (lateral stress) can be even more than the vertical stress. Therefore the stiffness of an overconsolidated sand bed will be more than that of a normally consolidated sand bed of the same relative density. As a first approximation, the variation in stiffness in overconsolidated sand deposits could be assumed to be inbetween the one corresponding to normally consolidated sand bed where the stiffness increases with depth and the one corresponding to heavily overconsolidated sand bed where the stiffness may be almost constant with depth as shown in Figure 8.

Therefore, the S_f/S_p vs B_f/B_p relationship would be inbetween the one corresponding to normally loaded sand deposits (Case c) and the one corresponding to a deposit where the stiffness is almost constant with depth (Case a) as shown in Figure 8. Leonards (1975) also suggests that the settlement ratio would increase with the amount of preloading.

Perloff and Baron (1966), based on an approximate analysis, presented S_f/S_p vs B_f/B_p relationship, taking into account the effect of overconsolidation. The analysis assumes that the compressibility decreases with the increase in applied load intensity. This assumption is valid only in situations where the settlement is due to only overconsolidation. (The fallacy in this assumption is discussed in some detail elsewhere in this paper). However, the results indicate that the S_f/S_p values increase due to overconsolidation.

D'Appolonia et. al. (1968) show that the correlation between the values of SPT and the soil modulus, E is different for normally loaded and preloaded sand deposits (Figure 10). For a measured value of SPT,

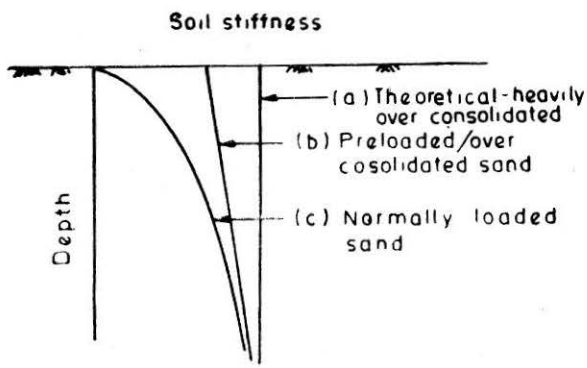


FIGURE 8 Variation of Soil Stiffness with Depth

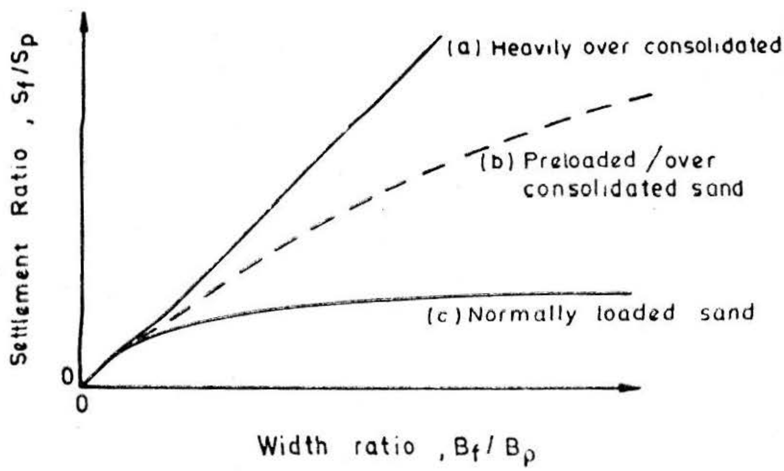
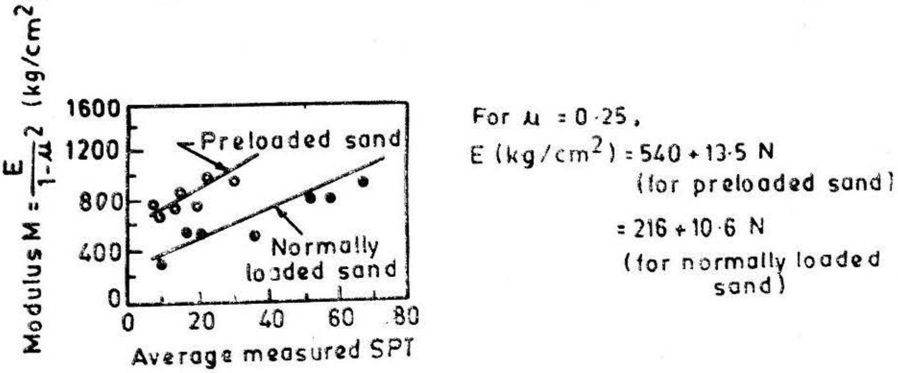


FIGURE 9 Effect of Preloading on S_f/S_p Vs B_f/B_p Relationship.



(Ater D'Appolonia, 19o8)

Figure 10 Relation Between '*N*' and Soil Modulus

the soil modulus, E is greater for preloaded sand deposits than for the normally loaded sand deposits. This, also suggested by Rowe (1975), reveals that the effect of preloading in increasing the stiffness is not reflected in the measured values of penetration resistance. Therefore, the use of settlement correlations based on the penetration tests would also result in underestimation of settlement in the case of footings resting on preloaded/overconsolidated sand beds.

The above discussion reveals that a detailed investigation is needed before the effect of overconsolidation is quantified and taken into account in the settlement computations.

Limitations of Model Tests on Prepared Sand Beds

Test on model footings on plates resting on prepared dry sand beds are quite common in research studies. The results of these studies could be quite different from what may be observed in actual practice, both quantitatively and qualitatively. However, the research workers appear to overlook the above. To illustrate the lack of agreement between research and practice, some examples are discussed in the following paragraphs.

Load Settlement Curves

Figure 11 shows load-settlement diagrams presented by Terzaghi and Peck (1967) to enable estimation of relative density from the results of load tests. Results of some load tests conducted on 30 cm square plate in the field on natural deposits as well as in the laboratory on prepared dry sand beds are also given in the Figure 11. It may be seen that the results of field tests get plotted at the appropriate relative density zones whereas the results of laboratory tests indicate gross discrepancy. For example, load tests conducted on sand beds of 80 percent relative density get plotted in the zone corresponding to loose sands. The discrepancy is so great that any attempt to use the results for quantitative prediction would result in misleading conclusions.

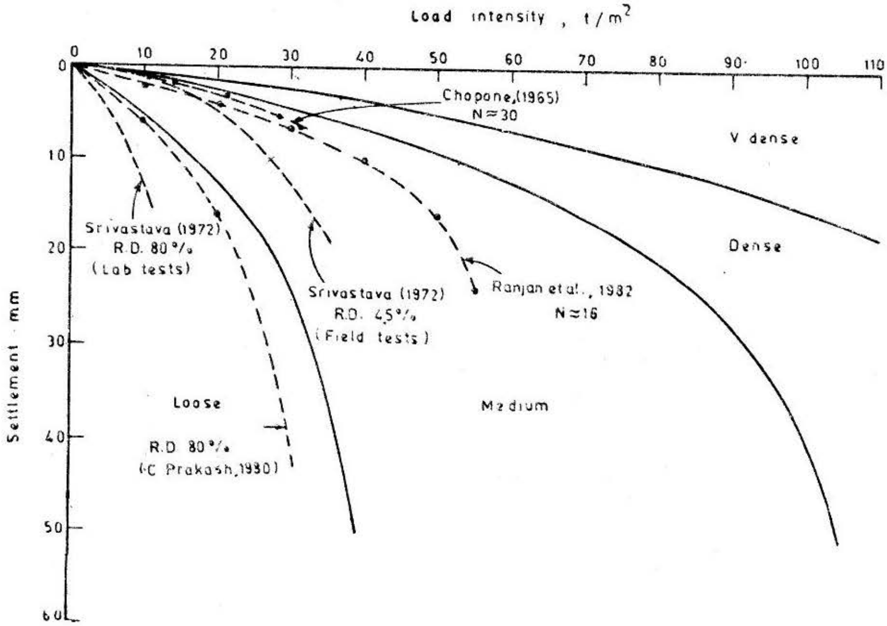


Figure 11 Load-Settlement Behaviour of Plates on Natural Ground and Prepared Sand Beds.

Effect of Embedment

An investigation to study the effect of embedment was carried out by conducting tests on 30×30 cm plates resting on prepared dry sand beds. Tests were conducted with different depths of embedment. The details of the test set up are reported elsewhere (Ramasamy et. al., 1982). The reduction in settlement due to embedment was expressed in terms of reduction factors, C_d defined as the ratio of the settlement of the test plate at depth D_f to the settlement of the plate at ground level. The values of reduction factors are plotted as a function of D_f/B_p in Figure 12.

Chopone (1965) reports the results of field load tests on circular plates of sizes varying from 30 to 105 cm at depths of 0, 54 and 190 cm below ground level. The tests were conducted in pits which are 15 cm larger than the size of the plate. The soil at site was classified as poorly graded sand of medium density. From the results of these tests, reduction factors, C_d are computed. These results are also plotted in Figure 12. It can be seen from these results that whereas the tests on dry sand beds indicate that the embedment causes considerable reduction in settlement, the field tests indicate that a relatively much smaller reduction in settlement due to embedment. For exampie, at $D_f/B_f = 1.0$, the laboratory tests indicate a reduction factor of 0.43 whereas the field tests indicate a value of 0.75. D'Appolonia et. al. (1968) also found that the settlement of a 4.5 m wide footing at $D_f/B_j = 1.0$ was roughly 75 per cent of the settlement of a footing with $D_f/B_f = 0$. The above suggests that even if the results are put in nondimensional form, the discrepancy between the field and laboratory tests could be significant.

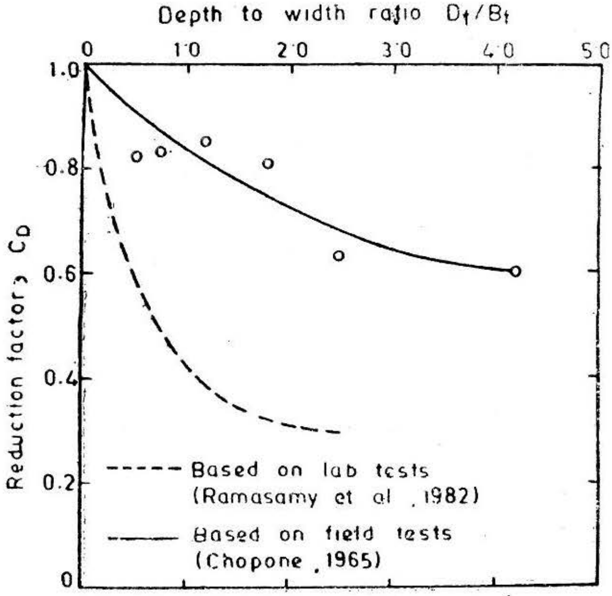


Figure 12 Effect of Embedment — Comparison of Field and Laboratory Test Results.

Small Model Tests

The limitations of small scale laboratory model tests, both for bearing capacity and settlement studies are best brought out by the results reported by De Beer (1965) and Koegler (1933). It was shown by De Beer (1965) that the bearing capacity factor, $N\gamma$ computed from the results of model tests varies with the size of the model, whereas for a sand of given physical and strength properties, it should remain constant irrespective of footing size.

Figure 13 shows the relation between the settlement and width of footing reported by Koegler (1933). It can be seen from the plots that settlement decreases with width for plates smaller than 30 cm diameter and increases with width for plates larger than 30 cm in diameter. The reasons for the above observed behaviour were discussed by Bond (1961) and Terzaghi and Peck (1967). Inspite of these wide exposure, the limitations of tests on small plates appear to have gone unnoticed by many investigators.

In the small scale model footing tests, complete similarity with the prototype is not achieved (Ovesen 1980). The most significant violation of similarity is with respect to the ratio between the stress due to the applied load and the insitu confining stress at corresponding points between the model and prototype. This results in the model behaviour not being quantitatively and at times even qualitatively similar to prototype footing. In this connections, it is pertinent to quote Terzaghi (1948):

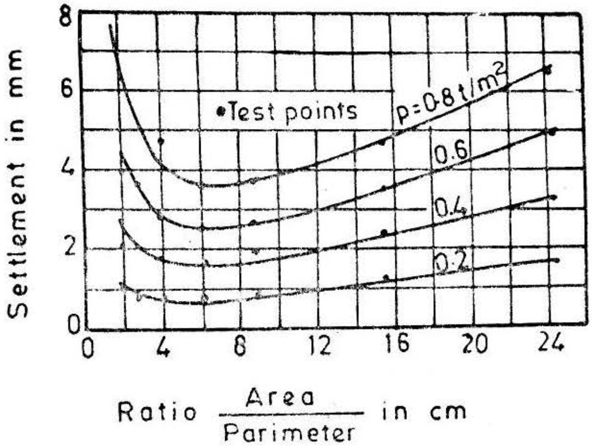


Figure 13 Relation Between Settlement and Loaded Area of Sand (after Koegler, 1933).

"Our knowledge of the difference between the behaviour of soils in the field and in the laboratory has lagged behind. Therefore, at the present time, a well documented case history should be given as much weight as ten ingenious theories and the results of laboratory investigations should not receive too much attention unless the validity of the conclusions has been demonstrated by adequate field observations on full-sized structures."

An encouraging feature about small scale model testing of foundations is the development of centrifuge model testing technique (Schofield, 1980). The technique has been used for studies on the behaviour of footings on sand (Ovesen, 1975). The technique helps to achieve stress similarity and this should help immensely in obtaining useful information from model tests.

Comments on the Method Proposed by De Beer and Martens (1957)

The method proposed by De Beer and Martens (1957) is widely referred and recommended (Tomlinson, 1975; Sutherland, 1975; IS 8009-1976) for the computation of settlement of footings on sand. In this method, the settlement is computed using the Terzaghi's consolidation settlement formula (Eq. 2).

$$S = \frac{2.3H}{C} \log_{10}\left(\frac{p_o + \Delta p}{p_o}\right) \qquad \ldots(2)$$

Where S is settlement, C is constant of compressibility, p_o is the effective overburden pressure at the depth considered and Δp is the increment of pressure at the depth due to the foundation loading and H is the thickness of the layer considered. It is obvious that Eq. 2 implies that the compressibility reduces with increasing footing load as it happens in the case of a consolidating clay layer. Therefore, Eq. 2 yields decreasing additional settlements for constant incremental loads. For example, if S_1 is the settlement for a load intensity q, the settlement for a load intensity of $2q$ would be less than $2S_1$. This is contrary to the known behaviour, Therefore, the use of Eq. 2 for settlement computation need reconsideration.

Computation of Modulus of Deformation

A simple method wherein the soil deposit is divided into a number of layers upto significant depth and the total settlement computed as the sum of the compression of all layers, is widely used for settlement computation. The compression of each layer is computed using Eq. 3.

$$\triangle S = \frac{\sigma_v}{E_S} \triangle H \qquad \ldots(3)$$

Where, σ_v is the vertical stress induced by the footing load and E_S is the modulus of deformation and $\triangle H$ is the thickness of the layer. The modulus of deformation, E_S is computed using the Eq. 4.

$$E_S \; a \; \sigma_m \qquad \ldots(4)$$

where σ_m is the mean effective stress.

In adopting the above procedure, some of the investigators (Perloff and Baron, 1976; Prakash and Puri, 1977; and Oweis, 1979) have calculated σ_m as the sum of the stress due to the insitu overburden pressure and the stress due to the footing load. This results in E_S value increasing with increasing footing load which would result in the settlements being computed in the same manner as of Eq. 2. In sands, the settlement is due to compression and shear and the modulus E_S should be a function of both shear and compression deformations. Therefore, the value of Es should in fact reduce with increasing footing load due to the nonlinear stress-strain behaviour of soils. However, in view of the fact that the settlements are computed for footing loads (design loads) which are much smaller than the ultimate loads, the modulus E_S for a given depth may be assumed to remain constant and may be computed as a function of only the mean effective insitu stresses.

Conclusions

Estimation of settlement and allowable soil pressure on footings on sand requires judicious interpretation of the results of load tests. Use of the available methods in a routine fashion without adequate knowledge of various factors affecting settlement of the plate and footing would lead to unrealistic estimates.

Capillarity in soils and preloading/overconsolidation are found to affect the settlement of the test plate and footing significantly. However, the effect of these factors is not adequately investigated and quantified for use in prediction. The available information is qualitative in nature but still a judicious use of the same could help improve prediction.

The results of the conventional small scale model tests resting on prepared dry sand beds may lead to erroneous conclusions both qualitatively and quantitatively.

The method proposed by De Beer and Martens (1957), by implication, assumes that the compressibility of sand decreases with increasing footing load. The same assumption appears to have been employed by some other investigators also in the computation modulus of deformation, E_S. The assumption leads to results which are not in agreement with the known load-settlement behaviour of footings. Therefore, the soundness of the methods based on the assumption is questionable.

Acknowledgement

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