Effect of Embedment on Settlement of Footings on Sand

by

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Introduction

The settlement of foundations resting on sand is estimated generally from the results of standard plate load test or from penetration tests. The results of plate load tests are extrapolated to obtain the settlement of actual foundations. The methods based on penetration tests, namely SPT or static dutch cone penetration test can be grouped under two catagories i.e. empirical and quasi-elastic. The correlations between plate load test results and SPT results are a basic factor in the case of empirical methods. The quasielastic methods adopt a different approach to the problem of calculating settlement in that the compressibility or equivalent elastic modulus is calculated for sand from results of penetration tests. However, in these methods also, the calculation of compressibility or equivalent elastic modulus is based on the correlations between the values of these parameters obtained from plate load tests or actual footing tests and measured penetration resistance.

A proper evaluation of settlement in sand can only come about through an understanding of the factors affecting settlement or compressibility. Jorden (1977) discussed the various factors which affect the compressibility of sand. The settlement, in general, is influenced by (*a*) loading intensity, (*b*) size and shape of loaded area, (*c*) relative density, (*d*) grain shape, (*e*) grain size distribution, (*f*) mineralogy and (*g*) in situ state of stress at the seat of settlement, which is dependent on depth of foundation or embedment depth, loading history i.e. normally or overconsolidated and position of water table. For a particular loading intensity, of all the factors, the size and shape of loaded area, relative density and in situ state of stresses are the most important. The in situ state of stress at the seat of settlement is different for a surface and an embedded footing. Hence any method of estimating settlement of footings should also take into account the effect of embedment.

In order to suggest a suitable method to account for embedment, analytical and experimental investigations were carried out and the same are reported in this paper.

Review of Methods of Estimating Settlement of Embedded Footings

Quasi-Elastic/ Elastic Solutions

Taylor (1948) assuming that the seat of settlement extends upto a depth

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of B below the base of the footing and the stress at $B/2$ below the base may be taken as average, obtained the following relationship,

$$\frac{q_1}{q_2} = \frac{S_1}{S_2}\left[\frac{1+2\dfrac{D_1}{B_1}}{1+2\dfrac{D_2}{B_2}}\right] \qquad ...(1)$$

where

S_1 = settlement for a footing of width B_1 at a depth of D_1 corresponding to a pressure intensity of q_1 and

S_2 = settlement for a footing of width B_2 at a depth of D_2 corresponding to a pressure intensity of q_2

It may be noted that as per Equation 1, for a given load intensity, i.e., $q_1 = q_2$, the size of the footing has no effect on the coefficient of settlement, q/s, for surface footings.

For a footings of width B having $D_1/B_1 = 0$ and $D_2/B_2 = D_f/B$, the settlement ratio S_2/S_1, termed hereafter as depth correction factor, C_D, corresponding to the same pressure intensity is given (from Equation 1) as follows :

$$C_D = \frac{S_2}{S_1} = \frac{1}{(1+2D_f/B)} \qquad ...(2)$$

Equation 2 implies that the settlement is reduced by 50 per cent for $D_f = B/2$. It also suggests that the reduction in settlement due to increase in D_f/B is curvilinear.

Fox (1948) obtained solutions for the relationship between depth correction factor C_D and $D_f/\sqrt{LB}$ for footings of width B and length L. Nishida (1966) (vide Poulos et. al 1974) derived the expression for vertical displacement of a flexible, uniformly loaded, embedded circular area. These solutions are based on Mindlin's equation.

Poulos (1967) obtained reduction factors based on elastic approach to calculate the settlement of a embedded flexible footing from the settlement of a surface footing.

Butterfield and Banerjee (1971), (Vide Poulos et al (1974)) presented solutions for the settlement of rigid circular and rectangular areas embedded within a semi infinite mass.

These solutions indicate that the settlement of an embedded footing reduces more rapidly in the initial range of embedment depth. The rate of reduction then decreases with the increase in D_f/B ratio and becomes more or less constant beyond a D_f/B ratio of about 2.5.

Methods Based on Plate Load Test

The extrapolation of settlement of actual footings from the results of plate load test is based on the following equation as proposed by Terzaghi and Peck 1948).

$$\frac{S_B}{S_1} = \left(\frac{2B}{B+30}\right)^2 \qquad ...(3)$$

where

S_B (cm) = settlement of footing of width B (cm) and

S_1 (cm) = settlement of test plate of width 30 cm loaded to the same pressure intensity

Equation 3 does not account for the effect of embedment in extrapolation of settlement. However, Terzaghi and Peck (1967) reported that for a footing of width B, the settlement decreases to some extent with embedment.

D'Appolonia et. al (1968) based on observations of settlement of actual footings reported that the sand compressibility decreased with increasing D_f/B ratio (Figure 1). The observed settlement of a 4.5 m wide footing with $D_f/B = 1.0$ was roughly 75 per cent of the extrapolated (using Equation 3) settlement of a similar footing with $D_f/B = 0$.

IS: 8009 (part I)—1976 suggests that the depth correction factor C_D as given by Fox (1948) should be applied to obtain settlement of embedded footings on the values extrapolated using Equation 3.

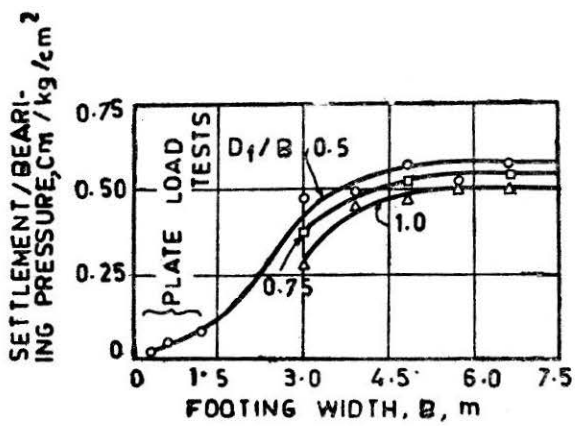

FIGURE 1 Correlation between sand compressibility, footing width and D_f/B ratio (D' Appolonia *et al* 1968)

Methods Based on Penetration Tests

Terzaghi and Peck (1948) while discussing the use of allowable pressure charts based on N values for footings on submerged sand suggested that the values obtained should be reduced by 50 per cent for small D_f/B ratios and for D_f/B ratio of unity, the reduction is to be made by one-third only. In other words this leads to a reduction in settlement by 25 per cent when $D_f/B = 1$. Subsequent investigators (D'Appolonia et al (1970), Meyerhof (1965) and Jorden (1977) have taken this to imply a depth correction factor of $(1-D_f/4B)$ upto $D_f/B = 1$ in settlement calculations.

The suggestion made by Teng (1962) to increase the allowable pressure for embedded footings implies a depth correction factor of $(1-D_f/2B)$ upro $D_f/B = 1$ with a limiting value of 0.5 beyond $D_f/B = 1$.

Peck and Bazaraa (1969) proposed the following depth correction factor:

$$C_D = 1.0-0.4\,(p/q)^{\frac{1}{2}} \qquad \ldots(4)$$

where

p (kg/cm^2) = effective overburden pressure at foundation level and

q (kg/cm^2) = applied pressure intensity on foundation

Schmertmann (1970) used the following depth correction factor in his method for estimating settlement of footing on sand.

$$C_D = 1-0.5\left(\frac{p}{q'}\right) \geqslant 0.5 \quad \ldots(5)$$

where

p (kg/cm^2) = effective in situ overburden pressure at foundation level

q' (kg/cm^2) = net load intensity at foundation level i.e.

$$q' = q-p$$

D'Appolonia et al (1970) also suggested the use of depth correction factor in their method. The values of depth correction factor suggested by them are same as those given by Fox (1948) for square footings.

It is evident from the above discussion that the effect of embedment has been considered in estimating the settlement of footings on sand by various methods including the method based on plate load test. The depth correction factor C_D as reported by various investigators is summarised in Table 1. The available values of C_D are either empirical or obtained from quasi-elastic or elastic solutions. No attempt has however been made to verify these experimentally. Hence an experimental investigation is carried out to study the effect of embedment on settlement of footings on sand. An approximate analysis similar to that proposed by Taylor (1948) to take into account the effect of embedment is also carried out. The results of these investigations are compared with the existing ones and an appropriate depth correction factor is suggested.

Approximate Analysis for Effect of Embedment on Settlement

Taylor (1948) presented an approximate analysis to take into account the effect of size and depth of embedment on settlement of footings resting on homogeneous soil assuming the stress-strain modulus, E (vertical stress/vertical strain) is proportional to the initial vertical stress at any point. However, this analysis does not exhibit the effect of width for surface footings resting on cohesionless soil.

The settlement of a footing is inversely proportional to the elastic modulus, E of the soil mass in the seat of settlement. Hence the effect of depth on settlement may be investigated by comparing the values of E for footings resting at different depths. For this purpose the value of E at a depth of one-half the footing width below the footing is considered in the present analysis. For comparison, two footings of the same width B, one at a depth of D_1 and the other at a depth D_2 are considered. If γ is the effective unit weight of the soil, the in situ normal stress at a depth $B/2$ below the footing in the two cases are given as

$$\sigma_{v_1} = \gamma\,(B/2+D_1) \text{ for the footing at depth } D_1 \quad \ldots(6)$$

and $$\sigma_{v_2} = \gamma\,(B/2+D_2) \text{ for the footing at death } D_2 \quad \ldots(7)$$

The elastic modulus E for sands is related to the initial effective

TABLE 1

Depth Correction Factor as Proposed by Various Investigators

Sl. No.	Reference	Approach used	Depth Correction Factor, C_D	Remarks
1.	Taylor (1948)	Quasielastic	$1/(1+2\ D_f/B) \geqslant 0.5$	D_f = Depth of footing B = Width of footing
2.	Fox (1948)	Elastic	Ranges from 0.93 to 0.66 for D_f/B from 0.25 to 1.5 for a square footing	The values become almost constant beyond D_f/B of 2.5. These also correspond to those suggested by D'Appolona et al 1970·
3.	Nishida (1966)	Elastic	Ranges from 0.8 to 0.54 for D_f/B from 0.25 to 1.5 for a circular flexible footing	The values correspond to Poisson's ratio of 0.3 and become almost constant beyond D_f/B of 2.5.
4.	Poulos (1967)	Elastic	Ranges from 0.86 to 0.59 for D_f/B from 0.25 to 1.5 for a circular flexible footing	——
5.	Butterfield and Banerjee (1971)	Elastic	Ranges from 0.8. to 0.57 for square rigid footings and from 0.84 to 0.58 for circular rigid footings corresponding to D_f/B from 0.25 to 1.5	The values correspond to Poisson's ratio of 0.3 and become almost constant beyond D_f/B of 2.5.
6.	Terzaghi and Peck (1948)	Empirical	$1-D_f/4B$ upto $D_f/B = 1.0$	The factor is to be used when settlement is worked out using S.P.T. (N) values from Terzaghi and Peck (1948) allowable bearing pressure charts.
7.	Teng (1962)	Empirical	$(1-D_f/2B) \geqslant 0.5$	
8.	Peck and Bazaraa (1969)	Empirical	$1-0.4\left(\frac{p}{p}\right)^{\frac{1}{2}}$	p = effective over burden pressure at foundation level q = applied pressure intensity at foundation
9.	Schmertmann (1970)	Empirical	$1-0.5\left(\frac{p}{q'}\right) \geqslant 0.5$	q' = net pressure intensity at foundation level $= (q-p)$

principal stresses according to the relationship given below (Lambe and Whitman-1969)

$$E \ \alpha \left[\frac{\sigma_v + 2K_0\ \sigma_v}{3}\right]^n \qquad \dots (8)$$

where

K_0 is the coefficient of earth pressure at rest

The value of n may vary from 0.4 to 1.0. The larger values of n tend to be applicable for loose sands. A reasonable value of $n = 0.5$ has been suggested. Also Equation 8 with $n = 0.5$ may be taken to hold good when $1/2 < K_0 < 2$ and when the factor of safety against failure is 2 or more (Lambe and Whitman 1969).

Let the settlement of the footing at depth D_1 and that of the footing of same width at depth D_2 be S_1 and S_2 respectively, from Equation 8.

$$S_1 \propto \frac{1}{E_1} \text{ i.e., } \frac{1}{\left[\dfrac{\sigma_{v_1} + 2K_0\, \sigma_{v_1}}{3}\right]^n} \quad \dots (9)$$

and

$$S_2 \propto \frac{1}{E_2} \text{ i.e., } \frac{1}{\left[\dfrac{\sigma_{v_2} + 2K_0\, \sigma_{v_2}}{3}\right]^n} \quad \dots (10)$$

where

E_1 and E_2 are the elastic moduli of soil for the footing at depth D_1 and the footing at depth D_2 respectively.

Substituting Equation 6 and 7 in Equation 9 and 10, the ratio S_2/S_1 for a given pressure intensity is given by

$$\frac{S_2}{S_1} = \left[\frac{1+2D_1/B}{1+2D_2/B}\right]^n \quad \dots (11)$$

For a particular case when $D_1 = 0$ (surface footing) and $D_2 = D_f$

$$\frac{S_2}{S_1} = \left[\frac{1}{1+2D_f/B}\right]^n \quad \dots (12)$$

The ratio of settlements S_2/S_1 (Equation 12) is termed as depth correction factor C_D. For $n = 0.5$, the values of C_D corresponding to various D_f/B ratios are given in Figure 2.

Figure 2 indicates that the settlement for an embedded footing reduces rapidly in the initial range of D_f/B. The rate of reduction decreases for larger values of D_f/B and C_D becomes constant for all practical purposes beyond D_f/B of about 2.5. This is in line with other elastic solutions for embedded footings (Nishid 1966), Poulos (1967) and Butterfield and Banerjee (1971). Equation 12 with $n = 1$ becomes the same as that obtained by Taylor (1948).

Extrapolation of Plate Load Test Data Accounting for Effect of Embedment

The plate load test is generally carried out at the proposed depth of foundation in a pit five times the width of the test plate. It is assumed that the removal of surcharge does not significantly alter the confining pressure below the plate and hence will have no effect on observed settlement of test plate. However, as suggested by Equation 8, the stiffness of soil is a function of the effective in situ normal and lateral stresses. Therefore the computed settlement of a footing using Equation 3 from the results of a plate load test may be taken as the settlement of a footing for which the surcharge is removed as in a plate load test. The depth at which the load test is carried out and the depth at which the foundation is laid may also differ due to some reason or the other. Hence the analysis described earlier is modified to take into account the effect of the removal of surcharge in a plate load test.

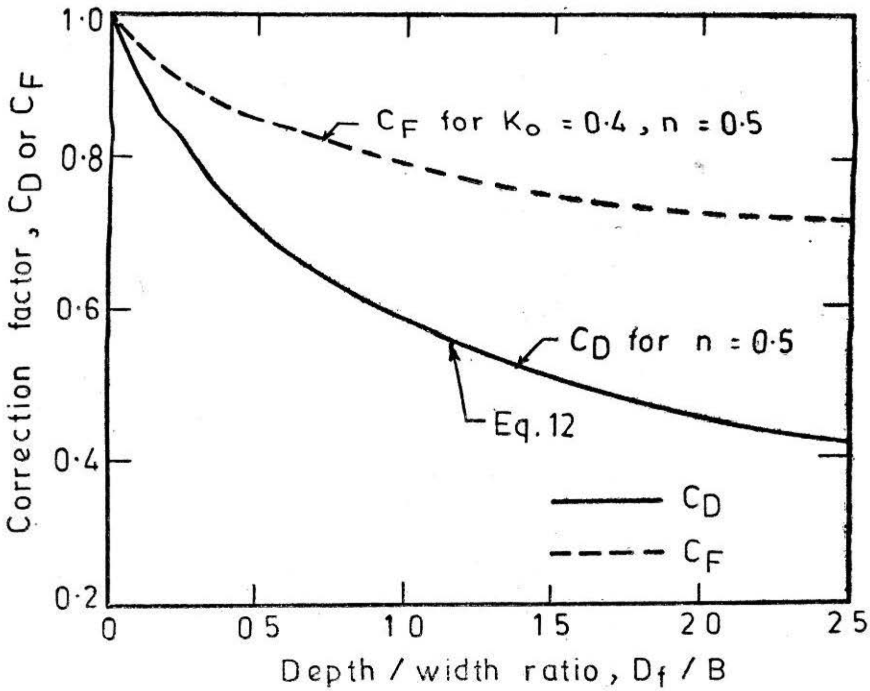

FIGURE 2 Variation of correction factors C_D or C_F with D_f/B

Considering a footing of width B at depth d_1 where the surcharge is removed and a footing of same width at depth d_2 for which no surcharge is removed, the settlement ratio may be obtained as

$$\frac{S_2}{S_1} = \left[\frac{\gamma\, B/2+2K_0\, \gamma\, (d_1+B/2)}{(d_2+B/2)+2K_0\, (d_2+B/2)}\right]^n$$

$$= \left[\frac{1+2K_0+4K_0\, d_1/B}{(1+2K_0)\,(1+2d_2/B)}\right]^n \qquad \text{... (13)}$$

where S_1 and S_2 are settlements of footings at depth d_1 and d_2 respectively.

Equation 13 is obtained assuming that the removal of surcharge does not alter the in situ horizontal stress at death $B/2$ below the footing. Equation 13 reduces to 12 for $d_1 = 0$ and $d_2 = D_f$. The effect of removal of surcharge can be studied by making $d_1 = d_2$. The ratio S_2/S_1 for $d_1 = d_2 = D_f$ may be taken as the correction factor C_F to be applied to the extrapolated settlement of a footing from standard plate load test data using Equation 3. The variation of C_F as function of D_f/B is shown in Figure 2 for $K_0 = 0.4$ and $n = 0.5$.

Experimental Investigation

Tests were conduced on a 30 cm square mild steel plate of D_f/B ratios of 0, 0.25, 1.0, 1.5, 2.0 and 2.5. The surface of plate in contact with soil was rough simulating roughness of actual footings. The ratio $D_f/B = 0$ corresponds to a surface footing (without embedment) and others to embedded footing. The soil used was air dried solani river sand classified as '*SP*' as per IS : 1498-1970. Its grain size distribution is shown in Figure 3. The physical properties are reported in Table 2. The tests were conducted on dry sand at two densities; 1.63 g/cc and 1.54 g/cc, corresponding to relative density, D_r of 80 per cent and 60 per cent respectively. Sand was filled by the rainfall technique to achieve the desired density. The values of angle of shearing resistance ϕ, reported in Table 2 were determined from direct shear tests.

TABLE 2

Properties of Sand used in Tests

Sl. No.	Description	State	
		Dense	Medium
1.	Maximum dry unit weight (g/cc)	1.76	1.76
2.	Minimum dry unit weight (g/cc)	1.28	1.28
3.	Unit weight of test bed (g/cc)	1.63	1.54
4.	Void Ratio		
	(*i*) Minimum	0.51	0.51
	(*ii*) Maximum	1.07	1.07
	(*iii*) Natural	0.63	0.72
5.	Relative density (%)	80	60
6.	IS Classification	*SP*	*SP*
7.	Uniformity coefficient	1	2
8.	Coefficient of curvature	1.3	1.3
9.	Angle of sharing resistance (degrees)	35.5	33.5
10.	Specific gravity of soil grains	2.65	2.65

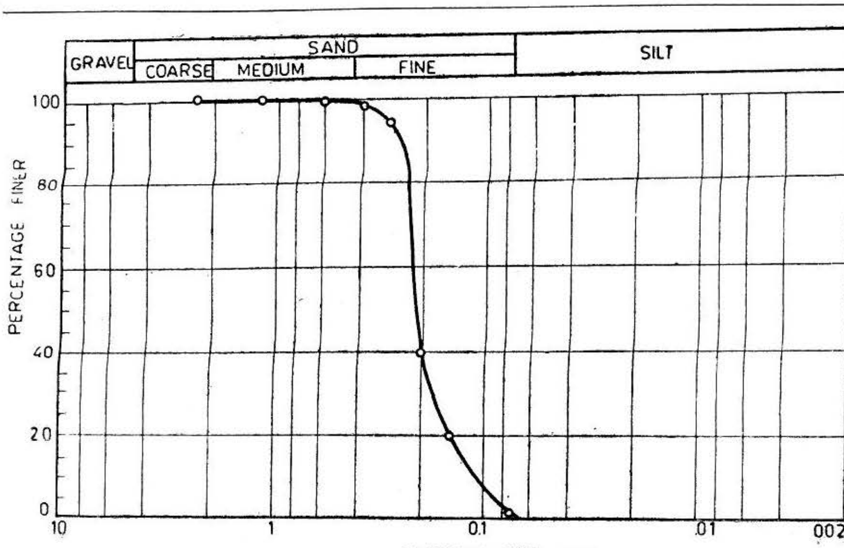

FIGURE 3 Grain size distribution of sand

The tests were conducted in a pit of 1.5 m $\times$ 1.5 m $\times$ 2.0 m depth. The test plate was kept at the centre of the pit over 1 m thick layer of sand. The loads were applied through a hydraulic jack reacting against a RSJ clamped down to the ground by hold fast type anchors. Four dial gauges of 0·01 mm sensitivity were used to record settlements. These were held through mangetic holders attached to datum bars resting on immovable supports sufficiently away from the pit. The test plate was 1 m below the top of pit. Four mild steel bolts of 18 mm diameter and about 1.2 m length were fixed to the nuts welded at four corners of the test plate. On the top of these bolts small pieces of plates were screwed. The top of dial gauges were made to rest on glass pieces pasted to the small plates.

For filling the sand above the level of test plate to the required height corresponding to a particular D_f/B ratio, a 31 cm square and 110 cm long hollow box made from 2 mm thick mild steel sheets was used. At the bottom an angle iron of 25 mm $\times$ 25 mm was welded along the periphery in order to avoid the penetration of the box into the sand. The general view of the set-up with the hollow box is shown in Figure 4 and a close up

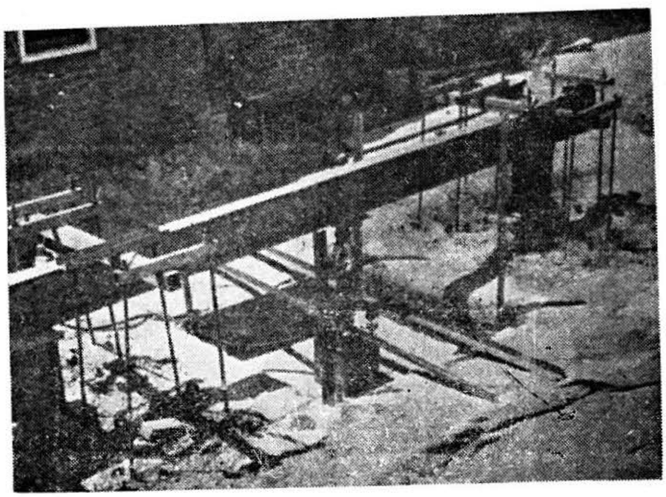

FIGURE 4 Load test set up for embedded plate

view in Figure 5. The following sequence was followed in placing the sand to obtain a particular D_f/B ratio: first, 1 m thick layer was filled. The test plate was next placed at the centre of pit on this layer. Then the hollow box was kept around the test plate. The sand was then poured through a sieve having a 32 cm square central cut. The cut was made so that the sieve could be moved when the hollow box was in position to achieve the desired height of fall. When the sieve was above the hollow box, a lid was placed on top of the hollow box to avoid falling of sand on the test plate.

The load was applied in equal increments. The magnitude of load increment was decided so that a sufficient number of points would be available to describe the load-settlement curve. Each load increment was maintained till the rate of settlement was less than 0.02 mm per minute or for a maximum period of one hour and the final settlement noted.

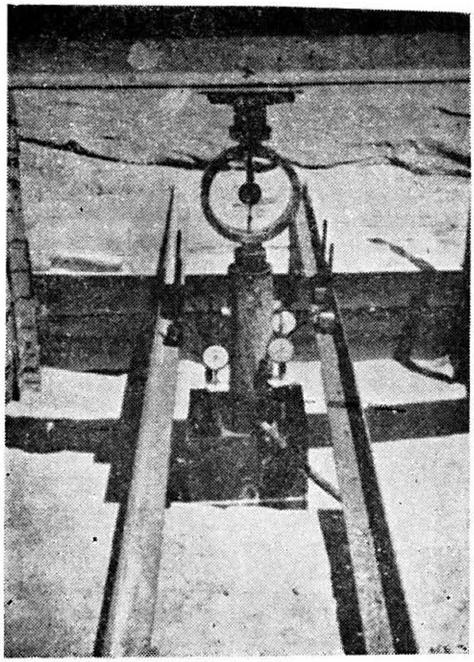

FIGURE 5 Close up view of load test set-up for embedded plate

Results and Discussion

Load-Settlement Behaviour of Embedded Test Plates

The pressure intensity versus settlement plots for the test plate resting on dense sand (relative density of 80 per cent) are given in Figure 6. It reveals that embedment reduces settlement for a given pressure intensity significantly upto $D_f/B = 1.5$. Beyond $D_f/B = 1\ 5$, the embedment does not seem to have any further advantage in terms of reduced settlement.

The reduction in settlement of embedded test plates can be attributed to the decrease in compressibility or increase in soil modulus with increase in situ mean effective normal stress resulting from embedment.

The pressure intensity-settlement curves for embedded test plate exhibit a more or less linear range upto larger pressure intensities in comparison to the pressure intensity-settlement curve for test plate with no embedment ($D_f/B = 0$). This so called linear range increases with increase in D_f/B. This behaviour may also be due to the stiffness induced by embedment, which increases with increase in D_f/B.

The results of plate load tests on sand having relative density of 60 per cent are given in Figure 7. It is evident that the behaviour exhibited by test plate embedded in loose sands in the general similar to that observed for test plate in dense sands.

Relation Between D_f/B and Settlement Ratio s_m/s_0

Figure 8 and Figure 9 present the variation in settlement with increase in D_f/B for different constant pressure intensities, corresponding to

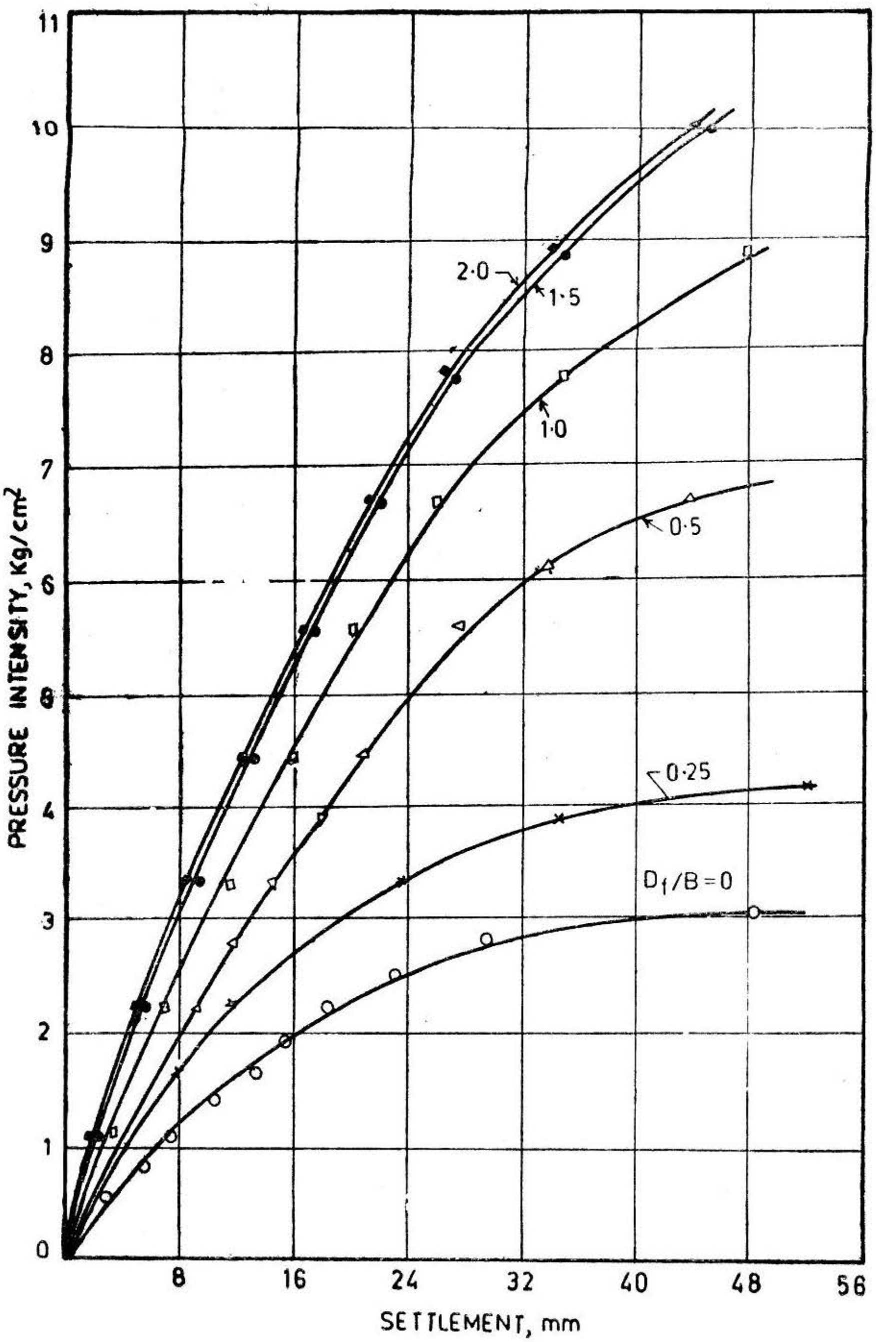

FIGURE 6 Pressure intensity Vs settlement curves for various D_f/B ratios ($D_f/$ = 80 per cent)

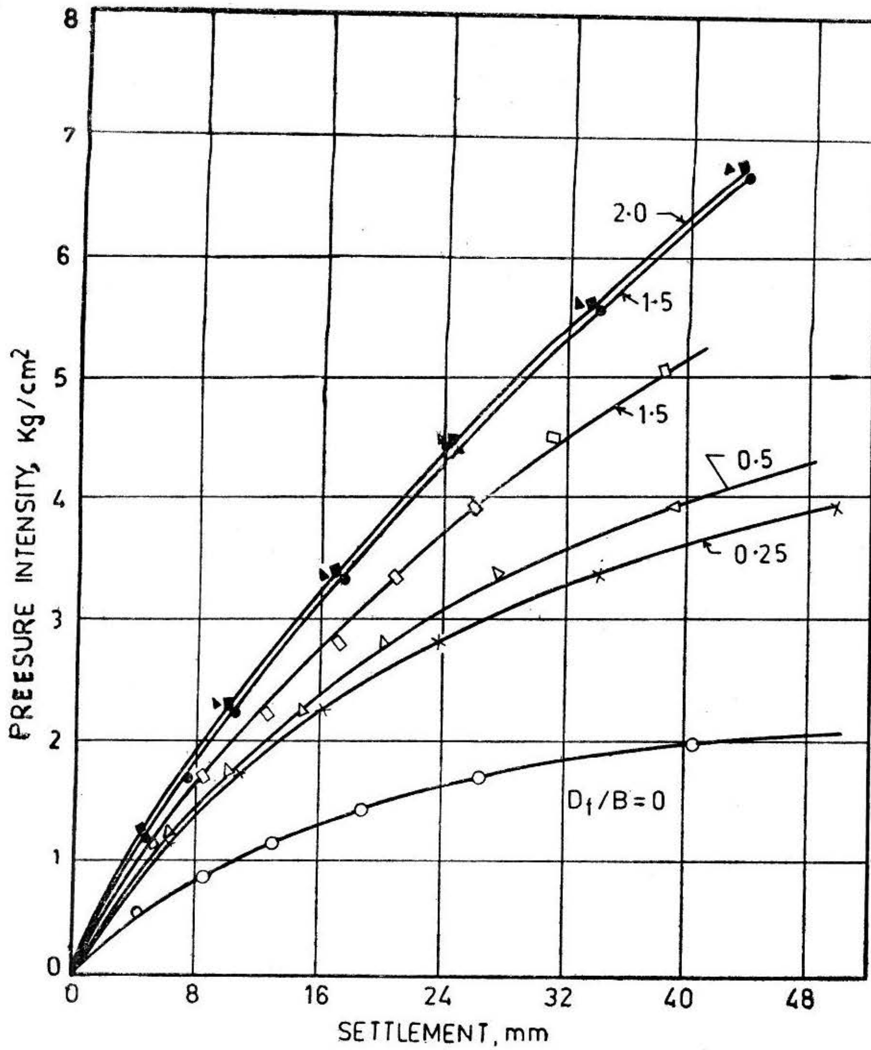

FIGURE 7 Pressure intensity Vs settlement curves for various D_f/B ratios (D_r = 60 per cent)

D_r = 80 per cent and D_r = 60 per cent respectively. They indicate that the rate of reduction in settlement is appreciable initially but becomes insignificant for D_f/B ratio higher than 2.0. This observed trend is in line with the elastic solutions and the analytical solution proposed by the authors.

The observed decrease in settlement owing to embedment becomes appreciable at higher pressure intensities. It is particularly so for pressure intensities greater than about 50 per cent of ultimate bearing capacity (This 50 per cent can be considered the limit for the linear range of pressure intensity-settlement curve, Rao and Ramasamy 1979), of test plate at

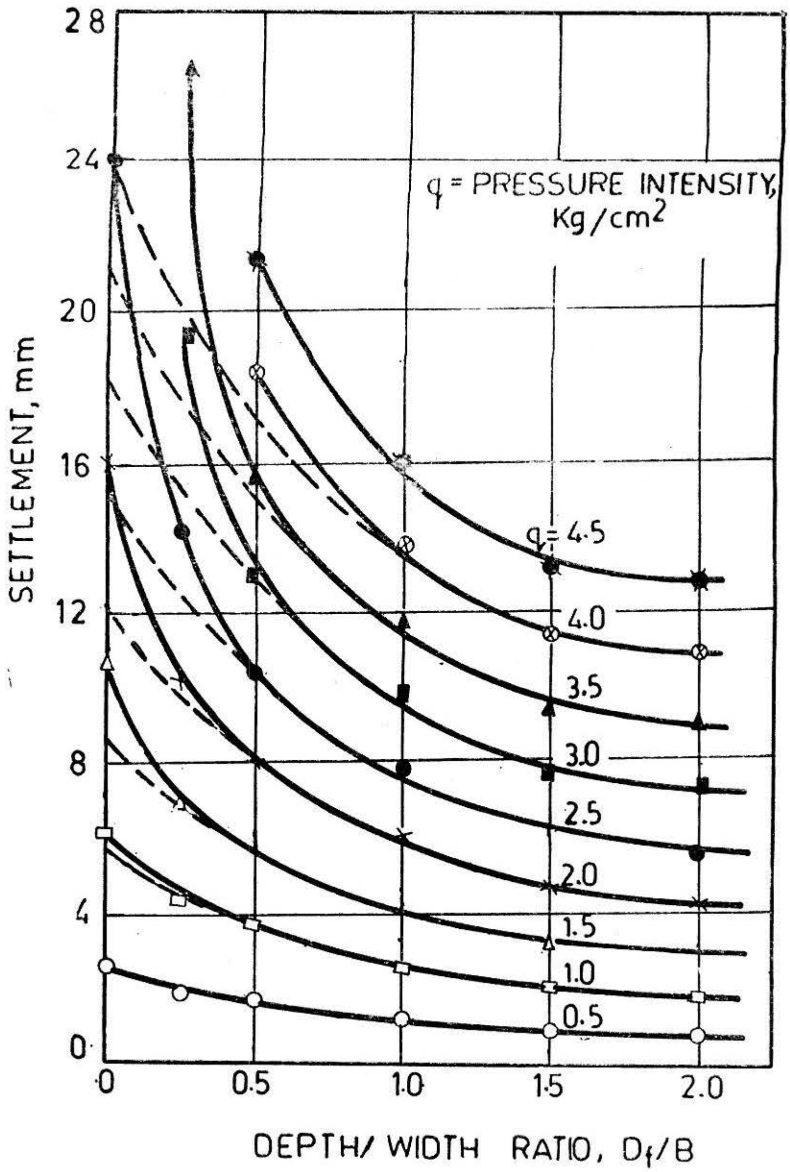

FIGURE 8 D_f/B **Vs settlement for different pressure intensities** (D_r = **80 per cent)**

$D_f/B = 0$, viz., 1.5 km/cm² and 1.0 km/cm² for relative densities of 80 per cent and 60 per cent respectively. It is because of the fact that while the soil approaches the plastic stage beyond these pressure intensities for $D_f/B = 0$, it remains more or less within elastic range upto much larger pressure intensities for test plate with embedments.

If the load-settlement behaviour were such that the soil is within elastic range even under larger pressure intensities in the case of $D_f/B = 0$, the s_0 values would have been smaller than those obtained experimentally. For any meaningful evaluation of the effect of embedment, s_0 values obtained under elastic conditions should be used. An attempt has been made to obtain these values by extrapolation as shown by dotted lines in Figure 8 and 9. In making the above extrapolation, the following guide lines have been followed:

(*i*) Deviation from the observed curve is effected from the point corresponding approximately to the limit for linear range of pressure intensity settlement curves.

(*ii*) The slope of the extrapolated portion of the curve is guided by the slope as that of the curve for the next lower pressure intensity.

The values of settlement ratio, S_m/S_0 (S_m—settlement of an embedded test plate and S_0—settlement of a test plate with no embedment) have been computed for various values of D_f/B and pressure intensities. The ratio, S_m/S_0 is the same as the depth correction factor C_D used earlier. The variation of C_D with D_f/B is shown in Figure 10 and Figure 11 for sand at

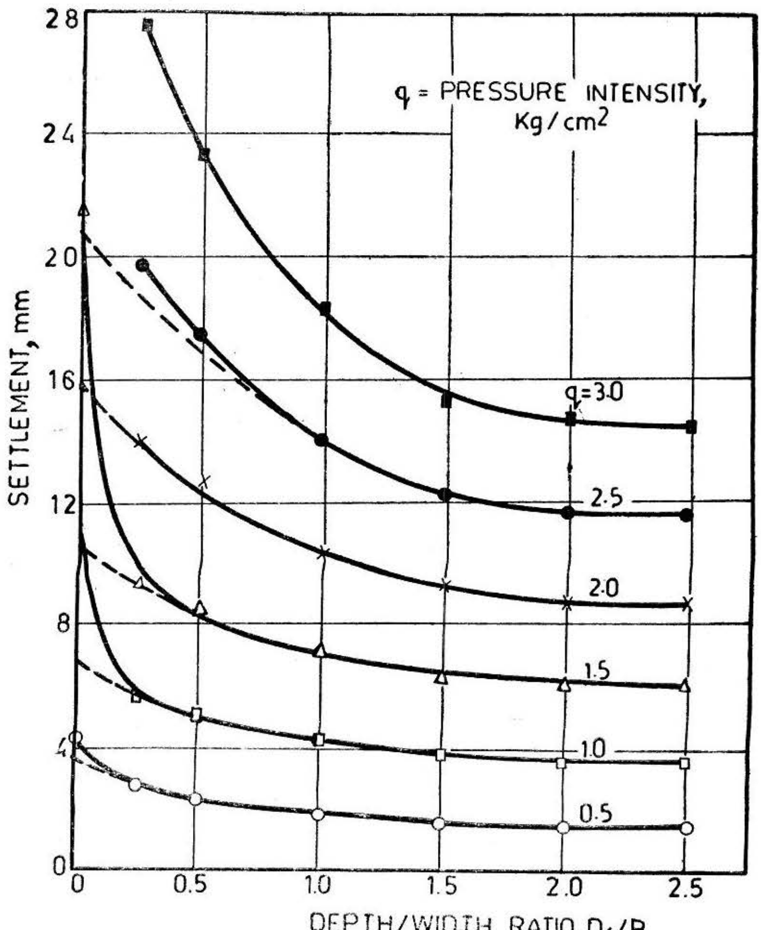

FIGURE 9 D_f/B **Vs settlement for different pressure intensities** (D_r = **60 per cent)**

$D_r = 80$ per cent and $D_r = 60$ per cent respectively. The curves have been drawn giving more weightage to the points which correspond to pressure intensities upto 1.5 kg/cm² for dense sand and upto 1.0 kg/cm² for loose sand; The solid line curves are based on observed values of S_m and S_0 and line curves are based on extrapolated values (dotted lines in Figures 8 and 9) of S_m and S_0.

A comparison of values of C_D obtained from experimental data (Figure 10 and 11) with those given by the Equation 12 for two values of n, viz., 1 and 0.5 is shown in Figure 12. It can be seen that the trend of variation of C_D with D_f/B is similar in both the cases viz., as obtained from experimental data and from Equation 12. The curves obtained from experimental data fall more or less within those obtained from the analytical solution for values of $n = 0.5$ and 1.0.

These results indicate that the values of C_D obtained from Equation 12 for $n = 0.5$ will provide a conservative estimate. In view of various factors affecting settlement in sand, it is advisable to use a conservative approach and accordingly the values of C_D given by Equation 12 may be used in practice.

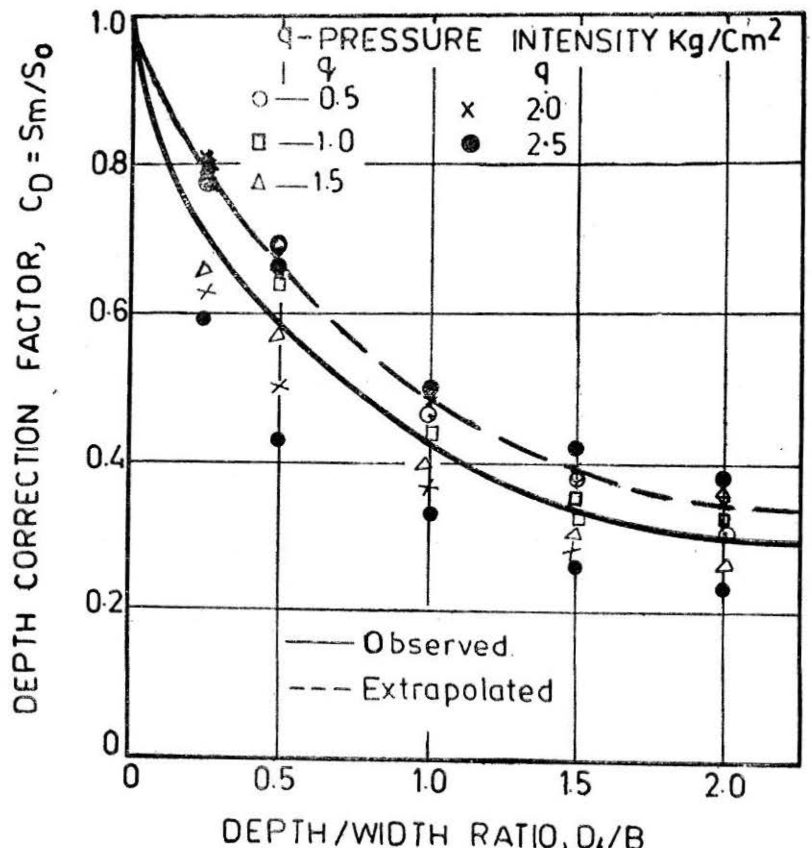

FIGURE 10 Relationship between D_f/B and C_D (D_r = 80 per cent)

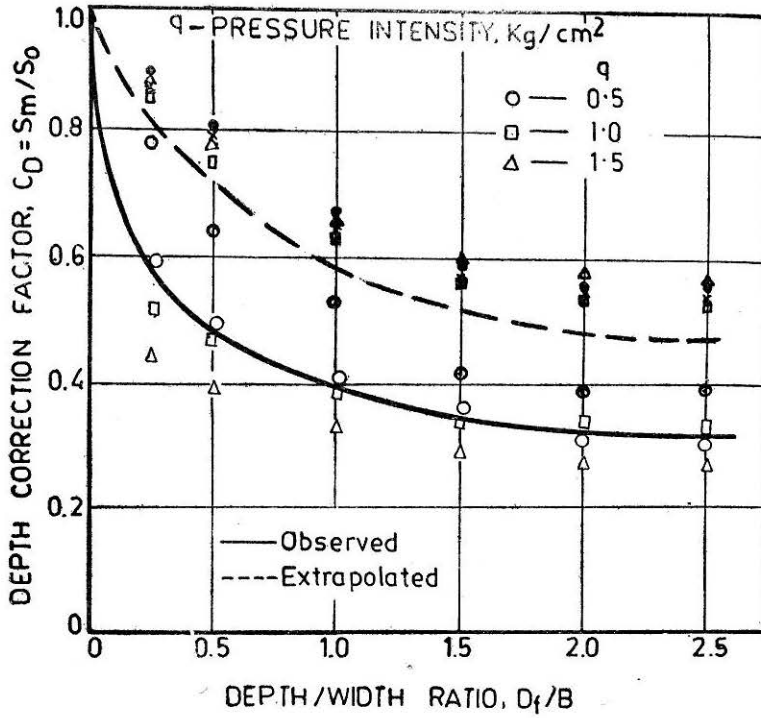

FIGURE 11 Relationship between D_f/B and C_D (D_r = 60 per cent)

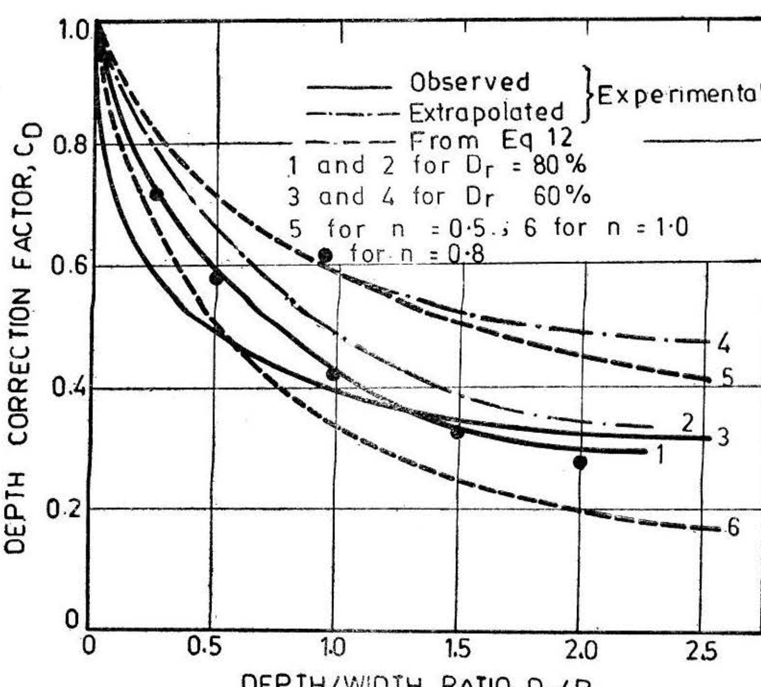

FIGURE 12 Comparison of experimentally obtained C_D values with those obtained analytically

Comparison of C_D Values Proposed by Various Investigators and Those Obtained Experimentally

Figure 13 shows the comparison of experimentally obtained values of C_D and those suggested empirically by various workers (Terzaghi and Peck (1948), Meyerhof (1965), Teng (1962), Peck and Bazaraa (1969) and Sehmertmann (1970); reported in Table 1). The comparison reveals that all the methods make conservative prediction of C_D when compared to those experimentally observed. The values of C_D suggested by Peck and Bazaraa (1969) and Schmertmann (1971) which do not incorporate the D_f/B ratio but only include the prrameter D_f, are over conservative. Out of all the four methods, C_D as proposed by Teng (1962) comes closer to the experimentally obtained values.

A similar comparison of experimentally obtained values of C_D with those obtained from elastic solutions given by Fox (1948), Nishida (1966), Butterfield and Banerjee (1971) and Poulos et. al (1974), reported in Table 1 is shown in Figure 14. The values of C_D obtained from the Equation 12 are also shown in this plot.

The predicted trend of variation of C_D with D_f/B from the elastic solutions and that obtained experimentally is almost the same. However,

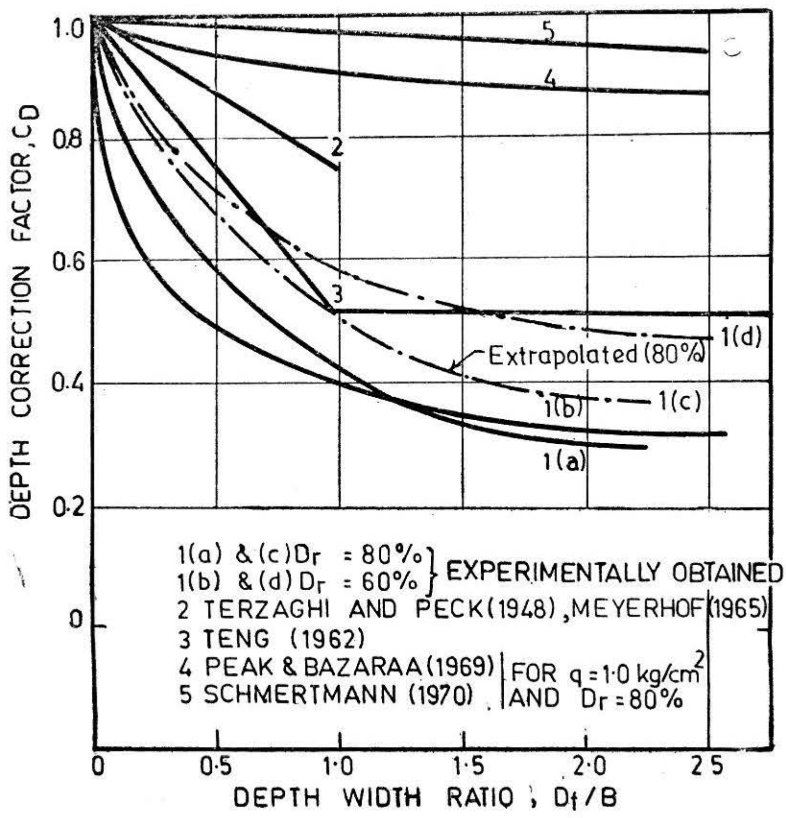

FIGURE 13 Comparison of experimentally obtained C_D values with those obtained on existing empirical relationships

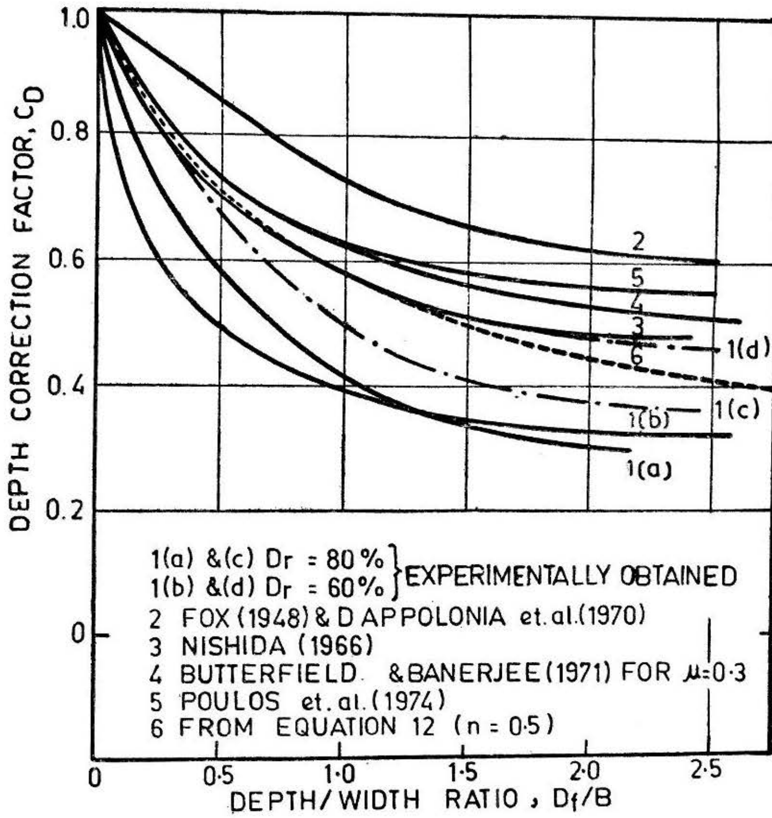

FIGURE 14 Comparison of experimentally obtained C_D values with those obtained from elastic solutions

the predicted values are conservative compared to those experimentally obtained. The values of C_D as proposed by Fox (1948) are the most conservative. This comparison (Figure 14) suggests that the elastic solutions by Nishida (1966) Butterfield and Banerjee (1971), Poulos (1974) *and Equation 12 may provide a reasonably* conservative estimate of settlement of embedded footings. It may be seen that Equation 12 with $n = 0.5$ gives values of C_D which are closest to the experimentally observed values as compared to the above elastic methods.

Conclusions

Based on the work reported herein, the following conclusions have been arrived at. These conclusions are applicable for footings on normally consolidated homogeneous sand.

1. The surcharge resulting from embedment reduces settlement of footings on sand. The reduction in settlement—the rate of reduction being diminishing in nature—is significant upto $D_f/B = 1.5$ and any additional embedment brings about practically no further reduction in settlement.

2. The analytical and experimental investigations carried out suggest that the depth correction factor C_D, as given by Equation 12 with

$n = 0.5$ may be used to estimate settlement of embedded footings on sand from the settlement of a footing with $D_f/B = 0$. Comparison with various available methods also suggests that the depth correction factor given by Equation 12 with $n = 0.5$ will provide a reasonable and conservative estimate of settlements of embedded footings on sand.

3. To account for the removal of surcharge in a plate load test, the following procedure may be applied;

   (*a*) If a load test is conducted at the proposed depth of foundation, the settlement of the test plate may be extrapolated using Equation 3 to obtain the settlement of the footing and then the correction factor C_F as given by Equation 13 for $d_1 = d_2 = D_f$ may be applied to account for the removal of surcharge and embedment.

   (*b*) If the depth of foundation is other than the depth at which the load test is conducted, the settlement of the footing may be obtained by extrapolation using Equation 3 and then the correction factor as given by Equation 13 with actual values of d_1 and d_2 may be applied to account for the removal of surcharge and embedment.

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