Drag-Load on Piles in Normally Consolidated Clay

by

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Introduction

While the primary function of friction piles in clay is to transfer their load on to the soil through skin friction, under certain circumstances the phenomenon reverses itself and the subsidence of the soil around the pile induces downward shear stresses on the mantle of the pile shaft thus imposing additional load on the pile. This phenomenon is popularly known as 'negative skin friction' or 'downdrag' on piles. Some of the circumstances which cause the relative settlement of soil around the piles are, regional subsidence as found in Mexico, the underconsolidated nature of the deposits in which the piles have been installed and an increase in the vertical effective stress either by the addition of a surcharge or by excessive dewatering operations. The realisation of such occurrence of drag-load on piles dates as far back as to 1914 (Aldrich, 1970), even before the science of soil mechanics developed. However, investigations on the problem of drag-load gained impetus only in the last decade. Because of its nature of reducing the safe working load of the piles and increasing their settlement, the phenomenon is even referred to as a 'silent enemy' of pile foundations (Van Veele, 1964). The problem, of late, has attracted the attention of many investigators and several reports from field measurements (Van Veele, 1964; Johannessen, 1965; Johannessen and Bjerrum, 1965; Bjerrum, Johannessen and Eide, 1969; Bozozuk and Labrecque, 1969; Endo et al, 1969; Bozozuk, 1972; Fellenius, 1972) on full size piles reveal the enormous magnitudes of drag-load to which piles can be subjected. The measured drag-loads ranged from 30 tons (Fellenius, 1972) to as high as 654 tons (Bozozuk and Labreque, 1969). It may not be out of place to mention that an entire speciality session had been devoted to discussions on the investigations of this particular problem in the 7th International Conference held at Mexico in 1969.

Among the various factors that would affect the ultimate drag-load on a pile, the primary ones could be listed as: (*i*) the stress history and consolidation characteristics of the soil, (*ii*) the intensity and nature of the surcharge imposed on the soil, (*iii*) the shear resistance mobilisation characteristics at the pile-soil interface, (*iv*) the end bearing conditions at the tip of the pile, (*v*) the compressibility characteristics of the pile and (*vi*) the influence of pile installation on the soil. A rigorous procedure should suitably incorporate all these factors in the analysis. Whereas the drag-load phenomenon

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is due to subsidence of the soil around the pile, the magnitude of subsidence is governed by the factors (*i*) and (*ii*) stated above. The shear stress mobilisation characteristics at the pile-soil interface is influenced by the surface finish of the pile and by the process of consolidation. The striking influence of surface finish of pile is revealed from the investigations of Bjerrum, Johannessen and Eide (1969) where an application of as thin as 1 mm thick coating of bitumen was found to result in the drastic reduction of drag-load on a pile to values less than 10% of the values observed on a similar but uncoated pile. The shear resistance mobilisation characteristics varies with time, for as the consolidation progresses the shear strength of the soil increases. Further, the shear stress mobilisation characteristics will vary along the length of the pile (due to increasing overburden). The location of the neutral point (point of zero relative movement between pile and soil) is *influenced by the end bearing conditions at the tip of the pile.* The maximum drag-load on the pile and the distribution of drag-load along the length of the pile are in turn affected. The compressibility characteristics of pile, like the end bearing conditions, will affect the relative movement between the pile and soil. The drag-load mobilised will tend to reach zero as the ratio of the stiffness of the pile to that of the soil approaches unity and it will increase with an increase in the value of relative stiffness. But, in general, the stiffness of the pile in relation to that of the surrounding soft clay is high enough to be justifiably considered as rigid without loss of any accuracy. Displacement piles in general and driven piles in particular tend to destory the soil fabric and remould the soil near the periphery of the pile, inducing excessive pore pressures. This affected zone, on dissipation of the pore pressure consolidates and induces a drag-load on the pile. However, quantification of the phenomenon is still eluding the researchers. Further, in cases of non-displacement type of piles, the influence of installation operations are not likely to be significant.

A perusal of the literature indicates that very simple to highly complex theoretical approaches have been so far suggested for the calculation of drag-load on piles. Probably the earliest analytical approach suggested is due to Terzaghi and Peck (1948) who assumed the full mobilisation of the soil shear strength at the pile-soil interface along the entire length of the pile. Since then many an analytical solution to the problem have been suggested by various investigators (Zeevaert, 1959; Buisson, Ahu and Habib, 1960; Silva, 1966; Johnson and Kavanagh, 1968; Poulos and Mattes, 1969; Walker and Durvall, 1970; Mazurkiewicz, 1971). Whereas the simpler of the solutions overlook many of the factors that play a vital role in the phenomenon, the more complex ones are too involved for easy comprehension and adoption in field practices. Apart from the analytical solutions, Elmasry (1963), Johannessen and Bjerrum (1965), Endo et al (1969) and Bozozuk (1972) have suggested empirical relationships for drag-load. The scope of these formulae is obviously limited and it is unlikely that these formulae and parameters developed from experimental and field tests could be extrapolated to other conditions. The evolution of a powerful and elegant solution to the problem is thus still in its formative stage. An attempt has been made in the following sections to develop rational and simple, though approximate, solution to the problem of downdrag on piles in normally consolidated clay.

Analysis

Assumptions: The following analysis developed for the calculation of drag-load on single piles in normally consolidated clay resting on hard stratum and subjected to surcharge infinite in areal extent is based on the following assumptions:

1. The imposed surcharge has been considered to be infinite in areal extent and is replaced by an equivalent thickness of the normally consolidated deposit added to the existing deposit (Figure 1*a*). In Figure 1*a*, h is the thickness of the soil deposit and ζh is the equivalent surcharge. $\triangle_z$ is the total displacement of the soil corresponding to any depth z and $\triangle_1$ is the displacement required for the full mobilisation of skin friction. ϵh is the depth at which the displacement of the soil is equal to $\triangle_1$.

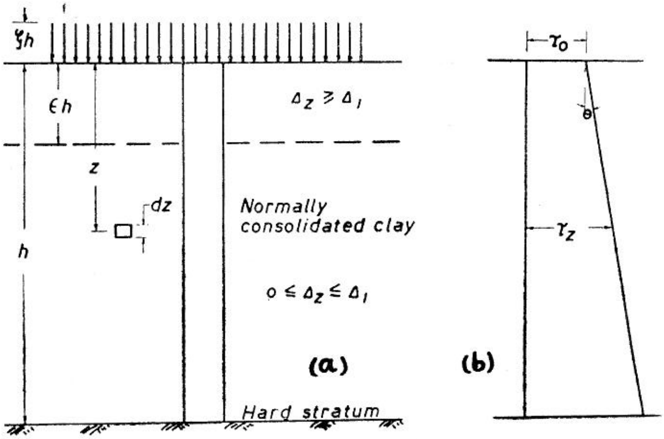

FIGURE 1*a*. Pile with infinite surcharge
1*b*. Variation of shear strength

2. The shear strength of the soil is assumed to increase linearly with depth (Figure 1*b*) with ultimate values of τ_o at the surface (to approximately account for the dessication near surface) and τ_z at a depth z below the surface. The shear resistance mobilisation characteristics at the pile-soil interface at any depth is approximated as in (Figure 2). The linearly elasto-perfectly plastic relationship explains that at any depth z the skin friction mobilised is a linear function of the relative displacement at that point up to a value of $\triangle_1$ after which the skin friction remains at $\alpha\tau_z$.

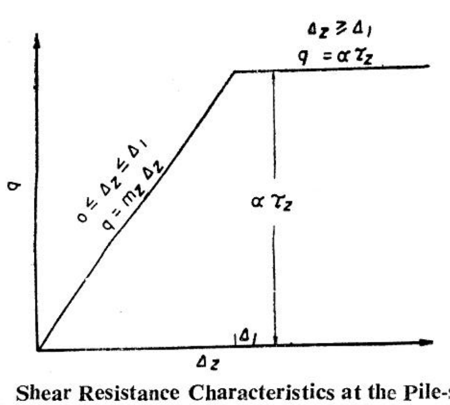

FIGURE 2. Shear Resistance Characteristics at the Pile-soil interface

The ultimate skin friction resistance increases linearly with depth but it is assumed that the displacement required to produce the full mobilisation of the ultimate skin friction $\triangle_1$, is constant. The ultimate skin friction, $\alpha \tau_z$ itself is considered to be a constant fraction of the ultimate shear strength τ_z, α being the adhesion factor.

3. It has been assumed that the settlement of the soil at a point remote from the pile and at the interface but at the same level are the same. This assumption though not strictly true has been made only for the sake of mathematical simplicity. In fact, a comparatively smaller settlement occurs near the pile than at a distance away from the pile resulting in dome shaped surfaces around the piles as observed in Mexico, (Correa, 1961).

4. The skin friction characteristics have been considered to be constant throughout the entire phase of consolidation which is also not true. There is a gain in the strength of the soil as consolidation progresses and skin friction characteristics are accordingly modified. It is considered that the phenomenon though difficult to be exactly incorporated in a simple analysis, can be approximately overcome by using strength and skin friction characteristics intermediate between their initial and final behaviours.

5. Due to the difficult nature of the quantification of the pile driving effects, they have not been considered in the analysis. Hence it might be said that the analysis is valid more for the non-displacement type of piles than for the displacement type of piles.

6. The pile is assumed to rest on a rigid hard stratum whereby no displacement of the pile tip is allowed to occur.

7. The pile is assumed to be rigid.

Assumptions 6 and 7 above are difficult to encounter in practice and their influence on drag-load will be discussed later.

Analysis for Drag-Load: The stress state of a normally consolidated deposit is defined by the effective stress at every point being equal to the overburden pressure at that point. Compression index C_c defines the consolidation characteristics of the soil. When the surcharge ζh is placed over the fully saturated normally consolidated deposit of thickness h, the excess pore water pressure induced at every point could be assumed to be the same as the surcharge intensity. It is the dissipation of this hydrostatic excess pore pressure which brings into play the consolidation of the deposit.

Considering an element of thickness dz at a depth z (Figure 1a), total compression of the element, $d\triangle_z$, due to expulsion of pore water is given by,

$$d\triangle_z = \frac{C_c}{1+e_o} \log \frac{p_z+\sigma_z}{p_z} dz \qquad \ldots(1)$$

where, e_o = average initial void ratio

p_z = overburden pressure at $z = \gamma z$

γ = bulk density of of the soil

and σ_z = increase in pressure at $z = \gamma \zeta h$

Substituting for p_z and σ_z and assuming $C_c/(1+e_o)$ to be a constant, the compression of the element is given by,

$$d\triangle_z = \frac{1}{C} \log \left(1 + \frac{\zeta h}{z}\right) dz \qquad \ldots(2)$$

where, $C = (1+e_o)/C_c$

The total settlement, $\triangle_z$, at a depth z is given by integration of Equation 2.

$$\triangle_z = \frac{1}{2.3C}\int_z^h ln \left(1 + \frac{\zeta h}{z}\right) dz \qquad \ldots(3)$$

Solving Equation 3,

$$\triangle_z = \frac{h\, I_1}{2.3C} \qquad \ldots(4)$$

where I_1 is a non-dimensional quantity and is given by,

$$I_1 = \zeta\left[\frac{1}{\zeta}\, ln\left(1+\zeta\right) + ln\left(1+\frac{1}{\zeta}\right) - \frac{z}{\zeta h}\, ln\left(1+\frac{\zeta h}{z}\right) - ln\left(1+\frac{z}{\zeta h}\right)\right]$$

The skin friction at z, q_z, corresponding to $\triangle_z$ is given by (Figure 2),

$$q_z = \alpha\, \tau_z \quad \text{if } \triangle_z \geqslant \triangle_1 \qquad \ldots(5a)$$

or

$$q_z = m_z\, \triangle_z \quad \text{if } 0 \leqslant \triangle_z \leqslant \triangle_1 \qquad \ldots(5b)$$

In Equation 5, τ_z is given by (Figure 1*b*),

$$\tau_z = \tau_o + z \tan\theta \qquad \ldots(6)$$

and from Figure 2, m_z can be expressed as

$$m_z\, \triangle_1 = \alpha\, \tau_z$$

which gives,

$$m_z = \alpha\, \frac{\tau_o + z \tan\theta}{\triangle_1} \qquad \ldots(7)$$

The drag force at depth z, per unit length of perimeter, Q_z, is obtained by integrating expressions 5*a* and5*b*.

$$Q_z = \int_o^z q_z\, dz \qquad \ldots(8)$$

Substituting for q_z from equations 5*a* and 5*b*,

$$Q_z = \tau_o\, \alpha\, z + \frac{\alpha\, z^2}{2} \tan\theta \quad \text{when } \triangle_z \geqslant \triangle_1 \qquad \ldots(9a)$$

and

$$Q_z = \tau_o\, \alpha\, \epsilon h + \frac{\alpha\, \epsilon^2 h^2}{2} \tan\theta + \int_{\epsilon h}^{z} \frac{m_z\, h}{2.3C} I_1\, dz$$

$$\text{when } 0 \leqslant \triangle_z \leqslant \triangle_1 \qquad \ldots(9b)$$

Substituting in Equation 9*b* from Equation 7 and rearranging,

$$Q_z = \tau_o \,\alpha\, \epsilon\, h + \frac{\alpha\, \tau_o\, h}{2.3C\, \triangle_1} \int_{\epsilon h}^{z} I_1\, dz + \frac{\epsilon^2 h^2}{2} \alpha \tan\theta$$

$$+ \frac{\alpha\, h \tan\theta}{2.3C\, \triangle_1} \int_{\epsilon h}^{z} z\, I_1\, dz \quad \text{when } 0 \leqslant \triangle_z \leqslant \triangle_1 \qquad \text{...(10)}$$

In Equation 10, the first two terms give the Q_z values when soil has a constant shear strength τ_o over the entire depth and the next two terms correspond to the condition when the soil strength increases linearly with the depth, from zero at surface to $z \tan\theta$ at depth z (i.e. triangular distribution). The solution of Equation 10 is given by,

$$Q_z = \tau_o\, \alpha\, \epsilon\, h + \frac{\tau_o\, \alpha\, \zeta h^2}{2.3C\, \triangle_1} I_2 + \frac{\epsilon^2 h^2 \alpha}{2} \tan\theta$$

$$+ \frac{\alpha\, \zeta\, h^3 \tan\theta}{2.3\, C\, \triangle_1} I_2' \quad \text{when } 0 \leqslant \triangle_z \leqslant \triangle_1 \qquad \text{...(11)}$$

where I_2 and I_2' are dimensionless quantities and are expressed by,

$$I_2 = I_3 + I_4 - I_5 - I_6$$

$$I_2' = I_3' + I_4' - I_5' - I_6'$$

in which,

$$I_3 = \frac{\left(\frac{z}{h} - \epsilon\right)}{\zeta} \ln(1+\zeta)$$

$$I_4 = \left(\frac{z}{h} - \epsilon\right) \ln\left(1 + \frac{1}{\zeta}\right)$$

$$I_5 = \frac{\zeta}{2}\left[\frac{z^2}{\zeta^2 h^2} \ln\left(1 + \frac{\zeta h}{z}\right) + \frac{z}{\zeta h} - \ln\left(1 + \frac{z}{\zeta h}\right)\right.$$
$$\left. - \frac{\epsilon^2}{\zeta^2} \ln\left(1 + \frac{\zeta}{\epsilon}\right) - \frac{\epsilon}{\zeta} + \ln\left(1 + \frac{\epsilon}{\zeta}\right)\right]$$

$$I_6 = \zeta \left[\frac{z}{\zeta h} \ln\left(1 + \frac{z}{\zeta h}\right) - \frac{z}{\zeta h} + \ln\left(1 + \frac{z}{\zeta h}\right)\right.$$
$$\left. - \frac{\epsilon}{\zeta} \ln\left(1 + \frac{\epsilon}{\zeta}\right) + \frac{\epsilon}{\zeta} - \ln\left(1 + \frac{\epsilon}{\zeta}\right)\right]$$

$$I_3' = \frac{\left(\frac{z^2}{h^2} - \epsilon^2\right)}{2\zeta} \ln(1+\zeta)$$

$$I_4' = \frac{1}{2}\left(\frac{z^2}{h^2} - \epsilon^2\right) \ln\left(1 + \frac{1}{\zeta}\right)$$

$$I_5' = \frac{\zeta^2}{3}\Big[\frac{z^3}{\zeta^3 h^3}\, ln\Big(1+\frac{\zeta h}{z}\Big) + \frac{z^2}{2\,\zeta^2 h^2} - \frac{z}{\zeta h} + ln\Big(1+\frac{z}{\zeta h}\Big) - \frac{\epsilon^3}{\zeta^3}\, ln\Big(1+\frac{\zeta}{\epsilon}\Big) - \frac{\epsilon^2}{2\,\zeta^2} + \frac{\epsilon}{\zeta} - ln\Big(1+\frac{\epsilon}{\zeta}\Big)\Big]$$

$$I_6' = \frac{\zeta^2}{2}\Big[\frac{z^2}{\zeta^2 h^2}\, ln\Big(1+\frac{z}{\zeta h}\Big) + \frac{z}{\zeta h} - \frac{z^2}{2\,\zeta^2 h^2} - ln\Big(1+\frac{z}{\zeta h}\Big) - \frac{\epsilon^2}{\zeta^2}\, ln\Big(1+\frac{\epsilon}{\zeta}\Big) - \frac{\epsilon}{\zeta} + \frac{\epsilon^2}{2\,\zeta^2} + ln\Big(1+\frac{\epsilon}{\zeta}\Big)\Big]$$

Now, from the definition of $\triangle_z$ by Equation 3,

$$\triangle_1 = \frac{1}{2.3C}\int_{\epsilon h}^{h} ln\Big(1+\frac{\zeta h}{z}\Big)\, dz \qquad \ldots(12)$$

Solving,

$$\triangle_1 = \frac{\zeta h}{2.3C}\, I_7 \qquad \ldots(13)$$

where I_7 is a dimensionless quantity and is given by,

$$I_7 = \frac{1}{\zeta}\, ln\,(1+\zeta) + ln\Big(1+\frac{1}{\zeta}\Big) - \frac{\epsilon}{\zeta}\, ln\Big(1+\frac{\zeta}{\epsilon}\Big) - ln\Big(1+\frac{\epsilon}{\zeta}\Big)$$

Substituting in Equation 11 from Equation 13 the expression for Q_z can be rewritten as,

$$Q_z = \alpha\,(\tau_o\, h\, K_D + h^2 \tan\theta\, K_D') \ldots \text{when } 0 \leqslant \triangle_z \leqslant \triangle_1 \qquad \ldots(14)$$

where K_D and K_D' are non-dimensional drag-load factors for a uniform shear strength and triangular distribution variation of soil strength respectively. In Equation 14, K_D and K_D' are given by,

$$K_D = \epsilon + \frac{I_2}{I_7} \qquad \ldots(15)$$

$$\text{and } K_D' = \frac{\epsilon^2}{2} + \frac{I_2'}{I_7} \qquad \ldots(16)$$

Expressions for I_2, I_7 and I_2' are the same as given in Equations 11 and 13. Equation 9*a* ($\triangle_z > \triangle_1$) can also be expressed in the same form as Equation 14. In this case $K_D = z/h$ and $K_D' = z^2/2h^2$.

In the above analysis it is assumed that for a given soil type, the magnitude of surcharge intensity is such that there occurs at a certain depth ϵh the settlement $\triangle_1$ required for the mobilisation of maximum shear resistance. But it can be recognised that for very low surcharge intensities such an amount of settlement will not occur at all even at the ground surface (ie. $\epsilon < 0$). With increasing surcharge intensities the magnitude of settlement will increase so that for a particular value of surcharge intensity the settlement at surface will become $\triangle_1$ (ie. $\epsilon \rightarrow 0$). The analysis for these two cases, namely, $\epsilon \rightarrow 0$ and $\epsilon < 0$ are developed and presented below.

(a) The Case when $\epsilon \rightarrow 0$ (i.e. $\triangle_z \rightarrow \triangle_1$ *at* $z=0$):

Proceeding in similar lines explained for the case when $\epsilon > 0$ or substituting the limit $\epsilon \rightarrow 0$ in Equation 14 it can be shown that the expression for Q_z in this case is of the same form as Equation 14, where K_D and K_D' are the appropriate non-dimensional drag-load factors and these are given by,

$$K_D = \frac{I_8}{I_9} \qquad \ldots(17)$$

$$K_D' = \frac{I_8'}{I_9} \qquad \ldots(18)$$

In expressions 17 and 18,

$$I_8 = I_{10} + I_{11} - I_{12} - I_{13}$$

$$I_8' = I_{10}' + I_{11}' - I_{12}' - I_{13}'$$

and $I_9 = (1 + \frac{1}{\zeta})\ ln\ (1+\zeta) - ln\ \zeta$

in which,

$$I_{10} = \frac{z}{\zeta h}\ ln\ (1+\zeta)$$

$$I_{11} = \frac{z}{h}\ ln\ (1 + \frac{1}{\zeta})$$

$$I_{12} = \frac{\zeta}{2}\left[\frac{z^2}{\zeta^2 h^2}\ ln\left(1 + \frac{\zeta h}{z}\right) + \frac{z}{\zeta h} - ln\left(1 + \frac{z}{\zeta h}\right)\right]$$

$$I_{13} = \zeta\ \left[\frac{z}{\zeta h} ln\left(1 + \frac{z}{\zeta h}\right) - \frac{z}{\zeta h} + ln\left(1 + \frac{z}{\zeta h}\right)\right]$$

$$I_{10}' = \frac{z^2}{2\ \zeta\ h^2}\ ln\ (1 + \zeta)$$

$$I_{11}' = \frac{z^2}{2h^2}\ ln\left(1 + \frac{1}{\zeta}\right)$$

$$I_{12}' = \frac{\zeta^2}{3}\left[\frac{z^3}{\zeta^3 h^3}\ ln\left(1 + \frac{\zeta h}{z}\right) + \frac{z^2}{2\ \zeta^2 h^2} - \frac{z}{\zeta h} + ln\left(1 + \frac{z}{\zeta h}\right)\right]$$

and $I_{13}' = \frac{\zeta^2}{2}\left[\frac{z^2}{\zeta^2 h^2}\ ln\left(1 + \frac{z}{\zeta h}\right) + \frac{z}{\zeta h} - \frac{z^2}{2\ \zeta^2 h^2} - ln\left(1 + \frac{z}{\zeta h}\right)\right]$

(b) The Case when $\epsilon < 0$ (i.e. $\triangle_z < \triangle_1$ *at* $z=0$):

The expression for Q_z in this case is also expressed again in the same form of Equation 14 and the appropriate expression for K_D and K_D' for this case are:

$$K_D = \frac{\zeta h}{2.3\ C\ \triangle_1}\ I_8 \qquad \ldots(19)$$

and $K_D' = \frac{\zeta h}{2.3\ C\ \triangle_1}\ I_8' \qquad \ldots(20)$

Expressions for I_8 and I_8' are the same as in Equations 17 and 18.

Computation of ϵ:

In order to calculate the drag-load factors K_D and K_D' according to the appropriate equations it is first required to know the value of ϵ. ϵ can be determined in the following manner. Equation 13 can be expressed as

$$\left(1+\frac{\epsilon}{\zeta}\right) \ln(\epsilon+\zeta)-\frac{\epsilon}{\zeta} \ln \epsilon+\frac{2.3\, C\, \triangle_1}{\zeta h} - \left(1+\frac{1}{\zeta}\right) \ln(1+\zeta)=0 \qquad \ldots(21)$$

In Equation 21 all the values except ϵ are known. The solution of this equation gives the value of ϵ(for $\epsilon > 0$).

Results and Discussions

The Case when $\epsilon>0$: The values of K_D and K_D' according to Equations 15 and 16 for z/h values ranging from ϵ to 1 have been calculated for ϵ values ranging from 0.1 to 0.9 and ζ values ranging from 0.001 to 1. The value of K_D for $z/h \leqslant \epsilon$ is given by the numerical value of z/h at that level and that of K_D' by $z^2/2h^2$. Figure 3 shows the variation of K_D with ζ for $z/h=1$, each

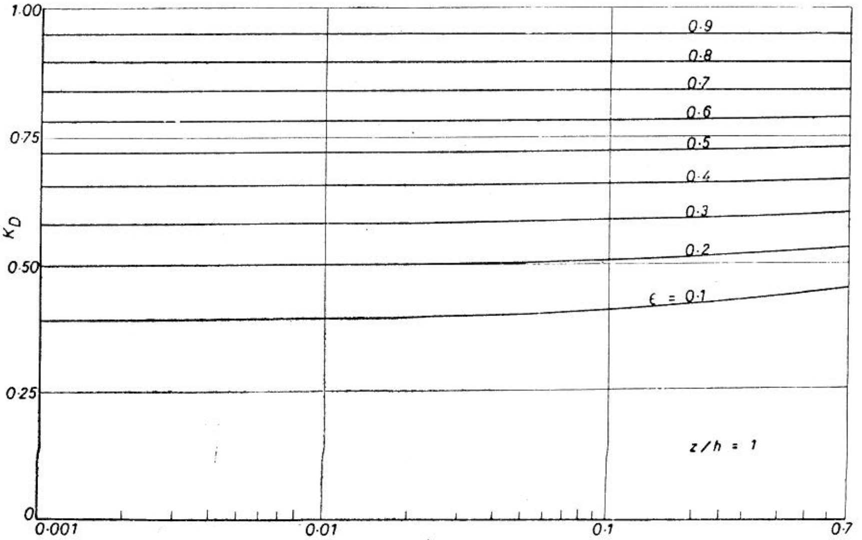

FIGURE 3. Variation of drag load factor, K_D with ζ

curve in it being for one value of ϵ and Figure 4 gives a similar plot for K_D'. From these two figures it could be seen that both K_D and K_D' are influenced chiefly by ϵ and the direct influence of ζ on K_D and K_D' is rather insignificant. The same trend was observed for other z/h values also. However, it should be stated that ζ is one of the major factors that govern the value of ϵ as would be seen later. Since ζ is not found to significantly influence K_D and K_D' values, average values of K_D and K_D' (averaged for $\zeta=0.001$ and $\zeta=1$) have been used to represent their variation with z/h

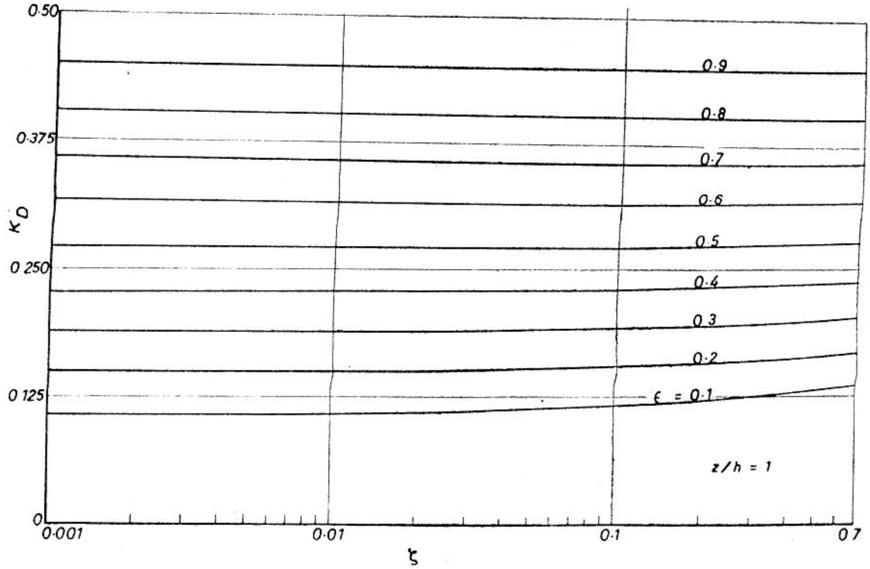

FIGURE 4. Variation of drag load factor, K_D' with ζ

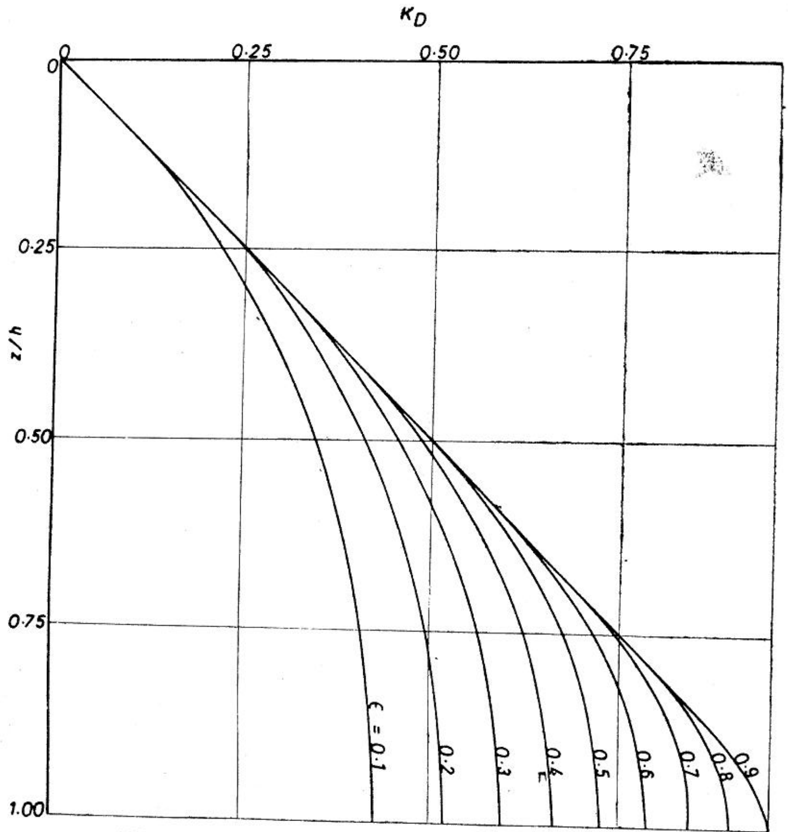

FIGURE 5. Variation of drag load factor, K_D with z/h

respectively in Figures 5 and 6. Each curve in these figures is for one value of ϵ. Further, these two figures present a visual picture of the distribution of drag-load along the length of the pile when the soil shear strength is uniform over depth and when it has triangular distribution, respectively. If the strength of the soil could be idealised as uniform with depth, then the drag-load per unit length of perimeter at any depth is merely the K_D value from Figure 5 corresponding to that value of z/h multiplied by $\alpha \tau_o h$. For cases where the soil strength increases linearly with the depth the contribution to Q_z only due to the increase of strength is obtained by multiplying the value of K_D' at that z/h from Figure 6 by $\alpha h^2 \tan \theta$.

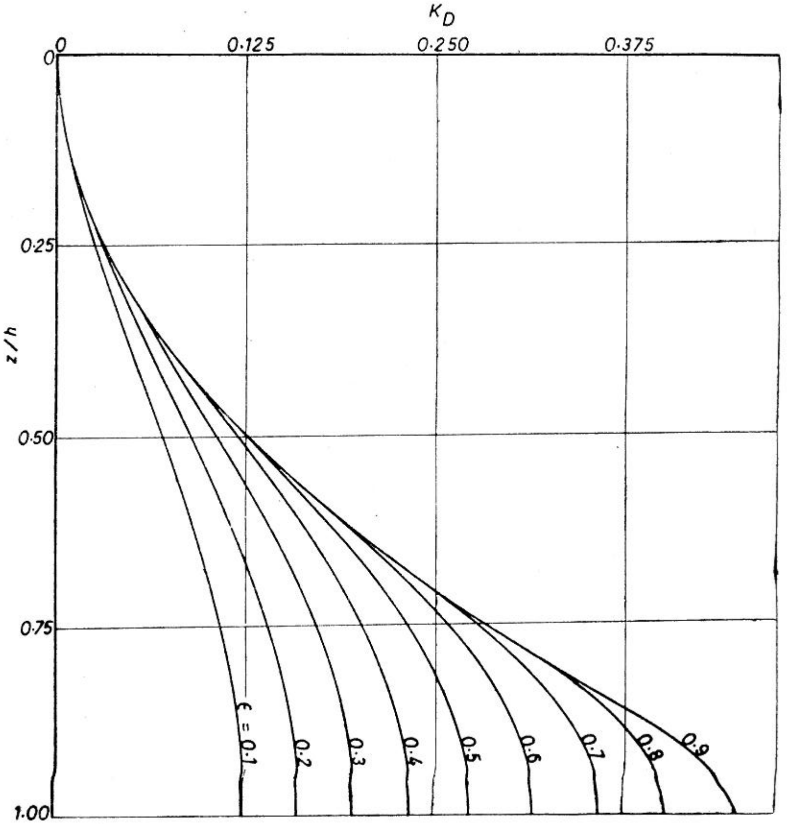

FIGURE 6. **Variation of drag load factor, K_D' with z/h**

The Case when $\epsilon \to 0$: The values of K_D and K_D' according to Equations 17 and 18 for z/h values ranging from 0.1 to 1 have been calculated for ζ values ranging from 0.001 to 1. Figure 7 shows the variation of K_D with ζ, each curve in it being for one value of z/h and Figure 8 gives a similar plot for K_D'. Unlike the previous case ($\epsilon > 0$, Figures 3 and 4) it is seen here that K_D and K_D' are influenced by ζ and hence the averaging of K_D and K_D' over the computed range of ζ has not been attempted in this case. Figures 9*a* and 9*b* report respectively the variation of K_D and K_D' with z/h, each curve in them being for one chosen value of ζ. These figures give a visual representation of the distribution of drag-load along the length of the pile for the cases of uniform strength and that of only strength increase

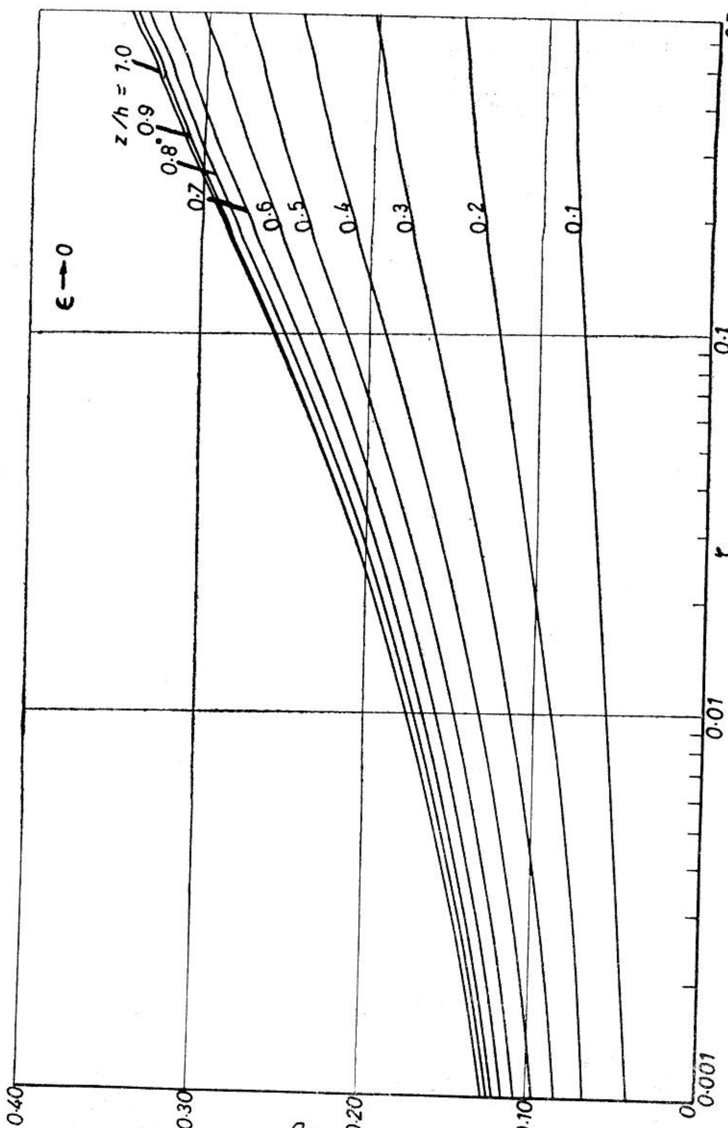

FIGURE 7. Variation of drag load factor K_D with ζ

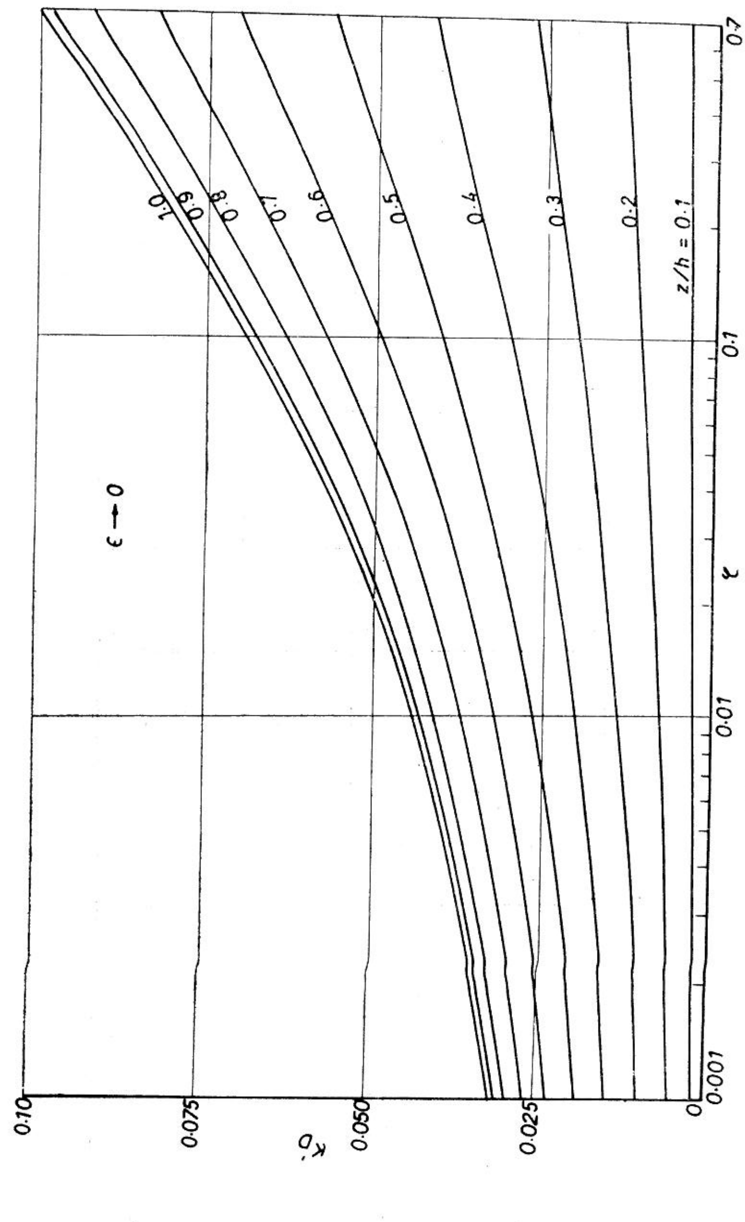

FIGURE 8. Variation of drag load factor, K_D' with ζ

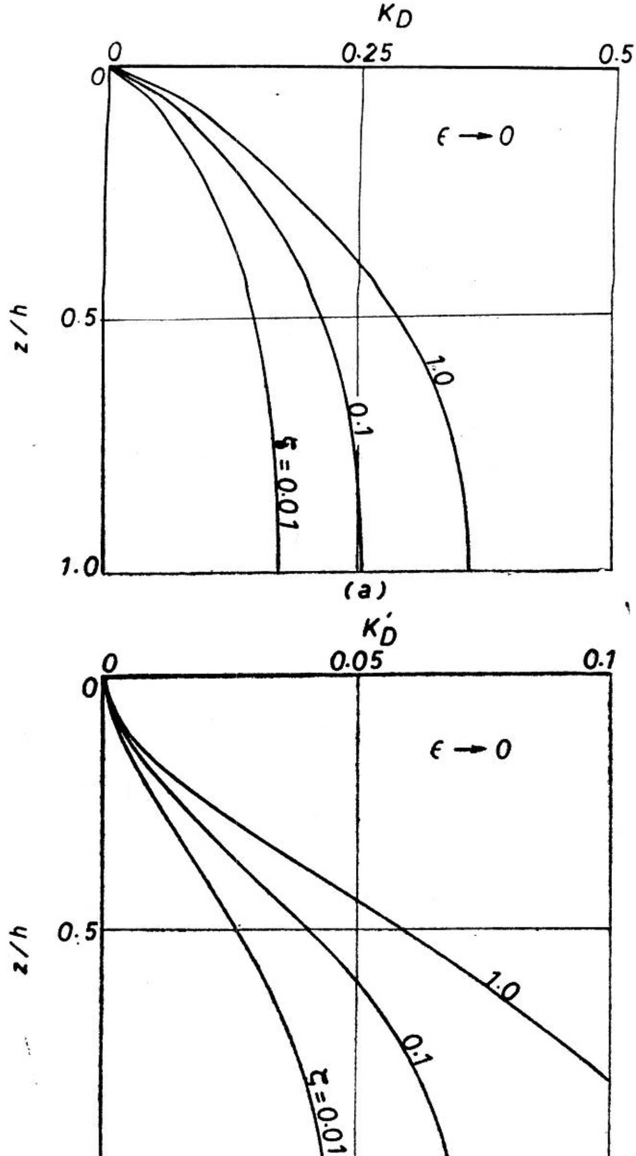

FIGURE 9. Variation of drag load factors, K_D' and K_D with z/h

(linear) with depth. It can be reasoned out that for a stiffer deposit, the surcharge intensity required for ϵ to approach zero will be higher than that for a softer deposit (h, $\triangle_1$ being the same for the two). The results reported in Figure 9 viewed together with the above fact would indicate

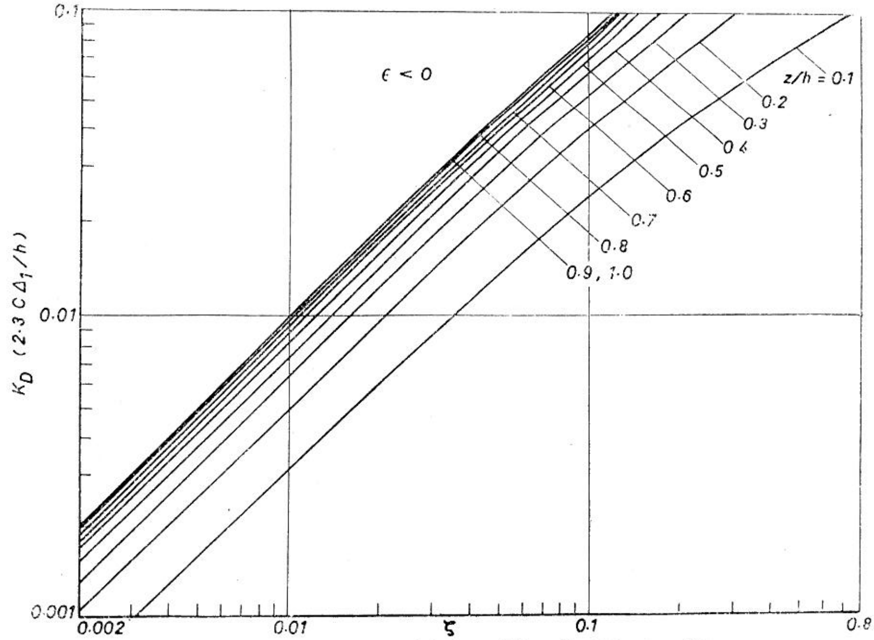

FIGURE 10. Determination of drag load factor, K_D

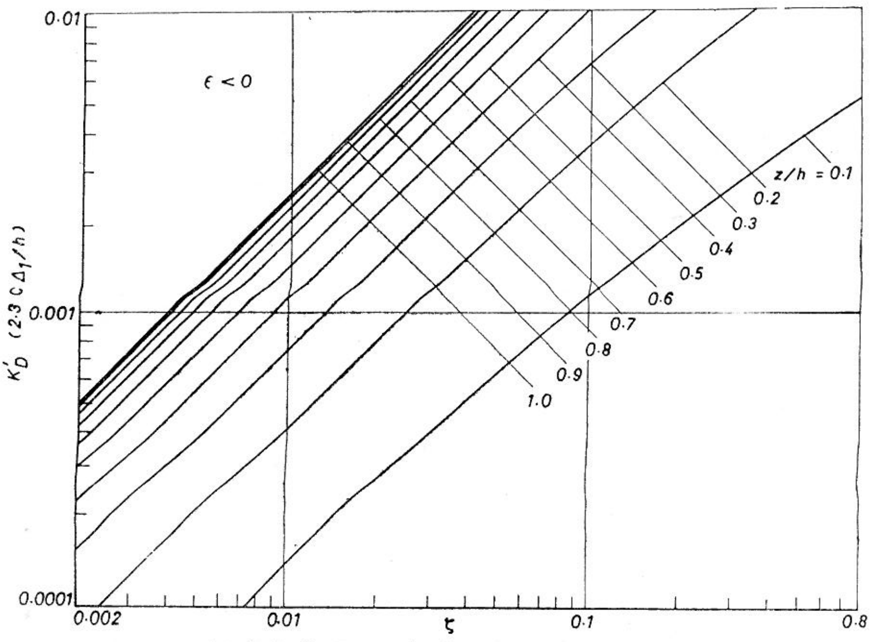

FIGURE 11. Determination of drag load factor, K_D'

that the drag-load would increase for stiffer deposits. To calculate the drag-load when $\epsilon \rightarrow 0$, from the known values of ζ and z/h, the appropriate value of K_D and K_D' can be selected from Figures 7 and 8. Substituting these values in Equation 14, Q_z can be determined.

The Case when $\epsilon < 0$: The expression for K_D and K_D' for this case is given by Equations 19 and 20 respectively. It is seen in these expressions that $(2.3C\triangle_1/h)\,K_D$ (ie. ζI_8) and $(2.3C\triangle_1/h)\,K_D'$ (ie. $\zeta I_8'$) turn out to be functions of z/h and ζ. Hence calculations for ζI_8 and $\zeta I_8'$ have been made for z/h values ranging from 0.1 to 1 and ζ ranging from 0.001 to 1. The results are reported in Figures 10 and 11. Figure 10 shows the variation of ζI_8 with ζ (in log-log scale) with each curve in it being for one value of z/h and Figure 11 gives a similar plot for $\zeta I_8'$. For known values of ζ and z/h in this case the K_D and K_D' values are obtained by first determining the respective values of ζI_8 and $\zeta I_8'$ from Figures 10 and 11 and then dividing these values by $(2.3C\triangle_1/h)$. Using Equation 14, Q_z can then be determined. Visual representation of drag-load distribution for the case of $\epsilon < 0$, for the two cases of uniform strength and only strength increase (linear) with depth is obtained by the data reported in Figures 12*a* and 12*b* respectively.

Computation of ϵ : Knowing the value of $C, \triangle_1$, h and ζ, ϵ can be determined by solving Equation 21 which has been done using Newton's Second Order Tangent Method for $C\triangle_1/h$ values ranging from 0.0001 to 0.1 and ζ values ranging from 0.001 to 1. Computations have been carried out to an absolute remainder accuracy of 0.001 or less. Figure 13 presents the results in which is shown the variation of ϵ with S_1 $(=C\triangle_1/h)$, each curve in it being for one value of ζ. Using this figure the ϵ value can be read out.

Yet another method of finding ϵ is from the known soil settlement at any depth. The depth at which the soil settlement $\triangle_z$ is equal to $\triangle_1$ gives the ϵh value from which ϵ can be calculated. The soil settlement at any depth is given by Equation 4. The expression for $\triangle_z$ at the surface (ie. $z/h \rightarrow 0$) is again given by an expression of the same form as Equation 4 but with the expression for I_1 being modified as,

$$I_{1,\,(z/h\rightarrow 0)} = (1+\zeta)\,ln(1+\zeta) - \zeta ln\zeta \qquad \ldots (22)$$

Computations for I_1 in Equation 4 have been made for z/h values ranging from 0 to 1 and ζ values ranging from 0.001 lo 1. The results are reported in Figure 14, showing the variation of I_1 with z/h with each curve in it being for one value of ζ. It is interesting to observe that the results of Figure 14 give to some scale the settlement profile of the soil along the depth. To compute the settlement value at any depth the appropriate value of I_1 is multiplied by $h/2.3C$.

Illustrative Example: For the example problem the following data have been assumed.

$h = 12.5$ m ; $D = 30$ cm ; $\tau_o = 0.7$ kg/sq. cm ;

$C = 10$; $\triangle_1/h = 0.001$; $\tan\theta = 0.001$ kg/sq. cm/cm

Let us first assume that $\zeta = 0.001$. Referring to Figure 13, it is seen that ϵ tends to very small values and from the shape of the curve $(\zeta = .001)$

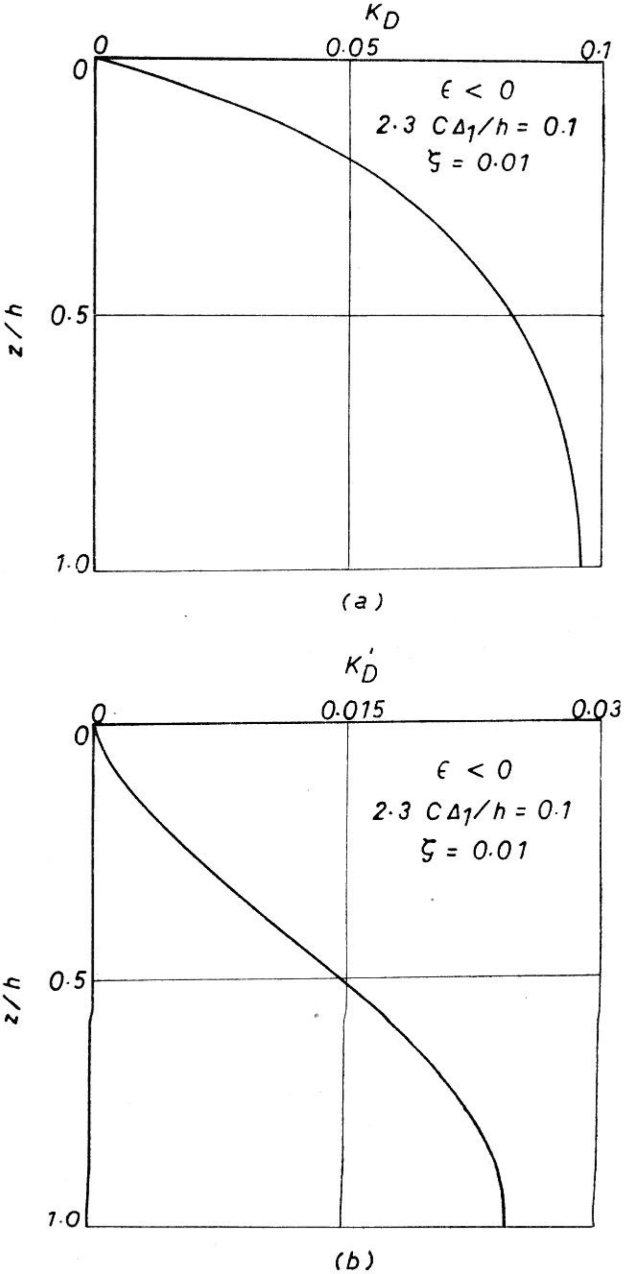

FIGURE 12. Variation of drag load factor, K_D and K_D' with z/h

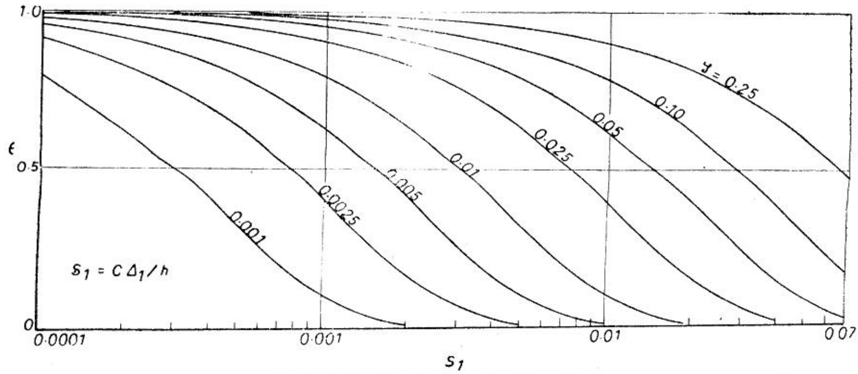

FIGURE 13. Determination of ϵ

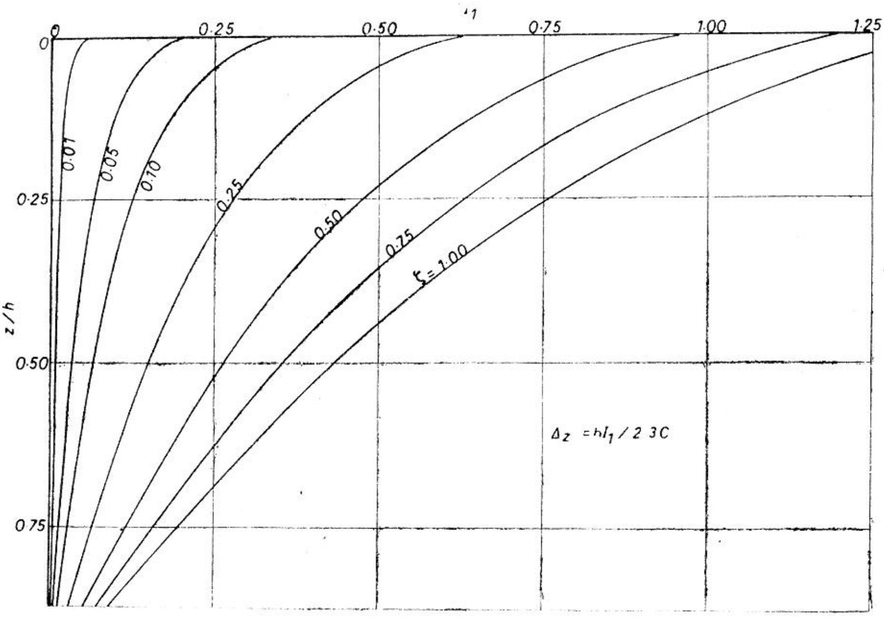

FIGURE 14. Variation of I_1 with z/h

ϵ is likely to be negative. Using Equations 4 and 22 the settlement of clay layer at the surface is found to be 0.43 cm which is less than $\triangle_1$ (ie. 1.25 cm). Hence ϵ is less than zero. So K_D and K_D' values for this case are obtained from Figures 10 and 11 respectively. The K_D and K_D' are found to be 0.045 and 0.011 respectively for $z/h=1$. Substituting these in Equation 14 and assuming α to be 0.5, the total drag-load, P_n, at the toe of the pile ($z/h=1$) is given by,

$$P_n = Q_z \times \text{perimeter}$$

$$= 0.5\,(0.7 \times 1250 \times 0.045 + 1250^2 \times 0.1 \times 0.011) \times 30\,\pi$$

$$= 2.6 \text{ T}$$

Now, let the value of ζ be increased from 0.001. It can be seen from Figure 13 that at some intermediate value of ζ between 0.0025 and 0.005,

ϵ value tends to become zero. In equation 4 keeping $\triangle_z = \triangle_1$ at $z/h=0$, ζ is evaluated. ζ is found to be nearly 0.004 For this value of ζ Figure 7 and 8 are consulted for K_D and K_D' value. The K_D and K_D' values are 0.15 and 0.038 respectively for $z/h=1$. Substituting in Equation 14 the drag-load at toe is found to be 9.0 T. If ζ is increased beyond 0.004, ϵ becomes greater than zero and Figure 5 and 6 should be referred to for getting K_D and K_D' values. Let the value of ζ be increased to 0.025. Then ϵ value is found to be 0.39 (Figure 13). The corresponding values of K_D and K_D' are 0.664 and 0.235 respectively for $z/h=1$. Substituting in Equation 14 the drag-load at toe for this case is found to be 44.4 T. From similar calculations for drag-load for different values of z/h the drag-load distribution along the length of the pile can be determined. Figure 15 shows the drag-load distribution for the three cases of ζ discussed

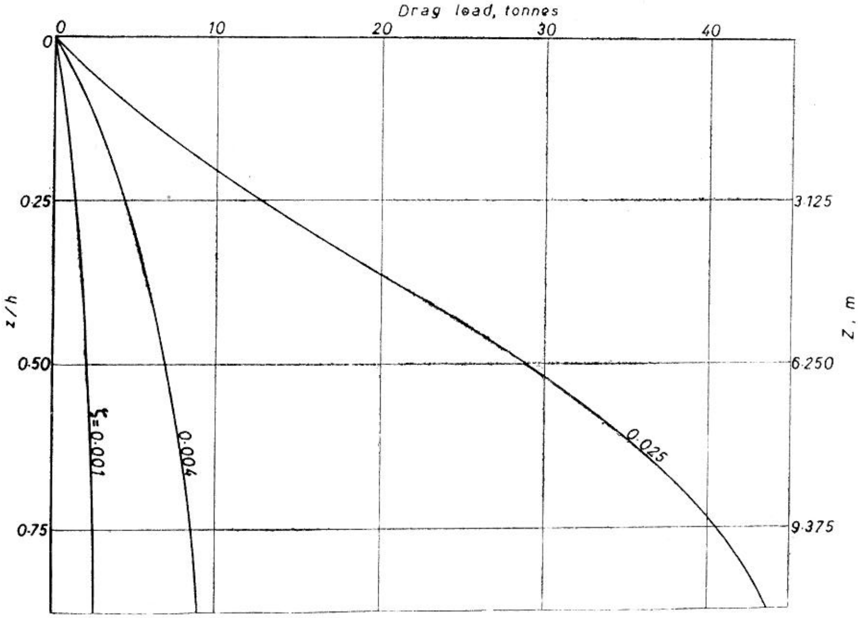

FIGURE 15. Results of example problem

above, namely, *0.001*, *0.004* and *0.025*. Figures *16a* and *16b* show the variation of K_D and K_D' with ζ for $z/h=1$, for the example problem. The curve in broken line shows the variation when reference for drag-load factors is made to Figures 10 and 11, ie. $\epsilon<0$, without considering actual ϵ values. The curve in full line shows the variation considering the actual ϵ values. It can be seen that if the solution for ϵ less than zero (Figures 10 and 11) are continued to be used, even after the value of ζ at which ϵ tends to become zero, it will result in the prediction of drag-loads higher than the actual ones.

The analysis assumes the piles to be resting on a rigid hard stratum, ie. the pile tips do not settle. But it is difficult to encounter such a condition in practice. The pile tip invariably settles and a neutral point results. The neutral point is where there is no relative movement between the pile and the soil. Below the neutral point the load is dissipated through positive

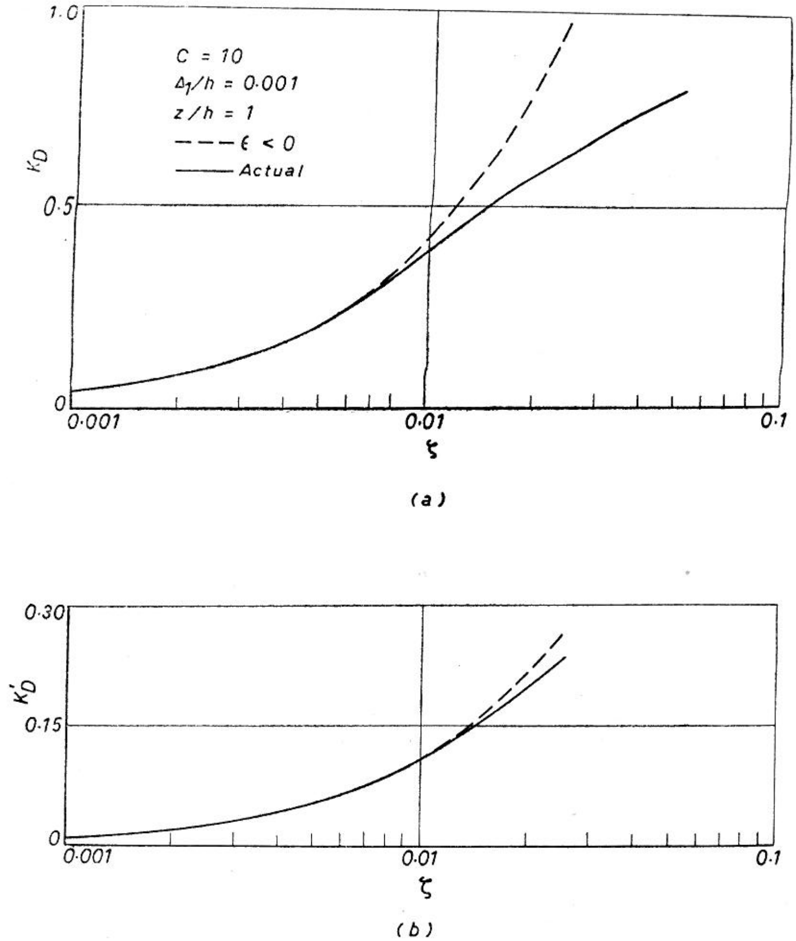

FIGURE 16. Variation of drag load factors, K_D and K_D' with ζ for example problem

skin friction. Also the settlement of the pile tip decreases the relative movement between the pile and the soil at any point. This tends to reduce the value of ϵ which in turn leads to lower drag-load factors. Hence the assumption of a rigid stratum would result in the conservative prediction of drag-load (i.e. higher than what actually would occur if this condition is not satisfied). Similar would be the effect of the assumption of rigid behaviour for the pile and equal settlement of soil near and away from the pile. The actual elastic compression of the pile tends to decrease the relative settlement between the pile and the soil which as stated earlier decreases the value of ϵ with a consequent reduction in the values of K_D and K_D'. The assumption of equal settlement of soil near and away from the pile is also not true as a comparatively smaller settlement occurs near the pile than at a distance away from the pile (Correa, 1961; Locher, 1965). This again gives rise to reduced relative movement between the pile and the soil. Hence the effect of these three assumptions which are difficult to realise in practice would be to err on the conservative side in the prediction of drag-loads. Results of investigations on piles resting on compressible stratum are being presented in a subsequent paper.

The assumption that constant relative movement between pile and soil is required for the full mobilisation of shear resistance at the pile-soil interface (ie. $\triangle_1$ is assumed invariant with depth) will also contribute to the over-estimation of drag-load. The relative movement required will possibly increase with increasing confining pressure. Thus, if $\triangle_1$ were to actually increase with depth, the slope of the linear portion (up to $\triangle_1$) of the assumed shear resistance characteristics curve, at the pile-soil interface is less than that of the curve drawn assuming $\triangle_1$ to be a constant. Hence, for any level of relative movement within this range, the intensity of drag force mobilised will be less. In this context it may be stated that there is paucity of data regarding the relative movement $\triangle_1$ at the pile-soil interface as affected by various factors like, soil type, surface condition of the pile, stress level, rate of strain, etc.

Conclusions

An analytical procedure for calculation of drag-load on a pile in normally consolidated clay, resting on a rigid stratum and subjected to surcharge infinite in areal extent has been developed. The analysis includes consideration for the various important factors influencing the phenomenon such as stress state and consolidation characteristics of the soil, skin friction characteristics and the nature of the surcharge imposed on the soil. The shear strength of the normally consolidated clay deposit is considered to increase linearly with the depth. Expressions for drag-load have been arrived at for three cases, namely, $\epsilon > 0$, $\epsilon \to 0$ and $\epsilon < 0$ (ϵ being the parameter defining the depth at which the relative movement between pile and soil is that required to mobilise full shear resistance) in terms of non-dimensional drag-load factors K_D and K_D' respectively for uniform strength and triangular strength variation with depth. The results are presented in the form of non-dimensional design charts. The use of the procedure and the charts is illustrated with an example problem and it is therein discussed how the use of charts for $\epsilon < 0$, when actually ϵ is > 0, will result in the calculation of higher than actual drag-loads. Calculation of ϵ from known values of C, $\triangle_1$, h and ζ is also explained. The assumptions of existence of a rigid bearing stratum, equal settlement of soil near and away from the pile, rigid behaviour for the pile and $\triangle_1$ to be *invariant with depth,* have been reasoned out to result in the calculation of conservative (higher) values for drag-load.

Notation

C	=	$C_c/(1+e_o)$
C_c	=	Compression index
e_o	=	average initial void ratio
h	=	thickness of the normally consolidated deposit
K	=	dimensionless drag-load factor for soil strength uniform over depth
K_D'	=	dimensionless drag-load factor for triangular distribution of soil strength

ln = natural logarithm

log = common logarithm

m_z = slope of the linear portion (up to $\triangle_1$) of the shear resistance characteristics curve, at the pile-soil interface

p_z = overburden pressure at z

P_n = total drag-load on a pile

q_z = skin friction at depth z

Q_z = drag force per unit length of perimeter at z

z = cartesian co-ordinate (Figure 1*a*)

α = adhesion factor

γ = bulk density of soil

$\triangle_z$ = settlement of soil at depth z = relative movement between pile and soil at z

$\triangle_1$ = relative movement between pile and soil required to mobilise full shear resistance

ϵ = parameter defining the depth at which the relative movement between pile and soil is $\triangle_1$

ζ = parameter defining the equivalence of surcharge intensity

θ = parameter defining the increase of soil shear strength with depth

σ_z = vertical stress increase at z

τ_o = shear strength of soil at surface

τ_z = shear strength of soil at depth z

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