The Relief of Negative Skin Friction on Piles by Electro-Osmosis

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SUMMARY The paper describes a method of analysing the magnitude and rate of reduction of negative skin friction on piles. The method combines a pile-soil interaction analysis (based on elastic theory) with a diffusion analysis for pore water flow under an electrical gradient. The method has been applied to predict the behaviour of a pile subjected to electro-osmosis in two cases.

(i) a full-scale field trial carried out in Boston some years ago in which one pile in a group supporting a bridge abutment was cut, released from the pile cap and then subjected to electro-osmosis. "Class A" predictions were made of the pile deflection at various stages in the trial, and comparisons are described between the predictions and the measurements.

(ii) a laboratory model test, in which negative friction has been induced in a pile by consolidation of the surrounding soil and then brief periods of electro-osmotic treatment have been applied to the pile. Predictions of the downdrag load versus time relationship are compared with the observed relationships.

1 INTRODUCTION

Consolidation and settlement of soil surrounding a pile may cause large downdrag forces to be developed in the pile due to negative friction (Johanessen and Bjerrum, 1965, Bjerrum et al, 1969; Walker and Darvall, 1973). One of a number of methods that have been suggested to relieve or reduce these downdrag forces is the application of electro-osmosis to the pile. In this method, an electric current is passed through the soil between the pile (which is made the cathode) and an anode. The passage of the current causes positive excess pore pressures to be generated near the cathode and negative values near the anode. Since the total stresses are unaffected, the increase in pore pressure near the treated pile causes a decrease in the effective stresses and consequently, a reduction in the pile-soil shear strength and the downdrag forces in the pile. As well as reducing negative friction, electro-osmosis can be used to reduce the penetration resistance of a pile during installation (Johnston, 1978) or, b making the pile the anode, to increase its ultimate load capacity (Soderman and Milligan, 1961).

Although some applications of electro-osmosis to piles have been reported in the literature, there appears to be little attempt to predict theoretically the effect of this treatment on the downdrag force. This paper therefore summarizes an approach for

(a) the prediction of the magnitude and rate of development of downdrag force in a pile, and

(b) the effect of electro-osmotic treatment on the downdrag force.

Two applications of this approach are then described, the first for a field trial in Boston in which " "Class A" predictions of the pile behaviour before and after electro-osmotic treatment, and the second a laboratory test on a model pile subjected to a sequence of consolidation and electro-osmotic treatment.

2 THEORETICAL ANALYSIS

2.1 Downdrag Force in Pile

The analysis is a simplified form of the boundary element method and is based on the use of elastic theory. The problem of an end-bearing pile is illustrated in Fig. 1. As previously described by the authors (1972, 1975), the pile is divided into a number of cylindrical elements, each acted upon a uniformly-distributed vertical interaction stress. Vertical movement of the soil at each element arises from two sources:

(i) settlement of the soil (e.g. due to consolidation), the distribution of which has to be specified in the analysis - these settlements will be referred to as "free-field" settlements.

(ii) the effect of the pile-soil interaction stresses; an expression for this component can be derived by use of Mindlin's equations for subsurface loading in an elastic mass.

By assuming the pile to deform as an elastic column, the movement of each element of the pile can be expressed in terms of the interaction stresses, the elastic properties of the pile and the applied load

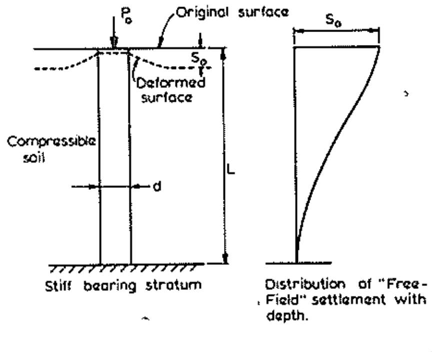


Figure 1 End-bearing pile subjected to negative friction

at the pile head. While conditions at the pile-soil interface remain elastic, the soil and pile displacements are equal, and hence by equating the expressions derived for these displacements, a solution may be obtained for the interaction stresses; the distribution of displacements and downdrag force in the pile may then be obtained.

However, because only small relative movements between the pile and soil are needed to cause pile-soil slip, it is important to incorporate the effects of such slip into the analysis. This may readily be done by checking the elastic interaction stresses against the available pile-soil shearing resistance; at elements where the available resistance, slip will occur and the displacement compatibility equation is replaced by an equation setting the interaction stress equal to the available resistance. The solution is then recycled until the interaction stresses on all elements do not exceed the limiting values.

The limiting pile-soil shear resistance τ_a' is generally calculated from the Coulomb expression, in terms of effective stresses, as

$$\tau_a' = c_a' + K_s \tan\phi_a' . \sigma_v' \qquad (1)$$

where c_a' = pile-soil adhesion

ϕ_a' = effective pile-soil friction angle

K_s = lateral pressure coefficient

σ_v' = vertical effective stress

Poulos and Davis (1975) present a series of solutions which indicate conditions under which full pile-soil slip along the pile shaft is likely to occur, and the range of pile-soil parameters for which a transition from full slip to purely elastic conditions occurs.

2.2 Time Effects

If the soil movements arise from consolidation (e.g. due to surcharging or dewatering), the development of downdrag force and displacement in the pile may be analysed by performing the above analysis for a number of times. At each time, the distribution of excess pore pressure and free-field settlement with depth can be determined from consolidation theory. The soil settlement distribution is input into the analysis while the excess pore pressures are used to calculate σ_v', and hence τ_a', in Eq. 1.

A series of solutions for the rate of development of downdrag force and displacement in an end-bearing pile are presented by Poulos and Davis (1975).

2.3 Effects of Electro-Osmosis

Consideration of electro-osmotic flow through a soil leads to the following equation for a homogeneous isotropic soil (Esrig, 1971):

$$\nabla\xi^2 = \frac{1}{c_v}\frac{\partial \xi}{\partial t} \qquad (2)$$

where ξ = u+MV

u = excess pore pressure

$M = \frac{k_e}{k} . \gamma_w$

k_e = electro-osmotic permeability

k = hydraulic permeability

γ_w = unit weight of water

V = applied potential difference between electrodes

c_v = coefficient of consolidation

Fig. 2 is of identical form to the consolidation equation from diffusion theory and can be solved analytically for simple boundary conditions, or by numerical methods (such as finite differences) for more complicated cases. The variation with time of excess pore pressure u within the soil, and in particular, at the cathode, can thus be determined, and the change $\Delta\tau_a'$ in the pile-soil shear resistance τ_a' can be calculated as:

$$\Delta\tau_a' = u . K_s \tan\phi_a' \qquad (3)$$

Consequently, it is then possible to calculate the shape in downdrag load in the pile with time (or correspondingly, increase in load capacity at the anode).

If a full analysis of pile-soil interaction is to be carried out, the effect of the excess pore pressure u on the free-field settlement near the pile can be calculated from conventional settlement theory; the use of an unloading Young's modulus for the soil would be appropriate in this case.

3 PREDICTIONS FOR CUTLER CIRCLE BRIDGE PILE

3.1 Prediction Symposium

As part of a research program at MIT (Boston, U.S.A.) into negative friction, a Symposium was held in 1973 in which a number of geotechnical engineers, including the authors, were invited to predict the downdrag force on a pile at the Cutler Circle Bridge near Boston, and the subsequent effects of electro-osmotic treatment of this pile. These predictions were then compared with the results of a field testing program. The Cutler Circle Bridge was built in 1956 as part of the Interstate Highway System but was not put into use as the section of highway adjacent to the bridge was not completed. There was evidence that the piles supporting the bridge abutment had been subjected to downdrag; this evidence included a 450 mm differential settlement of the approach slab, settlement and cracking of the slope protection beneath the bridge, and a backward tilting of the abutment.

A full description of the Symposium is given by Garlanger and Lambe (1973). Fig. 2 illustrates the soil profile and the abutment pile group. Because the research pile was not instrumented before driving, the actual load in the pile was not known, and had to be deduced indirectly. The field testing program consisted of the following five steps:

(a) a 1.5 m by 2.1 m shaft was excavated and braced behind the abutment, thus exposing the centre - pile (the research pile).

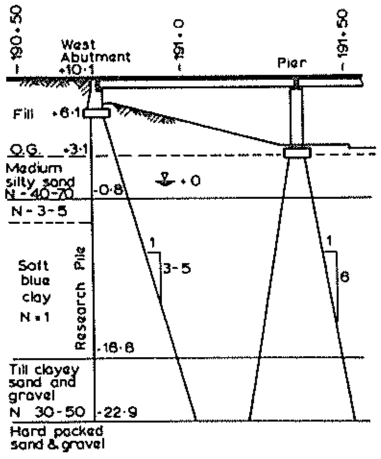
(b) a small section was cut out of the pile, and measurements were made of the strain and movement at a point (A) near the top of the pile.

(c) the fill and sand immediately surrounding the pile were excavated to the elevation of the clay layer, and the movement of point A measured.

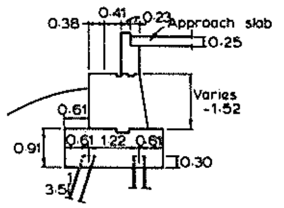
(d) the pile was subjected to electro-osmosis by passing a direct current between the test pile (cathode) and the neighbouring pile (anode), and the movement of point A was measured.

(e) with the electro-osmosis still applied, the pile was jacked back to its original position, and the required load at the top of the pile was measured.

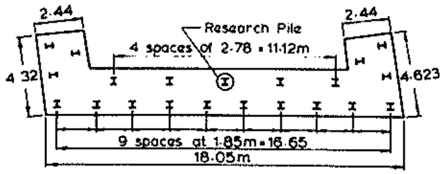
Working independently, each participant made a prediction of the behaviour of the test pile,



Note All piles 14 BP 73 steel H-piles minimum bearing capacity 6 MN



(b) ABUTMENT CROSS-SECTION.



(c) PILE LAYOUT FOR WEST ABUTMENT.

Figure 2 Details of Cutler Circle bridge abutment

including the pile movements in Steps (b) to (d) above, and the load in Step (e). The results of a field and laboratory sampling and testing program were made available to the predictions, and this data is summarized in Fig. 3.

3.2 Prediction Procedure

The first step in the Authors' predictions was to estimate the existing load at the pile head due to dead loading. In the absence of other data, this load was taken as two-thirds of the design load of 570 kN. The axial load distribution due to dead loading and negative friction was then calculated from the analysis described previously. In this analysis, the soil modulus was taken as 8.3 MPa and that of the underlying sand and gravel layer was assumed to be 345 MPa. The factor $K_s \tan\phi_a'$ was taken as 0.30 for the fill, the sand and the clay and the distribution of effective vertical stress σ_v' was calculated as the sum of the overburden pressure and the vertical stress due to the embankment (the latter being calculated from elastic theory). One - dimensional consolidation theory was used to estimate the free - field soil settlements in the 17 years since placement of the fill. The analysis showed that the relative movements between the pile and soil were sufficiently large to cause full slip along almost the entire length of the pile shaft.

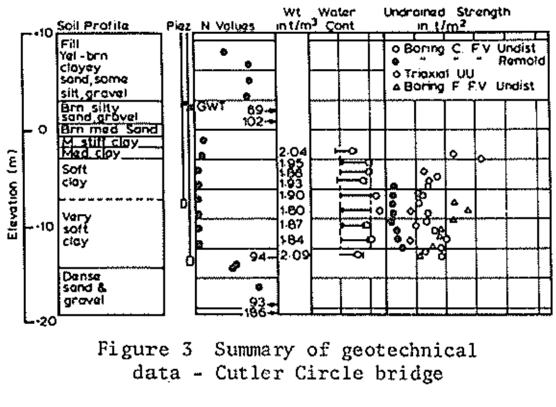


Figure 3 Summary of geotechnical data - Cutler Circle bridge

Having thus established the conditions along the pile prior to the field test program, the predictions for Steps (b) to (e) were made as follows:

Step (b): the cutting of the pile was simulated by removing the estimated axial dead loading of 380 kN. The strain and movement at point A of the pile were thus calculated.

Step (c): the effect of excavation of the fill and sand was considered by reducing the value of τ_a' for these layers to zero, and calculating the corresponding movement of point A.

Step (d): the effects of electro-osmosis were predicted by carrying out a finite difference solution of Eq. 2 using a two-dimensional analysis in the horizontal plane. This prediction presented some difficulty as the voltage and the treatment period was not known; a voltage of 32 V and a continuous treatment period of 48 hours were arbitrarily chosen. Other parameters used in the analysis were c_v = 32 mm²/sec, k = 1.5 x 10^{-6}mm/sec and k_e = 0.2 mm²/sec volt. From the excess pore pressures thus calculated, the change in τ_a' (Eq. 3), and hence the change in downdrag load, with time, was calculated. The consequent movement of the pile with time was calculated by integrating the strains in the pile developed as a result of the downdrag load changes.

Step (e): the load required to jack the pile back to its original position was calculated by finding the load necessary to cause a pile head movement equal to the sum of the movements in Steps (b) to (d). Full slip was assumed to exist along the length of the pile.

For the above predictions, no account was taken of the possible effects of horizontal movements, or of interaction between the research pile and the adjacent piles.

The Authors' predictions are summarized in Table I.

3.3 Other Predictions

All six predictors used much the same basic aproach. The load distribution in the pile was calculated for assumed conditions prior to cutting, after cutting, after excavation of the fill and sand, and after electro-osmosis. Differences between predictions arose in their estimates of the initial structural load, the factor $K_s \tan\phi_a'$, the vertical effective stress distribution, and the behaviour of the bearing layer. None of the other five predictors considered the time effects associated with electro-osmosis, all assuming that the treatment would be totally effective in reducing downdrag. Also, all the other predictors assumed full pile-soil slip along the shaft.

The range of predictions made by the other participants is shown in Table I. A more detailed description of their predictive procedures is given by Garlanger and Lambe (1973).

TABLE I

SUMMARY OF PREDICTIONS AND MEASUREMENTS FOR CUTLER CIRCLE BRIDGE TEST PILE

Quantity	Authors' Predictions	Range of Participants' Predictions	Measured Value
Strain at A after cutting of pile $(\mathrm{x}10^{-6})$	150	100 - 190	365
Movement of A after cutting of pile mm	3.0	1.5 - 19.2	10.2
Movement of A after excavating sand and fill mm	1.7	1.2 - 6.1	2.5
Movement of A after electro-osmosis mm	3.6	2.0 - 4.6	4.0
Load to return pile to pre-test position kN	1183	766 - 1677	≈ 1920

3.4 Comparison Between Predicted and Measured Values

The measured values in the field test are shown in Table I, together with the predicted values. It is immediately apparent that the largest discrepancy between prediction and measurement is in the strain and movement of point A after cutting, and this reflects an erroneously predicted initial load at the top of the pile. The load required to return the pile to its original position was also underestimated because of the underestimate in the initial load. It was determined from the measured deflections that this initial load must have been about 1090 kN i.e. two to three times the values assessed by the predictors. This difference suggests considerable non-uniformity of the load distribution within the group, and even the possibility of some piles carrying tension. This suggestion was confirmed by the subsequent finding that two of the batter piles had pulled out of the pile cap. This in turn indicates that the effects of negative friction on battered piles are much more severe than on vertical piles, as they are subjected to both normal and axial components of soil movement.

The predictions of the pile behaviour subsequent to the initial cutting (Steps (c) and (d) are in reasonable agreement with the measurements, thus demonstrating that the assessments of the negative friction and electro-osmosis treatment were satisfactory. An evaluation of the measured pile deformations indicated that the maximum downdrag force in the pile, prior to excavation, was about 1100 kN, as compared with the authors' prediction of 1375 kN.

The predicted and measured effects of electro-osmosis cannot be compared in detail because the field test procedure differed from that assumed in the prediction. In the field test, an initial test was performed using a 12 volt car battery, and then a second test was carried out using a welding generator at outputs of 30, 45 and 60 amps. The second test was continued until the pile movement ceased; however, it was found that 70% of the total movement was realized in the first 1.5 hours. This rate of movement, and hence downdrag load relief, is much more rapid than predicted. Fig. 4 shows the result of the electro-osmotic treatment and indicates complete relief of downdrag after only about 120 minutes whereas the predicted time was about 2 days. This discrepancy may be attributable to the assumed values of k and c_v being too low, or the assumed value of k_o being too large.

Despite the differences between predicted and measured behaviour, three clear points emerge from the Cutler Bridge Pile Test:

(i) large downdrag forces can be developed in piles due to negative friction

(ii) the effects of negative friction on battered piles are much more severe than on vertical piles

(iii) electro-osmosis can be effectively used to reduce downdrag forces in piles in a very short time and with a relatively small expenditure of electrical power.

4 MODEL PILE TEST

4.1 Apparatus

To obtain further data on the development of negative friction and its relief with electro-osmosis, a test was carried out on a model pile under controlled laboratory conditions. The apparatus was similar to that previously used for model pile tests by Mattes and Poulos (1971). The pile was situated in a cylindrical pressure vessel consisting of three sections, a base section through which drainage could be provided and into which the pile was screwed, a centre section approximately 400 mm long which was filled with soil, and an upper section which was separated from the centre section by a rubber membrane and which contained water under a controlled pressure. The pile itself consisted of a 25.4 mm diameter aluminium tube, 400 mm long, and had four strain gauges installed near the pile base in order to determine the axial load. To remove the possible effects of pile bending the mean of the four strain gauge readings was calibrated against axial load.

After the pile was screwed into the base, remoulded Kaolin (LL = 55, PL = 33) was placed around the pile, at about the liquid limit. By applying a pressure to the surface of the soil and allowing drainage at the base and top, the clay could be consolidated, thus inducing negative friction and downdrag

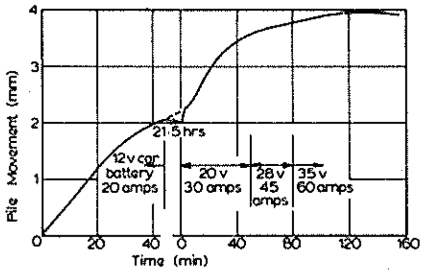


Figure 4 Pile movement vs. time during electro-osmosis-cutler circle bridge

into the pile. The axial load at the pile base could be determined from the strain gauge readings.

To enable application of electro-osmosis to the pile, a single 2 volt cell of a heavy duty battery was used as a voltage source, the potential difference being applied between the pile base (cathode) and the outer wall of the pressure vessel (anode).

4.2 Test Sequence

The test pile was subjected to the following sequence of events:

(a) an initial effective consolidation pressure of 34.5 kPa was applied to the soil as a "seating" pressure (a back pressure of 69 kPa was used for the pore water).

(b) the consolidation pressure was increased by 34.5 kPa and readings of axial load versus time taken over a period of about 2 days

(c) electro-osmotic treatment was applied to the pile by imposing a 2 volt potential difference between the pile and the vessel wall for 30 minutes. The voltage was then removed and a rest period of approximately 6 days was allowed, during which the consolidation pressure remained unchanged

(d) a further 34.5 kPa pressure increment was applied for a one-day period, with readings of axial load versus time being taken

(e) electro-osmotic treatment was applied (2 volts for 30 minutes), followed by a rest period of approximately 6 days during which the consolidation pressure remained unchanged

(f) a final pressure increment of 34.5 kPa was applied for a period of about 2 days.

4.3 Measured Behaviour

Fig. 5 shows the axial load versus time relationship for the pile, for the whole test sequence subsequent to the initial consolidation stage. The following observations may be made:

(i) the first application of electro-osmosis causes a significant and rapid reduction in downdrag load, and there is little recovery of this load in the 6 days following the treatment

(ii) the second electro-osmotic treatment has a similar short-term effect on the downdrag load; however, a complete recovery of the downdrag load occurs in the first 12 hours or so following cessation of the treatment.

The reason for the re-development of downdrag after the second treatment is probably that the treatment was applied at a much earlier stage of consolidation. The remaining consolidation settlements which occurred after cessation of the second treatment were sufficient to again cause full pile-sand slip and cause a build-up of downdrag load to the level it would have achieved if no treatment had been applied. In contrast, the first treatment was applied when the consolidation of the clay was nearly complete, and consequently, there was little further settlement and little recovery of downdrag load.

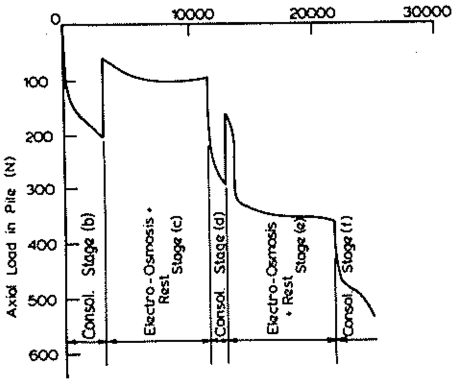


Figure 5 Time-load history of model pile

The implication of these results is that, in field situations, the reduction of negative friction by a short period of electro-osmosis may only be temporary if significant consolidation settlements can occur after the treatment ceases. In such cases, it may be necessary to apply frequent short periods of electro-osmotic treatment in order to prevent a long-term build-up of downdrag load.

4.4 Comparison With Theory

4.4.1 Downdrag force versus time

The solutions presented by Poulos and Davis (1975) were used to predict the magnitude and rate of development of downdrag load in Stages (b), (d) and (f) of the test. On the basis of previous tests, $K_s \tan\phi_a'$ was taken as 0.15 while, from oedometer tests, c_v was taken to be 0.38 mm^2/min. The theory indicated that, for all three Stages, pile-soil slip should occur along the pile during consolidation, and thus estimates of the elastic parameters of the soil were not necessary.

Fig. 6 compares the theoretical and measured load-time relationships. The load is the sum of the downdrag load at the pile base and the immediate axial load increase on the pile head caused by the increase in consolidation pressure. Because the same increase in pressure occurred in all three stages and the pile and soil parameters were assumed to remain constant, a single theoretical curve applies to the three cases. There is some variability between the three measurement curves, and the loads near the end of consolidation vary rather erratically, possibly due to drift of the strain gauges. Nevertheless, the overall agreement is quite reasonable and the theory appears to be capable of giving a satisfactory prediction of the magnitude and rate of development of downdrag force in a pile.

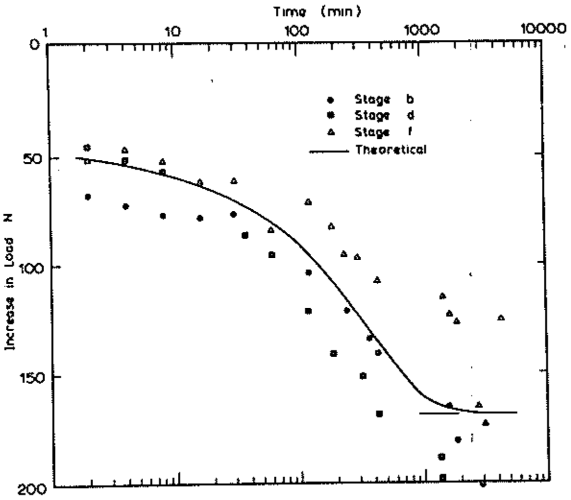


Figure 6 Measured and predicted load vs. time

4.4.2 Downdrag reduction due to electro-osmosis

The reduction in downdrag load due to electro-osmosis was determined by integration the change $\Delta\tau_a$' in pile-soil resistance (Eq. 3) over the pile surface. The excess pore pressure u at various times was obtained by numerical solution of Eq. 2. For this model test, the boundary conditions differed from those of the Cutler Circle pile test in that the anode in the model test was the circular wall of the pressure vessel, rather than an adjacent pile. A solution to the model test problem was obtained by Townley and Lo (1975) and is shown in Fig. 7.

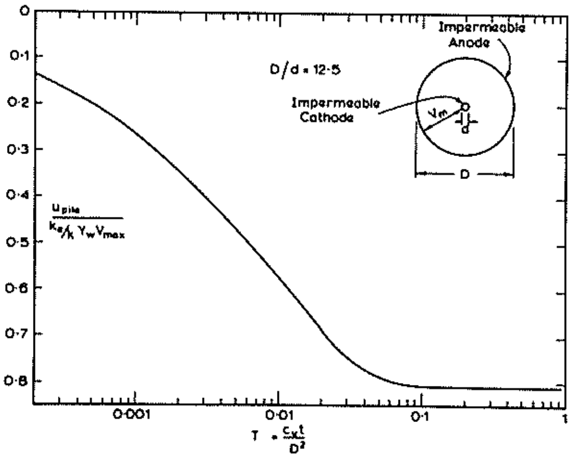


Figure 7 Theoretical solution for model pile

The magnitude of the final maximum excess pore pressure developed at the pile is given by

$$u_{max} = \left(\frac{R^2}{R^2-1} - \frac{1}{2\ell nR}\right)MV \tag{4}$$

where R = r_e/r_p (here R = 12.5)

V = applied potential difference between electrodes

M = is defined in Eq. 2 and r_e and r_p are defined in Fig. 8.

Tests were not carried out to determine the value of M for the kaolin and therefore various values were assumed within the range of previously - experienced values of 5 to 30 kPa/volt.

Fig. 8 shows the observed relationships between downdrag reduction and load obtained from Stages (c) and (e), together with the theoretical curves for three values of M. The two experimental curves are similar and are in quite good agreement with the theoretical curve for M = 24 kPa/volt. When the electro-osmosis was halted after 30 minutes, the theory suggests that only 75% of the final excess pore pressure had been developed. Consequently, for M = 24 kPa/volt, a further reduction of about 43 kN in the downdrag load could have been achieved if the electro-osmosis had been continued.

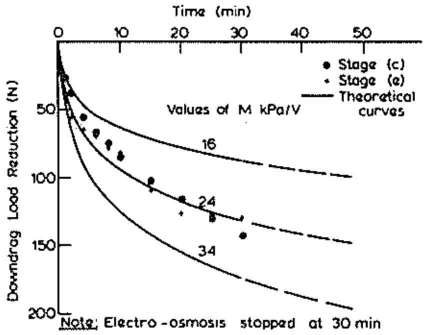


Figure 8 Comparison between measured and theoretical downdrag reduction due to electro-osmosis

5 CONCLUSIONS

The results of the field and model tests described herein clearly demonstrate that electro-osmosis can be effectively used to rapidly reduce downdrag forces in piles subjected to negative friction. The permanence of this reduction depends of the amount of soil movement which will occur following the cessation of electro-osmotic treatment. If this treatment is applied at a late stage of consolidation, there may be only small further downdrag forces developed, whereas, if the treatment is applied at a relatively early stage of consolidation, almost complete recovery of the downdrag force may occur.

The comparisons between theory and measurement, although limited, indicate that the theoretical approach used can predict, with fair accuracy, the downdrag force in the pile, its rate of development and the reduction of electro-osmosis to the pile. The main problem in applying this theory is (as is usually the case in geomechanics) the selection of the appropriate soil parameters, including here the electro-osmotic parameter M.

6 ACKNOWLEDGEMENTS

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