Negative Drag on an Instrumented Pile — A Field Study

Trainage Négatif sur Pieu Instrumenté — une Etude sur Chantier

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SYNOPSIS A programme of field measurements on a bored cast-in-situ instrumented concrete pile was undertaken in the marine clay deposits of Visakhapatnam dockyard. The pile was 43cm diameter and 5m long. It had strain gauge type load cells embedded at three levels. As the soft saturated clay deposit extended to a considerable depth, the pile top was held stationary by hanging it from a yoke. The negative drag was generated by loading the ground around the pile. It was monitored by observations on the load cells embedded in pile. Piezometers and settlement gauges were installed around the pile. To accelerate the development of negative drag on the pile, the area of 6m around the piles was provided with sand drains of 15cm diameter. The embankment load/unloading was done in three stages of 1t/m^2 each and the test observations were continuted over a period of more than one year. The data is analysed by effective stress and total stress approach and compared with the negative drag observed on the pile. It is shown that the distribution of drag along the pile length keeps on changing with time and loading. Any movement of pile can further modify this distribution. For determining ultimate skin friction, the pile was load tested. The paper compares the prediction and actual performance of the pile.

INTRODUCTION

The fact that a pile resists superimposed load by developing point bearing at the toe and skin friction on the stem is well known. But the phenomenon of 'negative skin friction' or 'down drag' and consequent additional loading on pile toe were only recognised in the beginning of this centuary. The first published case on record is by Warcester in 1914. Since then, particularly during last three decades, several experimental and field studies have been reported. As the negative skin friction developes on the pile-soil interface, studies were directed towards observing it on piles instrumented at various levels along its length. The study reported herein is on an instrumented pile cast in a bentonite filled borehole in ground reclaimed by depositing dredged material from sea over soft marine silty clay.

SUBSOIL

The pile test site is situated at Visakhapatnam, south east coast of India. The boring record (Fig. 1) shows that the top 3m is a dredged fill of clay with shells. This is followed by about ½m layer of sand. Below this lies 14.5m deep very soft clay deposit. The moisture contents are higher than liquid limit and penetration resistance is very low. The unconfined compression tests and in-situ vane shear tests show an average cohesion value of 1.4t/m^2 between ground level and 24m depth. There is not much increase in strength with depth. The test results indicate a slightly underconsolidated nature of deposits.

The very soft clay deposit is followed by two strata of stiff silty clay and very stiff clay of 4m and 5m depths respectively. The weathered rock is met at about 27m depth. It is of Khondolites and Charnockites which belong to Archaens.

INSTRUMENTED PILE AND TEST SET UP

The test pile was 43cm diameter and 5m long. It was reinforced with four bars of 16mm diameter and 8mm diameter helical stirrups. To accommodate the tremie for

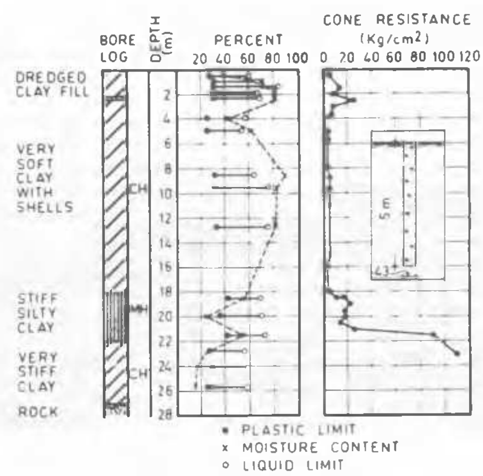

Fig. 1 Subsoil Details

concreting under bentonite slurry the three load cells mounted on steel tube (Fig. 2), were tied to the reinforcing cage along the longitudinal bars on one side. The concrete was of M200 grade (nominal mix 1:1½:3 with 10 percent extra cement) and the 28 days cube strength varied from 236 to 276 kg/cm^2.

Each load cell is a 100mm long and 52mm diameter unit with two concentric stainless steel tube pieces.

Fig. 2 Load Cells

The strain gauges are fixed on the outside of the inner tube of 25mm O.D. which is protected by the outer tube piece. The assembly is made water tight by sealing with O-rings and waxes. The strain gauges are mounted in full Wheatstone bridge configuration and respond to both axial compression and tension. All the leads of strain gauges were taken through the tube which is of the same diameter and thickness as the inner tube piece of the cell. A digital strain measuring bridge of one microstrain accuracy was used for recording strains.

The pile was firmly held from top by a yoke through which a girder passed which in turn was supported on cross girders resting on firm support of the ground (Fig. 3)

Fig. 3 Test Set Up

A load gauge was introduced between the yoke and the cross girder such that as the pile is dragged down, the load gauge shows corresponding load in compression.

The area around the pile was provided with 15cm diameter sand drains at 1.5m spacing in a radius of 3m. It was then sand blanketed. To observe the development of pore pressures two casagrande piezometers at 2m and 4.5m depth were installed in the loaded area. Settlement gauges both at surface and depth were installed at nine locations.

THE TEST AND OBSERVATIONS

To create downward drag on the pile the area around it was loaded by sand in three equal increments of $1t/m^2$ each. An increment was given when settlement got stabilised. The drag was read from the load gauge at the pile top. The distribution of drag along the pile depth was recorded by the load cells in microstrains which was converted to stress on the pile cross-section at the location of the cell. From the three cell readings a curve for distribution of negative skin friction can be drawn at a particular stage and subsequently total drag can be estimated on the entire pile surface. Better reliance can be put on the readings of the load gauge at pile top for registering negative drag, while load cells along the stem give a picture of its distribution and change with load and time. The unloading followed in similar steps. The test extended over a period of 480 days. The loading stages and load cells readings and excess pore pressure development at 4.5m depth are shown in Fig. 4.

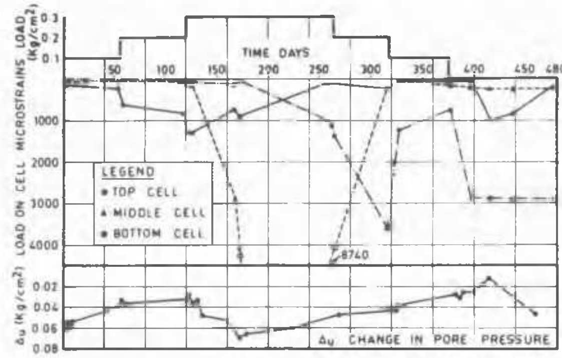

Fig. 4 Load, Time Versus Load Cell Readings and Pore Pressure

The distributions of drag in terms of strain shown by the load cells at few stages are shown in Fig. 5

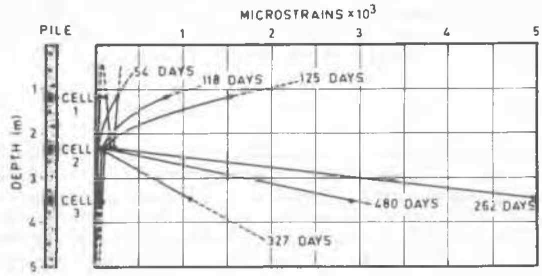

Fig. 5 Variation of Load Cell Reading with Depth

At the end of the test the pile was load tested by pull out to evaluate the ultimate skin friction. The pull out load-deflection curve is shown in Fig. 6. The ultimate value corresponds to about 5 percent of pile diameter.

DISCUSSION

Estimation of negative skin friction had two approaches : (I) the total stress method and (II) the effective stress

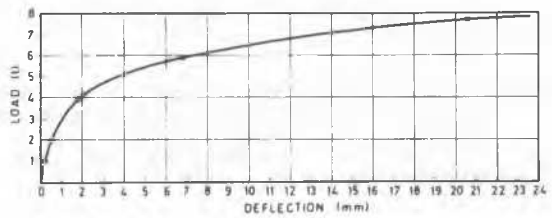

Fig. 6 Pull Out Test

method. In case clay,by total stress method, the skin friction f_s is given by

$$f_s = \alpha \, . C_u \qquad (1)$$

where α is a coefficient depending the pile material and surface, and C_u is undrained shear strength

By the effective stress method,

$$f_s = \sigma_v' \, .K.\tan\, \phi_a' \qquad (2)$$

where σ_v' is vertical effective stress, ϕ_a' is the effective friction angle at soil-pile interface and K is the coefficient of lateral earth pressure. K depends upon the soil properties and type of pile and its method of installation.

By putting k tan $\phi_a' = \beta$

the equation (2) becomes $f_s = \beta \sigma_v'$ (3)

The values of the coefficients α and β from the test are given in Table 1 for three stage loading.

TABLE I

Values of α and β

Days from start	Preload t/m^2	Total Drag t	Coefficient (average values) α	β
54	1.0	1.54	0.16	0.105
118	2.0	2.66	0.28	0.123
125	3.0	4.10	0.43	0.145

The load cell observations after the third step of loading and subsequent unloadings, show abrupt changes. During this period there was inundation of site by rain water and there was uncontrolled sinking of the supports of girders.

Fig. 5 shows two sets of load cell readings at 262 days and 480 days. The observations during the later part of test are not considered in the discussions. The coefficient α in Table I is calculated by estimating the ultimate friction from the shear strength determined by laboratory tests and field vane shear test and dividing this value by the total drag observed at various stages. The adhesion determined from the pull out test (Fig. 6) is 1.18 t/m^2, while the shear strength of soil (from tests) is 1.4t/m^2. Thus the adhesion factor (or reduction factor) is 0.84. This value in general is in agreement with recommendations for bored piles in soft clays.

It is clear from Fig. 5 that load cell readings change both with time and magnitude of preload. Thus depending upon the strain levels at various points along the pile length, the magnitude of drag stress and consequently the values of α and β will be different. It can be seen that during the loading stages, increase in cell readings with time under a particular load is not very marked. This is because of the presence of vertical drains which induce faster drag on piles and its substantial part developing within a few days of loading.

Johannessen and Bjerrum (1965) have reported an increase in β from 0.12 to 0.20 over a period of 1.3 years. In Table I the β value may be taken to the maximum values for the given preloads.

The change in α and β values with preload are quite marked (Table I). The range of α values are in agreement with the findings of others. The coefficient β has indicated that negative drag mobilised is about 10 to 15 percent of the effective stress for the preload of 1 to 3 t/m^2. Garlanger (1974) has proposed from analysis of test results that the β for clays may be taken as 0.20 to 0.25. Broms (1976) has suggested a value of 0.3 for clays of medium plasticity (I_p less than 50). The β value observed are therefore smaller. This difference will further increase if the decrease in β value is considered for reduced loading rates (Eide et.al, 1972), (Broms, 1979). In the present case the settlement rate is obviously much faster due to the presence of vertical sand drains.

The reasons for smaller values of β could be due to the different condition of drag development. The pile is in tension and any extension of pile under downward drag will tend to neutralise negative skin friction. Construction of pile under bentonite slurry may also effect the drag forces. Drag recorded could be more if the piles were in compression and α and β would have shown increased values. However, the test set up has the merit of simplicity and ease of recording the drag.

CONCLUDING REMARKS

1. Holding the pile from top and reading the drag by a load gauge can be used as a simple method to estimate negative skin friction and to evaluate average values of the coefficients on smaller lengths of pile. To cut down the time, faster consolidation can be achieved by vertical drains around the pile.

2. Both the total stress method and the effective stress method can be used to evaluate negative skin friction.

3. The coefficients α and β increase with increase in preload.

4. The values of coefficient α were found to be more or less in agreement, but β was about half of those suggested in literature for clays of similar nature.

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