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Down Drag on an Instrumented Bored Pile in Soft Clay

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SYNOPSIS The paper reports result of a comprehensive full scale field study expressly undertaken to monitor negative drag on a large diameter, bored cast-in-place reinforced concrete pile installed to rock in a deep deposit of soft marine clay of pleistocene to recent origin. The pile was instrumented with load cells and the ground around the pile with piezometers and settlement gauges. The negative drag was generated by loading the ground around the pile in stages. The ground settlements were accelerated by providing vertical sand drains. The observational data were utilized in figuring out the influence of surcharge on the depth of clay responsible for generating the negative drag. The study has provided computational methodology for estimation of negative drag in terms of total and effective stresses. The inevitability of large ground settlement with respect to the pile brought out the non-importance of precisely ascertaining the magnitude of relative displacement necessary for mobilization of negative drag.

INTRODUCTION

The unprecedented construction activity in the Indian coastal belt, Gulf of Kutch and on the shores of the Gulf of Cambay, where soft clays are in abundance, have focussed attention of the engineers on the unsatisfactory behaviour of foundations in general and on foundation failures in particular. The constructions in the past were as a matter of routine supported on pile foundations of either bored or of cast in place types. Failure of these foundations were invariably explained as a consequence of negative drag, the quantification of which was regarded as a matter of guess. Although the present state-of-the-art on negative drag throws enough light on the quantitative aspects of this phenomenon (Poulos, 1975, Brom, 1979 and Bozozuk, 1981), the fact remains that the practicing geotechnical engineer still fumbles with many uncertainties due to lack of a clear-cut understanding of the mechanics involved. The various theories or approaches do lead to a wide variance in the ultimate result adding to the strain on the designer. The following basic questions always came to fore:

1. How much is the total depth of clay that contribute to the development of negative drag? In other words, at what depth does the transition takes place from negative to positive friction?
2. What is the scientific methodology of estimating negative drag? The merits and demerits of total and effective stress approaches become important in this context.
3. In what manner does the distribution of negative drag around a pile vary with time?
4. What is the influence of spatial pore water pressures induced due to the combined effect of pile construction and surcharge loading?

The study reported in this paper was undertaken with a view to finding satisfactory answers to some of the above questions.

On a point bearing bored cast-in-place pile, the negative drag was introduced by consolidating the ground around it under load placed in stages. For accomplishing the study in a manageable time span, vertical sand drains were installed in the ground around the pile. The magnitude of preload was then placed to match the floor loading in the buildings of the area. The development and dissipation of pore pressures and settlements were monitored as functions of surcharge load and time.

THE TEST SITE

The full scale experiment was carried out at a site in the naval dock yard area (Fig 1) at Visakhapatnam located on the south eastern coast of the Indian subcontinent (latitude $17^{\circ} 36'N$; longitude $83^{\circ} 20'E$). The location of the test pile and the anchor piles are shown in this figure. Those of the sand drains, piezometers and the settlement gauges are shown in Fig 2.

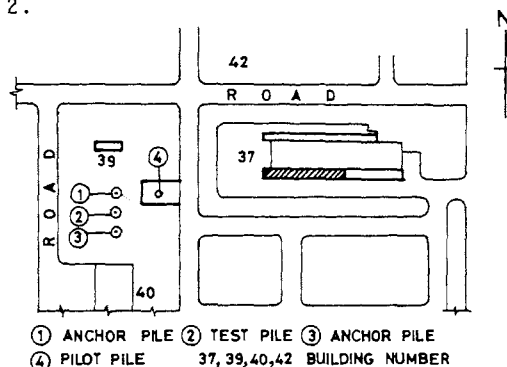


Fig. 1. Site Plan

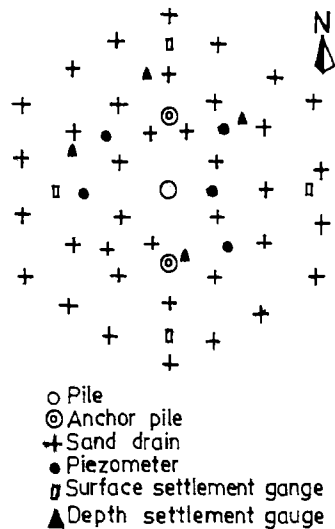


Fig. 2. Position of Instruments

Two bore holes were made and a large number of undisturbed samples were recovered for laboratory studies using thin walled piston samplers. The samplers had outer diameter of 85.6mm, inner diameter of 82.3mm and length of 455mm, yielding an area ratio of 8.8 per cent. A large number of Standard Penetration Tests, Static Cone Penetration Test and Vane tests were conducted. A number of open stand pipe piezometers were installed at different depths to obtain the picture of ground water conditions prior to launching of the experiment. A summary of the subsoil exploration data and test results is furnished in Fig 3.

Top 3m of subsoil represents dredged fill of clay with shells showing over-consolidation effect. This rests on about 0.5m thick layer of sand below which lies 14.5m deep bed of soft compressible clay. The soft clayey strata inturn rests on stiff to very stiff clay deposit

($Cu > 20t/m^2$) 4m thick followed by weathered rock met at about 27m depth. The natural water content of this deposit ranges from 44 per cent to 90 per cent and is generally found to be higher than the liquid limit of the clay.

The grain size distribution band based on a large number of hydrometer test results is presented in Fig 4.

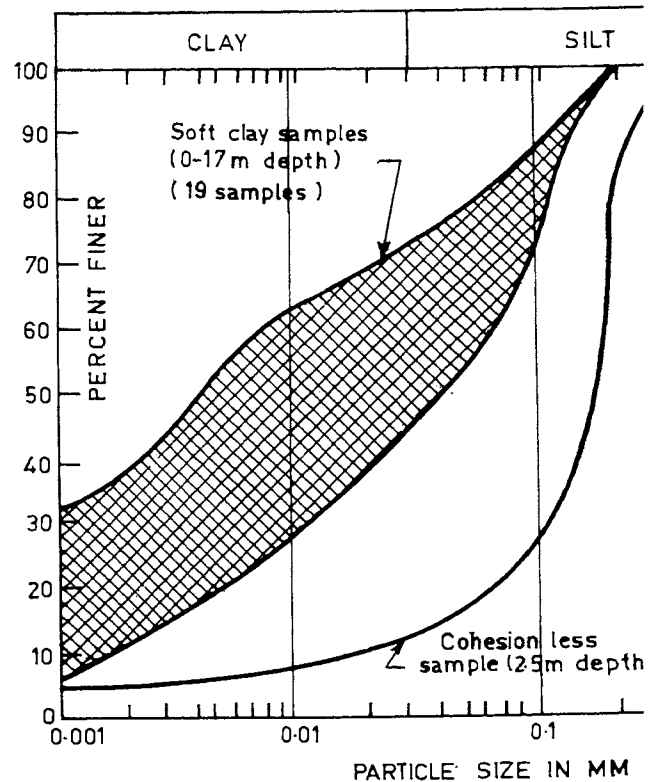


Fig. 4. Gradation Curve

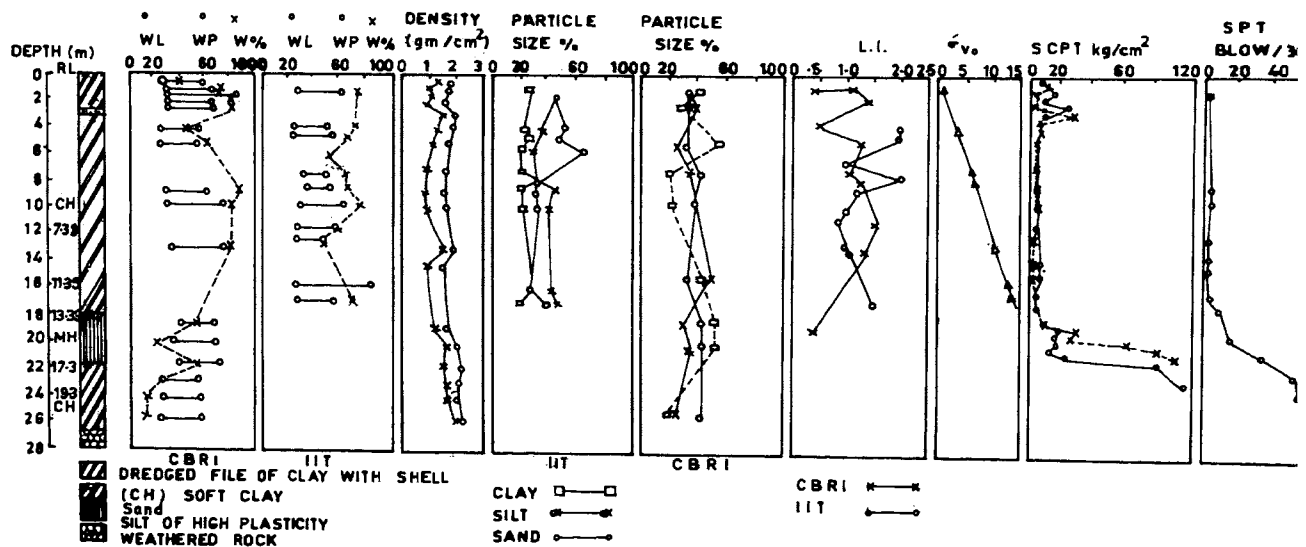


Fig. 3. Sub-soil Characteristics

The vane shear tests, the triaxial tests, the classification tests, and the results of the pull-out tests on piles in the area all combine to suggest a very low average undrained shear resistance of clay in the range of $1.4t/m^2$ - $2t/m^2$.

The other major property investigated was the compressibility of the soft clay in Oedometer tests conducted on undisturbed samples recovered from different depths. Very large number of results were synthesized to arrive at a relationship between $(C_c/(1+e_0))$ and the corresponding values of natural water content.

Test results clearly reveal that Visakhapatnam clay fits the model proposed by Lambe & Whitman (1969) according to which:

$$\frac{C_c}{1+e_0} = 0.358 \log w - 0.448$$

for $w \geq 34$ per cent

The above correspondence provides evidence of the normally consolidated nature of the clay.

Lambe & Whitman's relationship can, therefore, be regarded as a reliable and powerful basis for arriving at the values of compression index (C_c) with the knowledge of initial void ratio and natural water content, the scatter being within ± 10 per cent (Fig 5).

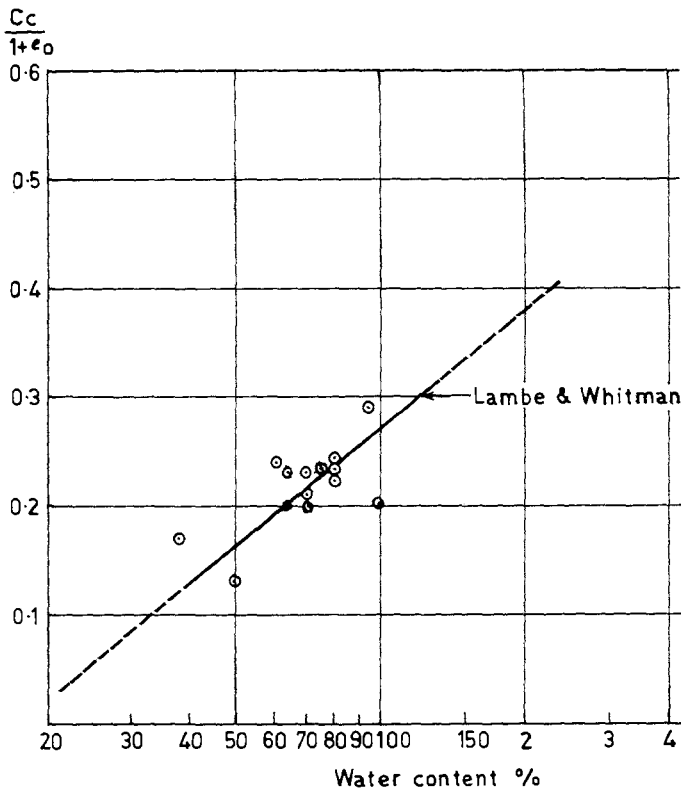


Fig. 5. $\frac{C_c}{1+e_0}$ and water Content Relationship

The piezometric observations extended over a long period of time at no load condition provide testimony to the normally consolidated nature of the clay strata. Results of seven

such measurements (Fig 6) suggest that ground water exists at 0.5m below the ground level and the ground water pressure distribution is hydrostatic.

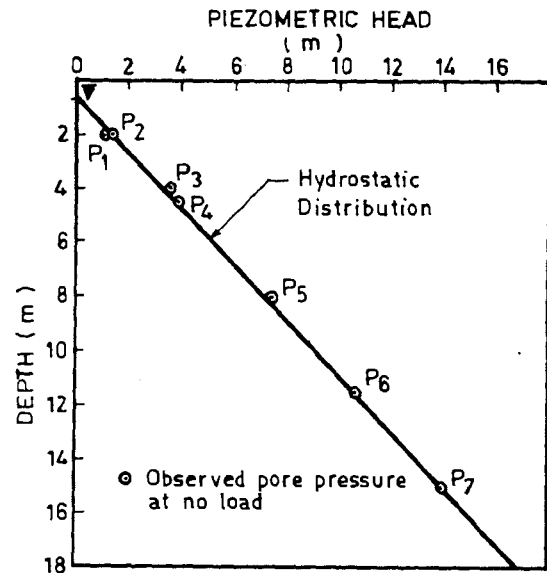


Fig.6. Measured depth Distribution of Pore Pressure

From the ground water observations reported above, the effective vertical pressures at various depths are calculated and plotted against plasticity index values in Fig.7. These results lead to the prediction of undrained shear strength values following Skempton's well known relationship between undrained shear strength, effective overburden pressure and the plasticity index. The predicted undrained shear strength values and those obtained by thick wall tube samplers and thin wall piston samplers are plotted in Fig 8. The observations bring to fore the tremendous impact of sample disturbance, warns against use of thick wall samplers and underlines confidence in the range of undrained shear strength specified earlier as $1.4-2t/m^2$.

THE PILOT PILE

Before launching the experiment with a full size pile, it was considered desirable to have a feel of the various facets of the problem by conducting a small scale study. The main issue to be settled were:

- selection of instruments and evaluation of their efficacy under field conditions.
- problems associated with the construction of sand drain using open-end mandrel method.

The pilot pile was constructed using bored, cast-in-place process. It was 43cm in diameter, 5m deep and was reinforced with 4 bars of 16mm diameter with 8mm diameter helical ring tied 25cm apart. Concreting was done using nominal mix of 1:1½:3 (M20 grade).

The pile was instrumented with load cells assembled in the shape of an instrumented tube which formed a part of the reinforcement cage while casting of the pile. It was firmly held at the

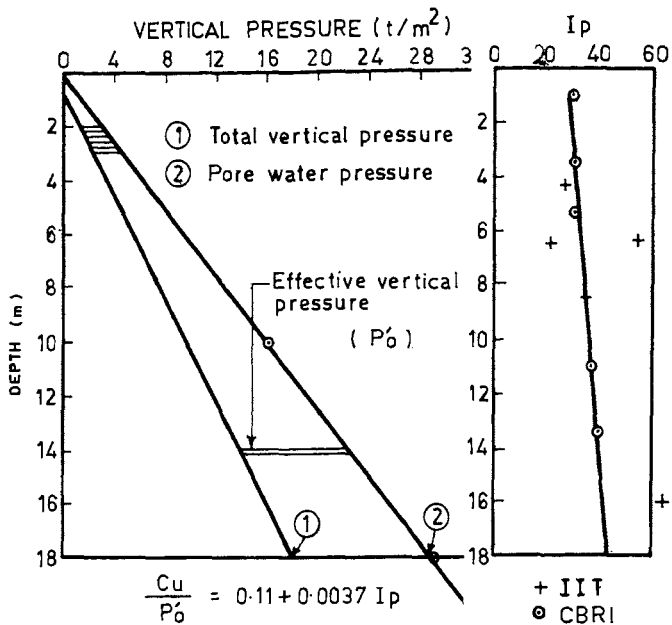


Fig. 7. Depth Distribution of Pore Pressure Total Pressure & Undrained Shear Strength

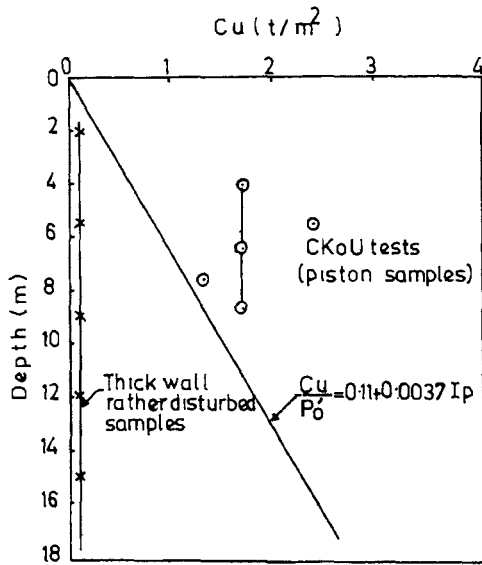


Fig. 8. Predicted Undrained Shear Strength from Pullout Test

top by a yoke-girder system. This was necessary to achieve point bearing for the pile.

Proving ring was introduced between the yoke and the cross girder to monitor load on the pile head.

The subsoil around the pilot pile was treated in a radius of 3m using sand drains, 15cm in diameter placed 1.5m apart in a triangular pattern. A blanket of sand, 15cm thick was laid to interconnect sand drain. Thereafter the entire ground was loaded in three equal increments of 1t/m².

Eventually pilot pile was tested in pullout to estimate the ultimate shaft friction value. The results of observations on the pile instruments, piezometers and those of the pullout tests are discussed by Mohan et.al (1981). The data are utilised in analysing results of the present study.

THE MAIN TEST PILE

The test pile was 66cm diameter and 28.4m deep with 0.4m of pile toe embedded in the weathered rock to ensure the point bearing effect. It was constructed in March 1979.

The pile reinforcement consisted of 12 bars of 32mm diameter high tensile steel. Additionally, it had two diametrically opposite 60mm O.D. (4mm thick) m.s. pipes on which load cells were mounted. The reinforcement cage (Fig 9) was lowered in three stages and welded in situ while lowering into the bentonite filled bore hole. The design of reinforcement was carried out on the consideration that the test pile may be subjected to a compressive load of 450t and uplift

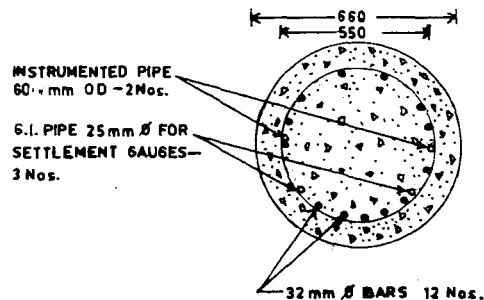
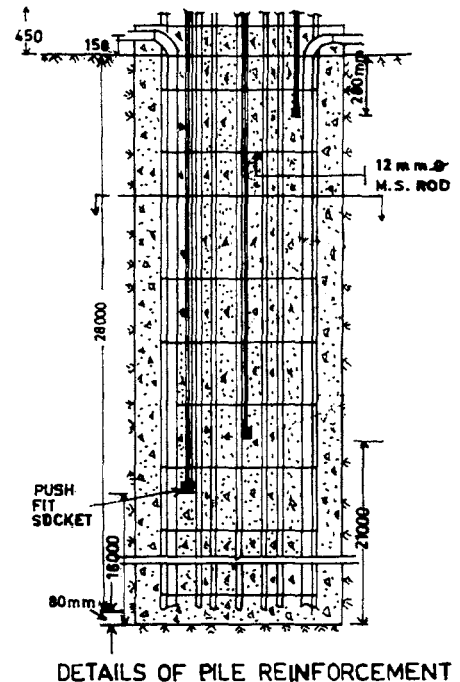


Fig.9. Details of Pile and Pile Reinforcement

load of 150t, to allow final load test. The concreting was done by tremie method using a nominal mix of 1:2:4 with 10 per cent extra cement. Two more piles of similar specification spaced at 2.5m on both sides of the test pile were constructed to serve as anchor piles.

The concrete was of a high slump (150-200mm). The three piles showed almost similar consumption of concrete quantities which agreed well with the borehole volume.

GROUND TREATMENT

The ground around the pile was treated with sand drains of 30cm diameter, 18m deep spaced at 2m centres in a radius of 6m. A total of 36 sand drains were provided. The area around the test pile was sand blanketed to a thickness of 40cm in circle of radius of 9m from the centre of the test pile.

INSTRUMENTATION

Open stand pipe type piezometers were installed at depths of 2m, 4m, 8m, 11.5m and 15m to record (a) initial ground water condition and (b) development and dissipation of pore water pressure upon loading. Some piezometers at 2m, 4m and 8m depths were also installed outside the loading area so as to record the change of water pressure with time under no load condition.

Sleeved cone type depth settlement gauges were placed at 4m, 8m, 12m and 16m depths to observe depthwise settlement of ground under each increment of load. Additionally surface settlement plates were placed at four locations. Appropriate sleeves were provided to eliminate effect of soil friction.

FIELD OBSERVATIONS AND DISCUSSIONS

To create a down drag on the pile, the area around the pile was loaded in two increments 0.32 kg/cm^2 and 0.16 kg/cm^2 in a circle of 18m diameter. The loading was spread such that developed stresses should be felt throughout. The total embankment loading of 0.48 kg/cm^2 approximately represents the practical situations of fill upto 3m. Unloading was done in steps, similar to those followed in loading.

Pore pressures and ground settlements were monitored throughout the test duration. The settlement of pile head was observed to be very small.

Depth of Clay Generating Negative Drag

The field study reveals that at a surcharge of 3.2 t/m^2 only top 8m of soft clay generates negative drag. The best field evidence of this is provided by the piezometric observations made at 2, 4, 8, 11 and 15m depths (Fig 10). Observations clearly show that the surcharge of 3.2 t/m^2 could actuate piezometers only at 2 and 4m but not those at 8, 11 and 15m. The point of transition from negative to positive skin friction was therefore at 8m. No sooner did the surcharge increased to 4.8 t/m^2 that water pressures shot up particularly in piezometers at 11 and 15m. The settlement measurements lend further credibility to the above observation (Fig 11 and Fig 12). It is readily seen in

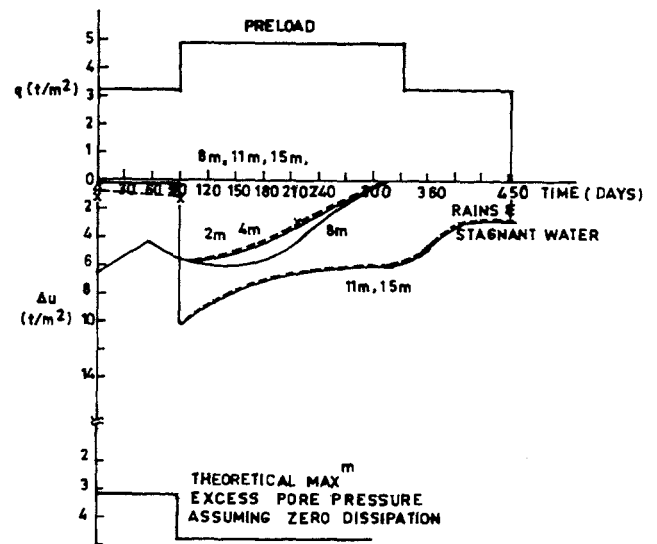


Fig.10. Excess Pore Pressure and Time-rate of Dissipation

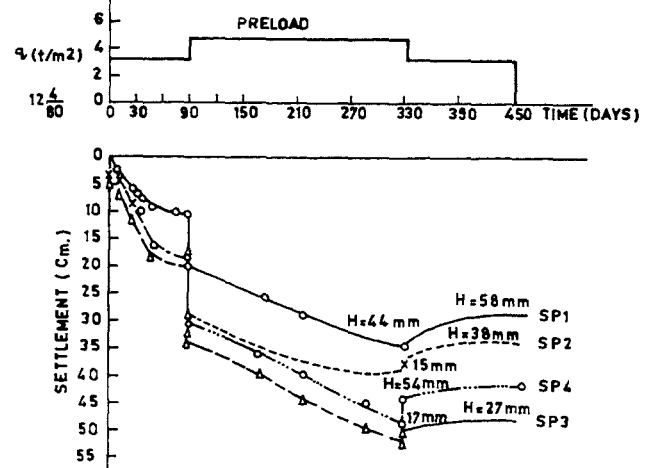


Fig.11. Load-Settlement-time curves (Surface settlement)

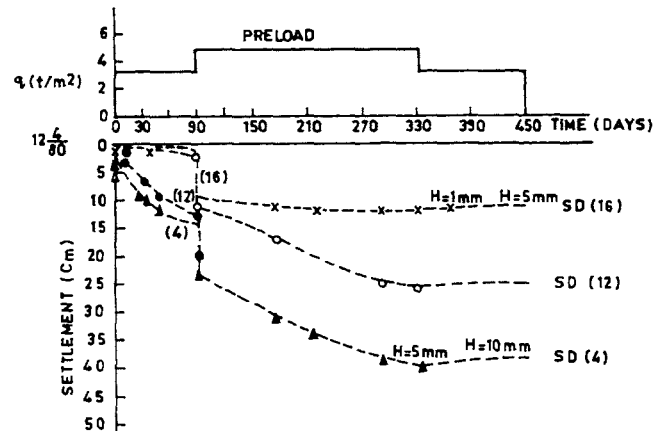


Fig.12. Load-Settlement-time curves (Depth Gauges)

Fig 12 that the settlement gauges at 12m and 16m did not respond much to the surcharge of

3.2t/m² whereas the one at 4m did. At the surcharge of 4.8t/m², however, all settlement gauges did respond. The entire depth of soft clay was, therefore, responsible for generating negative drag.

The observations of settlement at depths of 0, 4, 12 and 16m are combined in (Fig 13). This figure drives home the conclusion that if surcharge intensity exceeds 3t/m², the entire depth of clay strata should be considered in the estimation of negative drag. It is quite probable that the top 3m of the dredged fill which reflects the effect of over consolidation provided 'raft action' preventing transfer of stress below 8m depth under surcharge of 3.2t/m². Breaking up of the raft action at stress exceeding 3.2t/m² seem to explain sudden increase of excess pore pressures in deeper piezometers and sudden increase of settlements in depth settlement gauges.

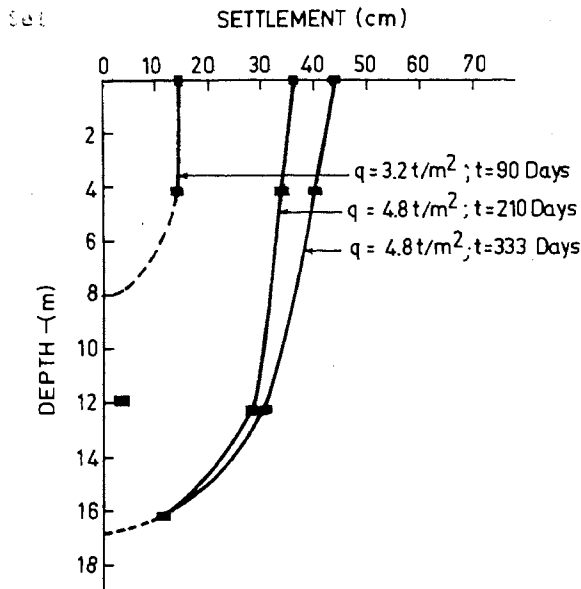


Fig.13. Depth Variation of Measured Settlements

Settlements, Settlement Rates and Engineering Implications:

The magnitudes of observed settlements corresponding to floor loadings of 3t/m² and 5t/m² at the end of 90 days, 210 days and 333 days are recorded below:

Table I - Observed and Predicted Settlement

Time (days)	Load t/m ²	Observed Settlement			Predicted Settlement (cm)
		Min (cm)	Max (cm)	Average (cm)	
90	3	10	20	15	14
210	5	26	42.8	34.4	35.5
333	5	35	52.5	43.75	60.2

The correspondence between the observed and the predicted settlements, (Fig 14) is considered reasonable. The total settlement at 5t/m² is estimated to be about 105cm. This is clearly excessive and would call for making up of the

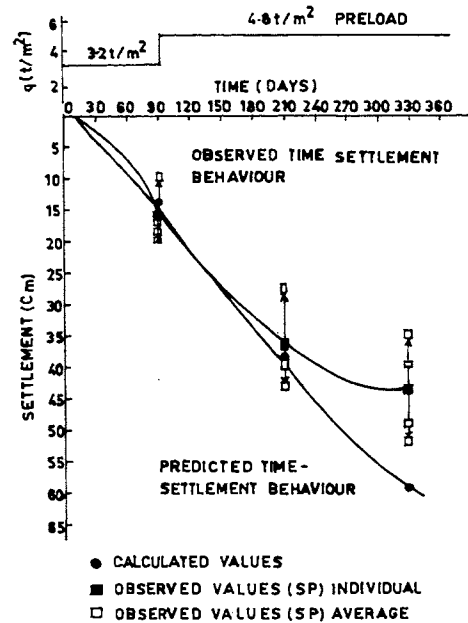


Fig.14. Observed and Predicted Settlement time Behaviour

grade level of flooring from time to time by additional filling. Such a measure would in turn generate further settlements.

The rate of settlement in the field study was hastened by sand drains. In practice floors are laid on virgin ground itself. As a result the problem of maintenance generally spans over the entire useful life of the structure.

The above observations drive home the point that it is not enough to design foundations for negative drag, the geotechnical engineer must also realise the implication of large settlements.

Yet another problem born of large settlements could be the development of additional lateral pressures on the inner face of a piled foundation. Piles are usually placed in a cluster. Those on the inner face of the pile cluster alone receive negative drag, the imbalance may lead to vertical and lateral forces on the pile cluster as a whole. These forces are not usually realised while making a design.

Magnitude of Negative Drag:

In engineering practice, the upper bound of negative drag is usually determined by doing pull out tests on individual piles. The pullout load is converted into average undrained shear strength of the soil in which the pile is embedded. Results of two such load tests (PT1 & PT2) are summarised below.

Back calculation reveals undrained shear strength to range between 1.4-2.0t/m² and that tally well with results of soil investigation.

The result of pull out test conducted on the pilot pile is presented in the shape of a load settlement curve in Fig 15. The ultimate pull out resistance is estimated to be 8t and from this the undrained shear strength is worked out

Table II - Pile Pullout Results

Test	Pile Dia- meter (m)	Pile depth (m)	Ultimate pullout resistance (t)	Displacement (mm)	$C_u(t/m^2)$
PT1	0.66	16	87	78	2.2
PT2	0.66	16	58	-	1.4
PT3	0.43	5	8	24	0.75

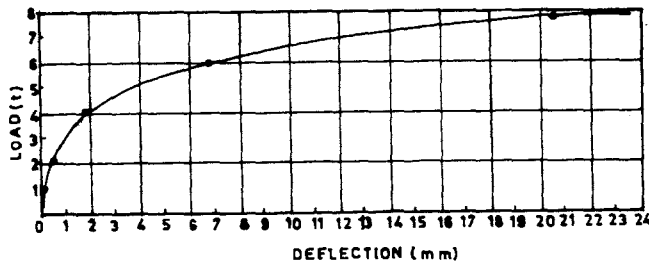


Fig. 15. Uplift Load Test on Pilot Pile

to be $0.75t/m^2$. This value is much lower than the average undrained strength for top 16m of soil strata probably because it refers to a shallow depth of 5m of the dredged fill at the top.

In terms of total stress the negative drag (P_n) can be written as:

$$P_n = \alpha C_u \pi DL$$

in which coefficient α stands for $\frac{P_n}{C_u \pi DL}$. The value of coefficient α estimated from pullout test and from the instrumented pile test are 0.46 and 0.53 respectively. For $C_u = 2t/m^2$, the negative drag for 0.66m diameter, 28m long pile embedded in 16m thick compressible strata works out to be 35t.

In terms of effective stress the negative drag (P_n) can be written as:

$$P_n = \beta \sigma'_v$$

and thus coefficient β could be written as $\frac{P_n}{\sigma'_v}$

Based on the results of pull out test and strain gauge values, β works out to be 0.22 and 0.24 respectively. Based on subsoil properties ($K_0 = 1 - \sin \theta = 0.7$; $\gamma = 1.63t/m^3$; $H = 5m$; $\Delta u = 4.5t/m^2$), β works out to be 0.2. For Vizag clay value of β is therefore seen to vary between 0.2 and 0.24.

For this range of values, the negative drag on the main pile is estimated to be between 40t - 48t.

The above results therefore puts the range between 35t and 48t. Negative drag of 48t could, therefore, be recommended for design.

Influence of Stress-relief on Negative Drag:

Superficially it may appear that by removing surcharge, resulting stress relief may cause ground heave thereby reducing settlement and

therefore, the negative drag. The results of the field experiment provides evidence that this does not happen. For example, the preload of $4.8t/m^2$, when reduced to $3t/m^2$, caused ground heave on the order of 50mm at the ground surface and 5mm at a depth of 16m. Such small order of heave only negligibly reduce the magnitude of relative settlement so that the negative drag value remains practically unchanged.

In actual foundation design, piles are subjected to axial load which do lead to pile settlement, thereby diminishing the magnitude of relative settlement between the pile and the ground. In the present case, even if the pile was to settle as much as 9cm, the relative movement between the soil and the pile will still be enough to generate full magnitude of negative drag.

The lesson one could learn from the above observation is that all piles in the Vizag area should be socketed in the hard stratum as they are in any case to be designed for full negative drag.

CONCLUSIONS SPECIFIC TO THE PROJECT

For floor loadings exceeding $3t/m^2$, all pile foundations which are likely to settle less than the surrounding soft clay deposit in the Vizag area must be designed for the full magnitude of negative drag. If the floor loading is less than $3t/m^2$, only top 8m of the soft clay strata participate in generating the negative drag. The measured load-excess pore pressure response corroborate this finding. At the flooring loading of $5t/m^2$, the entire 16-17m of the clay deposit is subject to incremental porepressures and consequent settlement. The magnitude of settlement is measured to be far in excess of that required to generate negative drag.

2. The negative drag on a 66cm diameter, 28m long bored pile is concluded to be 35t, if the interpretation of the observational data is made in terms of total stress. The value lie in the range 40-48t in terms of effective stress. The designer should, therefore, adopt a value of 48t.

The generalised calculations, total stress coefficient which lies in the range 0.46-0.53, can be taken as 0.5. The effective stress computations can be made using $\beta = 0.24$, its experimental range being 0.2-0.24.

3. The strain gauge measurements in short piles of 5m length suggest negative drag of the order of 9t. The negative drag along the pile shaft seem to readjust itself with time. The figure of 9t tally fairly with the result of pull out tests on pile.

4. The magnitude of observed settlements corresponding to floor loading of $3t/m^2$ and $5t/m^2$ at the end of 90 days, 210 days and 333 days are in Table I.

The correspondence between the observed and the predicted settlement is good. The total settlement at $5t/m^2$ is predicted to be about 105cm. This is clearly excessive and would call for making up of the grade level from time to time by additional filling. That would in turn lead to further settlements.

The rate of settlement in the field study was hastened by sand drains. In practice floors are laid on untreated ground and keep on settling slowly, calling for regular maintenance.

5. The unloading operation or reduction of floor load is observed to lead to stress relief and consequent heave of ground which is measured to vary from 50mm at the surface to 5mm at a depth of 16m when load is reduced from $5t/m^2$ to $3t/m^2$. The ground heave however, only negligibly reduce to magnitude of relative settlements and thus the negative drag value remains practically unchanged.

6. In actual foundation design, piles are subjected to axial load which do lead to pile settlement, thereby diminishing magnitude of the relative settlement between the pile and the ground. In the present case, even if the pile was to settle by 9cm, the effect of negative drag will still remain.

ACKNOWLEDGEMENT

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