# A Close Look at Problems of Research and its Application to Pile Foundations*

*by*

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## Historic Development

The word pile is derived from Anglo-Swiss 'Pil' meaning an arrow or a sharp stake: from Latin 'Pitum' meaning Javelin. Piles for support of pre-historic dwellings have been found in Lake Lucern. Romans used piles to build their ports and harbours. Caesar and his army had built a pile-supported bridge across river Rhine to march into the territory of the Sugambri. This is, infact, an excellent example of an early use of raker timber piles driven in to varying depths of water and stiffened with ties and braces. The design and the execution work undertaken at that time reflect full awareness of the dynamics of the forces of water current.

Piling systems used in India until after the Second world war were limited to pre-formed, solid displacement piles of either timber or concrete. Precast concrete piles were used restrictively for marine structures and driven, cast-in-situ piles were not very common. Timber piles of 15 to 20cm diameter and 5 to 6m lengths were used for the foundations of medium rise-buildings and other structures. Also, there are stray examples of the applications of coconut piles close to the Cochin harbour and screw piles for the piers of the bridge connecting the Willingdon Island to Ernakulum in Cochine. Uses of pipe-piles are also known, particularly for foundations of Jetties.

It was in the fiftees that my Institute (Central Building Research Institute : CBRI) had introduced for the first time in the country, the under-reamed piles (Dinesh Mohan & Jain, 1958). Although originally thought of as a fool-proof solution to the foundation problems in the expansive black cotton soil areas, these piles have now become popular even in other soils. In later years, my institute introduced bored compaction piles which combine the advantages of both bored and driven piles (Jain et. al, 1975)

Before fiftees, piling was largely the monopoly of a few construction firms who obtained the know-how abroad and often furnished conservative designs under the pretext of guarantee for safety. The client being ignorant of the 'mysteries of piling' and being afraid to interfere with the designs submitted by the firm, normally accepted these without questioning. It was only during the late fiftees when the Indian Standard Institution set up the Foundation Engineering Sectional Committee (which I had the privilege to Chair) that the users started questioning the various aspects of designs and a serious effort was made to unravel the 'mysteries' of 'the

pile foundations' by producing Standards and Codes of Practices on various facets of piling. The art and the science of piling kept advancing as the construction activities grew over the period of years. By the seventees, driven, cast-in-situ and bored piling systems became popular. Among the driven, cast-in-situ piles, the commonly used were Simplex, Vibro and Franki-each of which differ from one another in the manner in which the casing is driven and subsequently withdrawn from the ground as the concreting is done. Bored piles found favour in situations where hard stratum was encountered at a reasonable depth ; for example, in Bombay where the marine clay overlies the hard stratum.

Around 1950s, pile having capacities around 300 to 400 KN and in lengths of about 15m were about the maximum used. Today, driven cast-in-situ piles carry upto about 2000KN, penetrate to about 40m depth. Large diameter bored piles carrying 3000KN are also common. It now seems possible to construct such piles upto 2m in diameter and lengths uptc 65m. Use of drilling mud is made for stabilising the pile hole and the concreting is done by 'tremie process'. Sometimes, concreting is done under pressure to keep the water out of the pile hole.

Among the recent developments, one may mention precast piles in prebored holes, granular piles, lime piles, diaphragm walling elements etc.

India could justly be proud of its growth in the field of piling. The Indian Standard Institution deserves appreciation for being alive to the needs of the profession and for marching hand in hand with it. The Journal of the Indian Geotechnical Society (IGS) is known for some of the quality papers in the field of pile foundation analyses. Some good papers have also appeared in the IGS Annual Symposia. Some of you might recall the initiative my Institute (CBRI) had taken in Feburary 1964 to organise a symposium exclusively on the 'Bearing Capacity of Piles'. In September last year, the Indian National Group of the International Association for Bridge and Structural Engineering organised a seminar on 'Pile Foundations, Corrosion and Ground Anchors' which presents an excellent account of the Indian work related to pile foundations. Recently (5 Oct. 1980), a seminar on Piles was organised in Calcutta by the West Bengal Centre of the Institution of Engineers, India. All this is symbolic of the importance we geotechnical engineers attach to the many sided aspects of piling. Today, I wish to share with you my views on some selected problems of research and its application for the advancement of the art and the science of piling.

**Basic Questions**

The following basic questions normally arise in the analysis of a piled-foundation

1. Estimation of pile capacity from seemingly different predictions made by using dynamic formulae, static methods, and field load tests. Further difficulties may arise depending on which dynamic formula, which static method or which criterion for load test evaluation is favoured.

2. Estimation of load carried by shaft friction and by point bearing.

3. Predicting the group behaviour of a pile from the known load-displacement behaviour of a single pile.

4. Contribution of the pile cap when it rests clear-off the ground (free standing condition) or when it rests directly on the ground (resting conditon).

*Point Bearing*

To illustrate the first point, I would draw upon the results of a full scale study on bored under-reamed piles undertaken by my Institute (Mohan et. al 1976). A large number of load tests on under-reamed piles were conducted at a site (SP-SM deposit) close to CBRI. Values of ultimate load calculated by different methods from a typical load displacement curve for a single pile are reported in Table 1.

TABLE 1 Ultimate load for a single pile

| Criteria | Ultimate load (KN) |
|---|---|
| 1. Single Tangent Method | 133 |
| 2. Double Tangent Method | 93.5 |
| 3. Load corresponding to 50mm displacement (Terzaghi and Peck 1967) | 136 |
| 4. IS: 2911 (Part IV)-1979 | 111 |
| 5. From $\delta/p$ and $\delta$ plot as proposed by Chin (1970) | 136 |
| 6. From time-settlement plot | 137.5 |

Criteria 1 and 2, which rely on subjective judgement of drawing tangents, may result in erroneons estimates. The selection of load corresponding to any particular value of settlement (Criteria 3 and 4) arbitrarily fixed can also be open to question. Criteria 5 and 6 do give reasonable estimates of ultimate pile capacity provided a sharp break is not indicated in the load settlement curve.

Comparisons of the above kind may not always be satisfactory particularly when different criteria yield widely differing results. In such cases it may be wrong to blindly calculate the safe load on piles as the the least of the following (IS: 2911 Part IV).

(a) Safe load = 2/3 load corresponding to 12mm settlement.

(b) Safe load = ½ load corresponding to 10 per cent of pile diameter for uniform piles and 7.5 per cent of bulb diameter for under-reamed piles.

The point may be illustrated by considering the field load test results on a 456 mm diameter, 21.55 m long, driven, cast-in-situ pile in an alluvial deposit. The casing was driven using 60KN single acting hammer to a final set of 3 blows for the last 50 m and it took 750 blows to drive the pile. For this case the above two criteria yield safe loads of 1050 KN and 1475 KN respectively, depicting a very large difference. A pile of comparable dimensions at the same site yielded safe loads of 1210 KN and 1150

KN respectively. These two values are remarkably close, and corroborate well with the value computed using Hiley Formula, for final set of 3 blows for the last 25mm penetration. Putting such observations together, the researchers should look for the possible reasons of variation which may possibly be on account of defects in piles, testing of the piles without allowing for adequate setting time or errors of calibration of the hydraulic jack.

Differences in pile capacities due to different interpretations to a load test curve are relatively easy to reconcile but those on account of reasoning associated with the theoretical computations or with the choice of design parameters are far more difficult to resolve. Take for example the variability of recommendations for computing the bearing capacity factor $N_q$ (Figure 1). Even if one were not to question either the samples, tests or
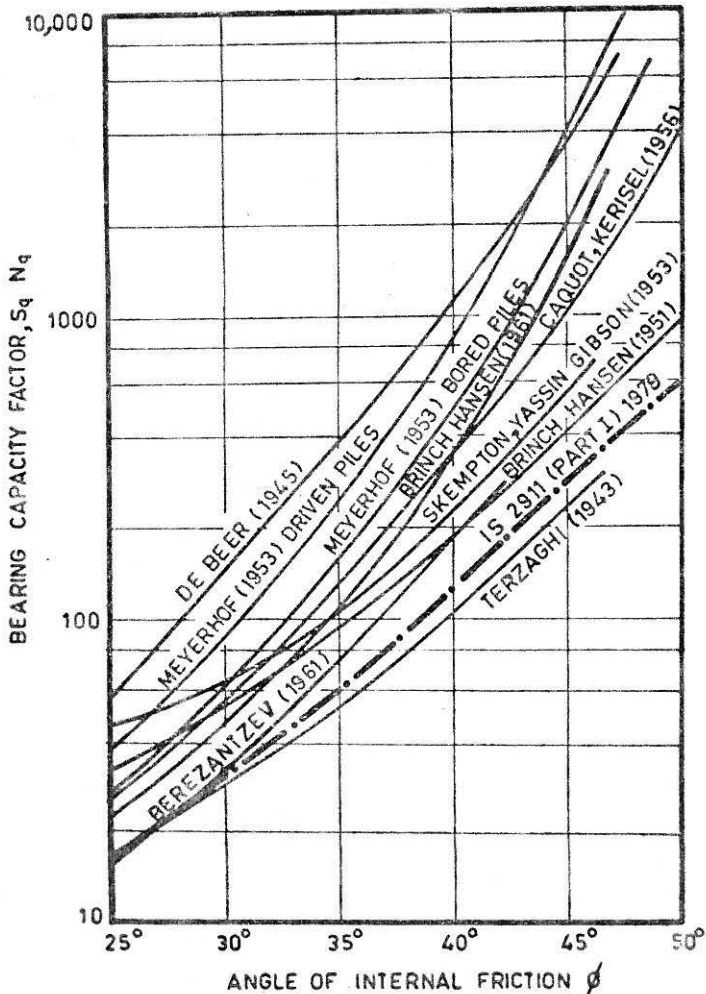


FIGURE 1 Theoretical bearing capacity factors

correlations that decide the value of angle of shearing resistance ($\phi$) or the influence of the method of installation, the calculated point bearing capacities would be widely different. In attempts to resolve the issue, the approaches based on the theory of plasticity are being replaced by more relevant non-linear elasto-plastic theories. However, the problem still remain unsolved and the Indian Standards could not do better than to side with the recommendations that fall well within the bounds of variation (Figure 1).

For bored piles in stiff clay the Bearing Capacity Factor $N_c$ is usually taken as 9 and the multiplier ($\alpha$) to undrained shear strength to be $0.75 \leqslant \alpha \leqslant 0.9$ depending on the pile diameter etc. Very little data from India seem to be available on this aspect as well.

*Shaft Friction*

The subject of estimating the unit skin friction of piles has also been under research for many years. The current state-of-the-art suggests that for soft clays, the undrained shear strength ($c_u$) is to be multiplied by a reduction factor ($\alpha$) to account for the effect of disturbances during pile installation. In driven piles, disturbance remoulds the clay to a distance of about one diameter. In the bored piles, it is believed to be restricted to a distance of about 25 mm from the pile. Besides remoulding, pile driven in clays lead to development of excess pore pressures. As the excess pore pressures dissipate, adhesion builts up. On the othe hand, in a bored pile the excess pore pressure may be much less but the clay surrounding the pile may considerably soften (*a*) by absorbing water from the concrete of the pile and (*b*) due to stress release. The value of $\alpha$ is therefore, known to depend on the type of the soil, the type of pile, the undrained shear strength of the soil, the method of installation of the pile and time since its installation. The recommendations coming from various researchers, e.g. Tomlinson (1957), Peck (1958), Woodward, Lundgren and Boitano (1961), Flaate (1968), Tomlinson (1970, 1971) are divergent. Some favour the recommendations by Flaate (1968) because it atleast represent an attempt to recognise the significance of the plasticity index of the soil (Figure 2). Meyerhof's (1979) recommendations cover both
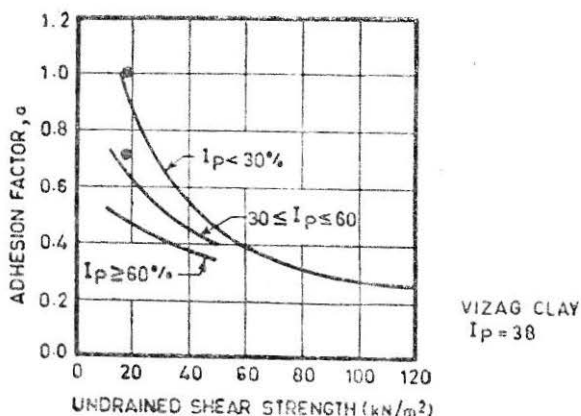


FIGURE 2 : Relationship between Adhesion Factor for Driven Piles and Undrained Shear Strength of Clay (After Flaate 1968)

driven and bored piles in soft and stiff clays and take note of the over-consolidation ratio of the soil as well. These, however, appear to be conservative and are yet to be verified for Indian soils.

In view of the above, the designer lacks clear direction on the choice of the adhesion factor. For driven piles, the adhesion factor is assumed to be close to unity for soft clays and 0.5 or less for stiff clays. For bored piles in stiff clays, it is usually taken as 0.5 or less. These are very broad guidelines which deserve to be qualified into definitive recommendations for the clays of different regions in our country. For doing so, recourse will have to be taken to field observations and to experimental studies in the field and in the laboratory.

The values of adhesion factor could also be back-calculated from the results of pull-out tests on piles. In Visakhapatnam soft clay $w_L = 68$; $w_P = 30$, $w = 78$, two pull out tests were conducted on piles of 0.66 m diameter and 16 m length. The first pile got pulled up by 78 mm at a load 850 $KN$. The average undrained shear strength was, by back-calculation, worked out to be 19.6 KN/m². The second pile got pulled out at 569 KN and the inferred undrained shear strength was, therefore, 13.7 KN/m². The undrained shear strength measured on undisturbed piston samples revealed an average of 19.7 KN/m² of undrained strength indicating that the adhesion factor for Vizag clay range between 0.7 and 1.0. The results are compared with the recommendation by Flaate (1968) in Figure 2.

I have no doubt that many more such results must be hidden in the files of the piling specialist in India who must come out to help the ISI in evolving more definitive recommendations, particularly for stiff clays where variations of $a$ could be very high. Take for example, the stiff brown clay ($w_L = 74$ per cent $w_P = 31$ per cent, $w = 26$ per cent) through which large diameter bored piles were installed for the oil jetty at Kandla port. The effective shear strength parameters for undisturbed specimens ($c' = 30$ KN/m² ; $\phi' = 30°$) were found to drop to the residual $C'_r = 0$ ; $\phi'_r = 24°$ determined by cut-plane shear box tests. A closer study of the pile installation procedure brought out that the residual shear strength parameters were more relevant to effective stress analysis of the piles for the oil jetty for which an $\alpha$ factor worked out to be 0.75. The values borrowed from the stiff clays such as the Londen Clay would never have permitted values greater than 0.5.

*Pile Point and Skin Resistances from Field Tests*

Field penetration tests and pressuremeter tests are being increasing favoured for estimating unit point and skin resistances, because the alternative of undisturbed sampling and stress path testing is often expensive and cannot be easily justified except in the case of very important structures to be founded on uniform soils. A detailed descriptions of various static penetrometers have been given by Sanglerat (1972), the commonest of which is the Dutch Cone Penetrometer. A penetrometer essentialy resembles a small diameter jacked pile. It is usually 35 mm in diameter having 60° cone at its tip, It was first introduced in Holland and Belgium and later used in other countries of the world. The vast experience now available suggests that, the cone resistance ($q_c$) is nearly equal to the unit pile point resistance ($q_o$), with due consideration of scale effect as outlined by DeBeer (1971),

Some researchers have tried to correlate the frictional resistance of a pile ($f_s$) with the cone point resistance ($q_c$) in the form.

$$f_s = \rho \, q_c \qquad \qquad ...(1)$$

The researches done at the CBRI (Mohan et. al. 1963) using 60° cone with a 10 sq. cm base area have shown that
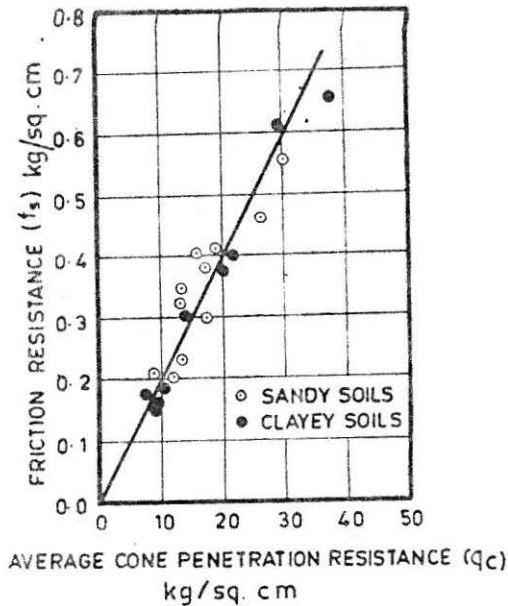
$$f_s = \frac{q_c}{50} \qquad \qquad ... (2)$$



FIGURE 3 Cone Penetration Resistance and Frictional Resistance of Piles

The above relationship was found to be valid for cone penetration values ranging between 980 and 9800 KN/m². Vesic (1967), later, highlighted the dependence of the coefficient $\rho$ on the soil type and its mechanical characteristics, particularly the angle of shearing resistance $\phi'$ and the rigidity index $I_{rr}$. For example, tests on driven piles yielded the following approximate relationships

$$f_s = q_c \, (0.11) \, (10)^{-1.3 \tan \phi'} \qquad ...(3)$$

$$f_s = \frac{3}{I_{rr}} q_c \qquad ...(4)$$

In some countries such as North America, Standard Penetration Tests are more common. Experience shows that point resistance of driven piles $q_o$ in ton/ft² can also be related to $N$ by the relationship

$$q_o = \beta N \qquad ...(5)$$

The coefficient $\beta$ is known to vary with soil type and mean stress level. Meyerhof (1956) suggests $\beta = 2$, for saturated clays and $\beta = 4$ for sands. For soils of intermediate types, the values of $\beta$ between 2 and 3 have been reported, Sutherland (1963), Crowther (1963).

For skin friction, Meyerhof (1956) suggests

$$f_s = \frac{N}{50} \qquad \qquad ...(6)$$

All the above correlations should serve as mere guidelines and should be modified to suit a particular site condition. Whenever elaborate soil investigation programme is executed on a piling job, the geotechnical engineer should attempt to refine these correlations for the Indian soils.

Recently, attempts have been made to relate the limit pressure ($p_u$) measured in a pressuremeter test with the pile point resistance ($q_o$) by means of a stress transfer factor $\lambda$ in the relationship

$$q_o = \lambda p_u \qquad \qquad ...(7)$$

The Central Building Research Institute has developed a pressuremeter in India jointly with the Associated Instrument Manufactures (India) Pvt Ltd, New Delhi which has now been released to the users. It is hoped that with its increasing use, reliable correlation coefficients would become available for different Indian soils for the benefit of the design engineer.

*Group Action of Piles*

Upheaval of the ground surface caused by driving closely spaced piles into denser or stiffer soils must not be allowed and so also the intensive over lapping of pressure bulbs for piles in clays. Hence a minimum pile spacing is necessary. If the spacing is too large, uneconomical pile caps may result. Ideally piles should be spaced so that the bearing capacity of the group is not less than the sum of the bearing capacities of the individual piles. This is, however, not always practicable because groups containing more than about nine piles may lead to spacings of more than four pile diameters, which is normally avoided as uneconomical. On the other hand, spacings less than $2.5d$ are not favoured to allow for the possible deviation in alignment of piles as well as for reasons of ground disturbance. The optimum spacing is usually selected in the range of 3 to $3.5d$ (Vesic, 1977). CP 2004 : 1972 suggests the following mimimum pile spacings

| Type of Pile | Minimum Spacing |
|---|---|
| Friction | Perimeter of the pile |
| End-bearing | Twice the least width |
| Screw Piles | $1\frac{1}{2}$ times the diameter of screw blades |

The Norwegian Code of practice on Piling, Den Norske Pelekomite (1973), gives the minimum pile spacings as given in Table 2.

TABLE 2  Minimum pile spacings given by Norwegian code of practice

| Pile length (m) | Friction Piles in sand | Friction Piles in clay | Point bearing piles |
|---|---|---|---|
| 12 | 3d | 4d | 3d |
| 12-24 | 4d | 5d | 4d |
| 24 | 5d | 6d | 5d |

Note :   1. d is the pile diameter or largest side.

2. The pile spacing is measured at pile cut-off level, unless raking piles are used, in which case the spacing is measured at an elevation 3m below pile cut-off level.

The Norwegian Code seem to favour wider spacing which may prove highly uneconomical as in the case of deep friction piles in clay, In my view, the Indian Standards should follow Vesic's recommendation of 3 to 3.5d, which may be altered only in special cases such as for compaction piles used for densifying loose deposits of sand. Once the spacing of the piles is decided, the designer attempts to link the ultimate capacity of the group $(Q_g)$ with the ultimate capacity of a single pile $(Q_s)$ times the number of piles $(n)$ in the group. The group efficiency factor depends on parameters such as soil type, size and shape of the pile group, spacing and relative length of piles, as well as the construction procedures. It is also known that the settlement of the group is normally greater than settlement of individual piles at the comparable working load.

The present state-of-the-art does not afford any scientific theory of evaluting pile group behaviour. A number of empirical efficiency equations have been suggested to provide for the group effect but none of them are reliable because of the great number and divergency of factors involved. At the present stage of knowledge, therefore, it seems preferable to consider each case individually and to evaluate bearing capacity and settlement of a group on the basis of physical properties of the soil to which the load is transmitted by the piles.

A satisfactory answer to the above problem could be found only by more of research. A few years ago, my Institute undertook a large field study with a view to predicting the pile group behaviour from the known load-displacement behaviour of a single pile. A very large number of field load tests were performed on single piles and pile groups of 2,4 and 6 piles, (diameter=15 cm; under-reamed diameter $D_u$=37.5 cm, length=3 m) spaced at 1.5, 2.0 and 2.5 $D_u$. Broadly speaking, the variation of spacing was found to have little effect on the variation of group efficiency in the pile cap free standing condition. On the other hand, for pile spacings of 1.5, 2.0 and 2.5 $D_u$, the group efficiency exhibited an increasing trend for the pile cap resting conditions.

From the load test results relationships were obtained between percentage increase and settlement (Figure 4 & Figure 5)
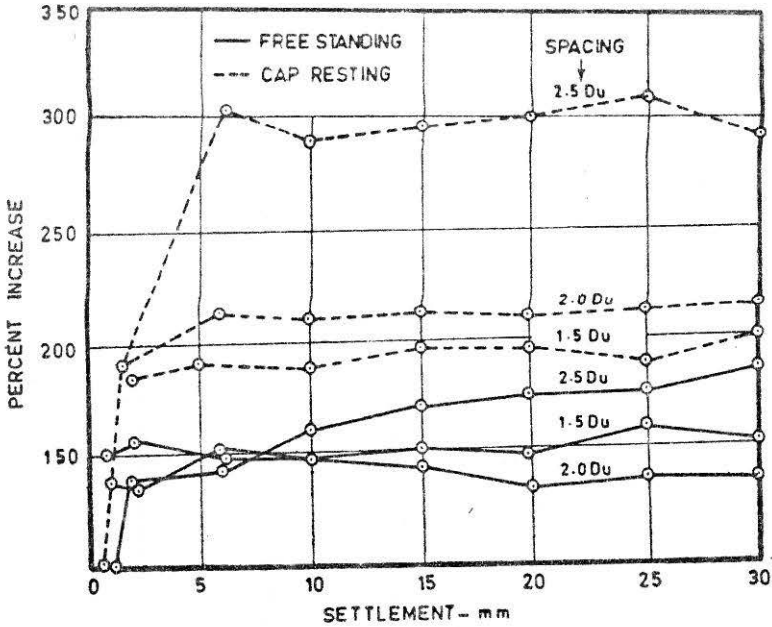
FIGURE 4   Percent Increase Versus settlement for 2 Pile Groups

The percentage increase was defined as

$$\frac{Q_{gs} - Q_{ps}}{Q_{ps}} \times 100 \qquad \qquad \ldots(8)$$

where $Q_{gs}$ = capacity of pile group corresponding to a given settlement

$Q_{ps}$ = capacity of an isolated pile corresponding to the same settlement.

These relationships are for pile groups of 2, 4 and 6 for pile cap resting and pile cap free standing conditions which enables calculations of group capacity from the known behaviour of a single pile. If the pile groups of 2, 4 and 6 are considered at spacings other than 1.5, 2.0 and 2.5$D_u$, Figure 6 representing relationship between group efficiency and pile spacing should be used for predicting group capacity. If the pile groups consist of numbers other than 2, 4 and 6 but if the spacings are the same as those studied i.e. 1.5 $D_u$, 2.0 $D_u$ and 2;5 $D_u$, the group behaviour could be predicted with reference to Figure 7 which represents curves of efficiency versus number of piles corresponding to different settlement values.

For pile groups having different number of piles at spacings other than 1.5, 2.0 and 2.5 $D_u$, linear interpolation may be done using Figure 7.

Many more similar studies are required on various other pilling system under varying soil conditions before a generally acceptable approach could be possible.
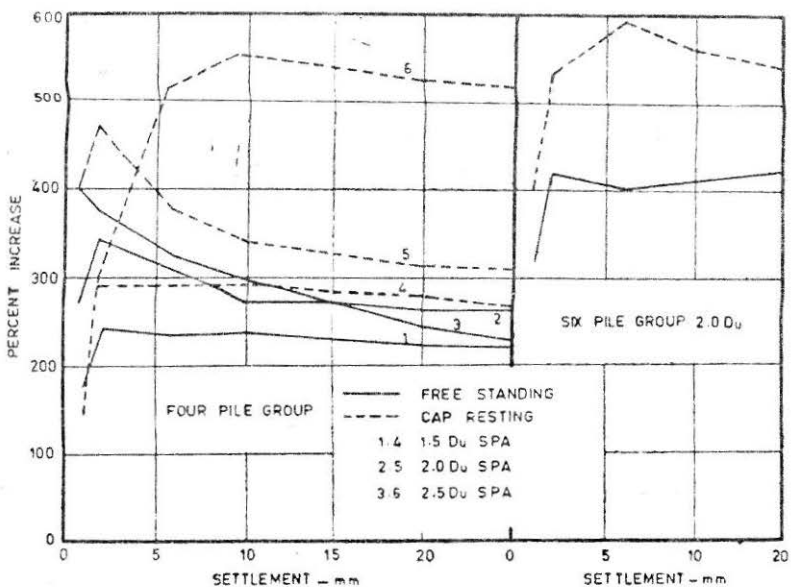
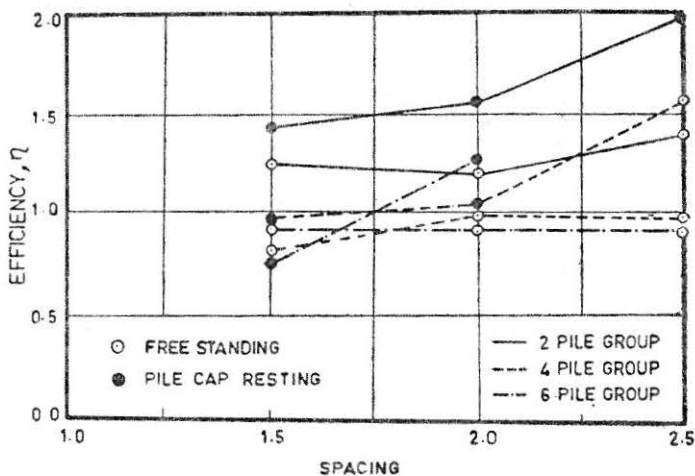FIGURE 5 Per cent increase versus settlement



FIGURE 6 Efficiency versus spacing for pile groups

*Contribution of a Pile Cap*

A pile cap adds to the axial and the lateral load carrying capacities of a pile group due to the resistance offered by the soil surrounding it. These added capacities are however, seldom utilised by the designers, in view of the potential danger of the loss of soil support to the pile cap such as by scouring around a pile-supported bridge pier. There are, however, situations in which a pile cap is well embedded in a stiff soil and the designer
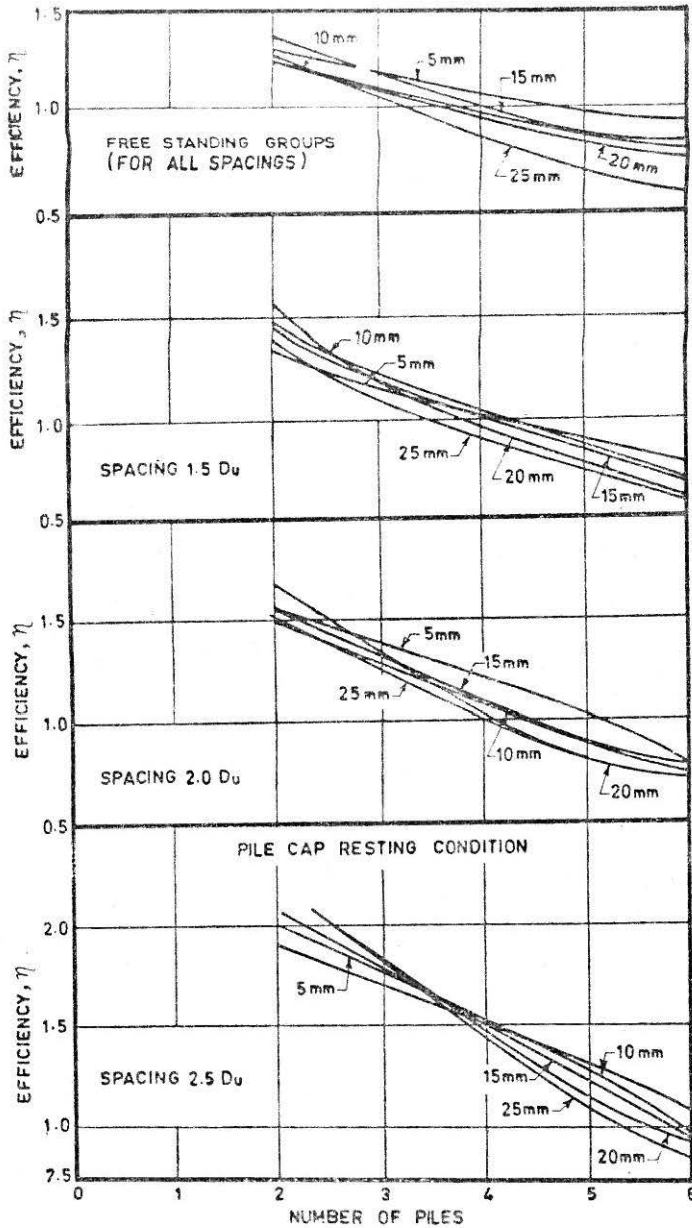
FIGURE 7  Efficiency versus number of piles

may well be justified in taking advantage of the contribution of pile cap in the interest of economy.

Kishida and Meyerhof (1965), among many others, have studied the contribution of a pile cap considering two distinct possibilities. The first was a general shear failure under the outer rim of the cap contact surface in the event of the pile group failing as an equivalent pier. The second was that if the pile spacing is large enough to make the piles act individu-

ally, the cap might contribute through the entire contact surface.

The pile cap contract increases the pile group stiffness by 5-15 per cent and reduces settlement by almost the same amount. Some investigations have been done on this subject at the CBRI. Field load tests were carried out on a single and group of two 30cm diameter, 4m long under reamed piles in a silty clay, with pile cap resting and free standing conditions. The spacing between piles was 3.75d and cap projection was 15cm beyond the piles. The test result indicated that the pile cap contributed almost equal to its bearing.

A further study was carried out in silty sand deposit on 15cm diameter 3m long single under-reamed pile (bulb diameter 37.5 cm) on 2,4,6 pile groups. It was seen that the pile group with cap in contact with the soil settled 25-50 per cent less than the group with cap free standing above the ground. A typical result is shown in Figure 8, The reduction was larger for bigger groups. Another field test on a group of 30 cm diameter, 3.6m long campaction piles with cap resting on silty sand deposit showed that the settlement corresponding to the design load 246 KN per pile was about one fourth as compared to the group with pile cap in free standing condition.

It is, therefore, seen that the cap, when in contact with ground makes a positive contribution to the load carring capacity of the piles. This is also confirmed by some work in the USSR where it has been concluded that the cap shared a load upto the extent of about fifty per cent, Arshakime and Nagorywykli (1975).

*Sequence of pile-Driving*

For a group of driven piles in loose sand, it is a matter of common sense to expect higher capacities for the pile next to the one already driven. Effect of driving sequence on the efficiency of piles in loose sand, studied by Kishida (1967) vividly brings home this point (Figure 9). This observation raises two questions : the first that should one follow the sequence of driving from centre-outwards to avoid problems of (a) tightening up of the ground (b) damage to the adjacent piles. The second, that should one move from outside-inwards, particularly in loose sandy soils to be able to take advantage of the increased group capacity? On careful consideration of both of the above points, one would likely conclude that driving sequence is not the issue to be settled regardless of the type of soil formation and the type of pile system in question. If the design is not governed by the settlement considerations, it may well be possible to take advantage of the increase in group capacity at least for piles totally embedded in loose sands (Vesic, 1977).

A particular caution seems necessary in dealing with cast-in-situ piles in soft clays. It is a situation, where, it may perhaps never pay to think of proceeding from the outer boundary of the group to its centre. Because by so doing, one would always make piles vulnerable to damage by necking or strangulation.

Seldom does the contractor of the client seem alive of this important fact. In practical problems the sequence of pile construction is often guided by the convenience of the operator dictated by consideration of speed of construction. Take for example the sequence of pile driving (Figure 10) used for the construction of a pile supported tank foundation
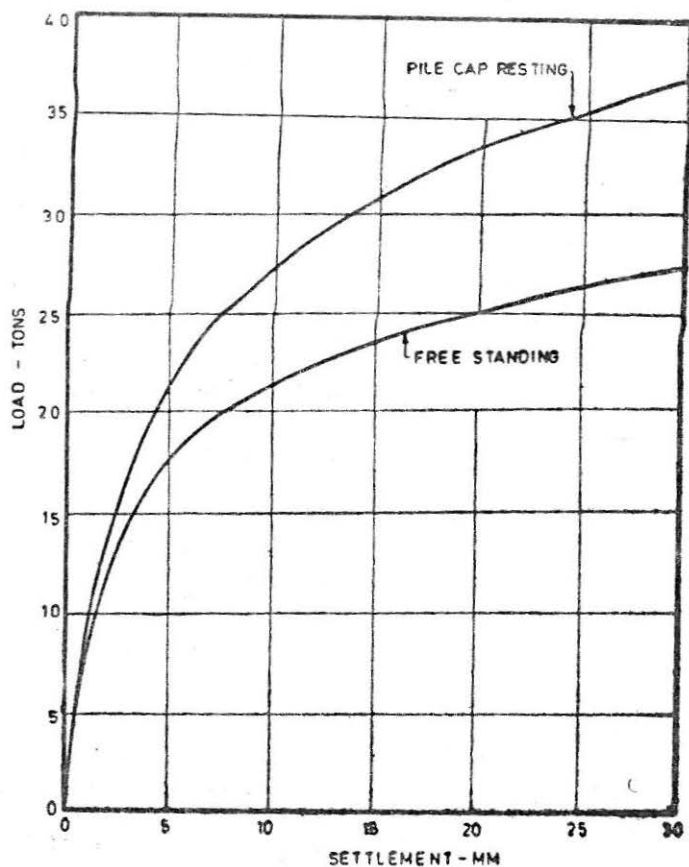
**FIGURE 8**  Load displacement characteristics of two pile group at $2\,D_u$ spacing showing pile cap contribution.
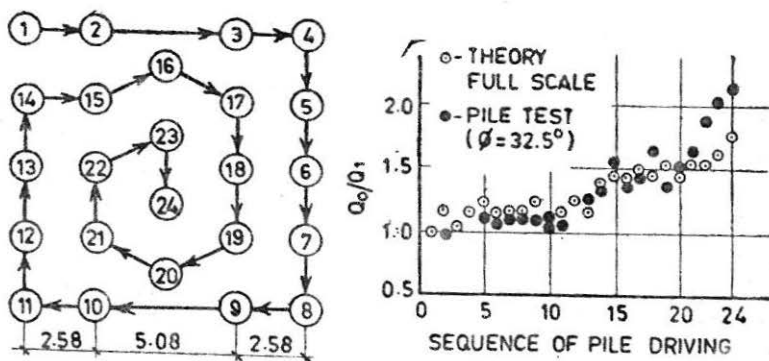


**FIGURE 9 :**  Effect of Driving Sequence on Efficiency of Piles in Loose Sand (Kishida, 1967)
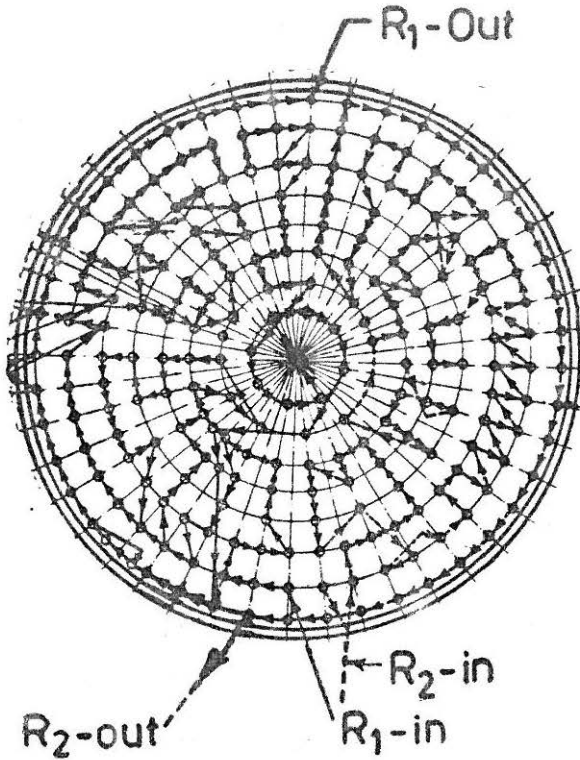
FIGURE 10 : Sequence of Piling for the Steel Tank Foundation

(Mohan et. al 1978). No definite pattern of driving seem to emerge in this case.

I, therefore, wish to re-emphasize on the importance of the sequence of pile driving which a designer must invariably recommend with his design. He must do so in consultation with the contractor and any deviations should be allowed only with the full understanding of the implications.

*Piles Subjected to Lateral Loads*

In addition to the axial capacity of a pile or a pile group, the designer often requires the knowledge of the lateraal load carrying capacity for pile-supported bridge abutments, piers, water-front-structures, transmission line towers, multi-storeyed buildings, chimneys etc. Sometimes, the designer also encounters problems in which pile foundations receive lateral loads not from the super structure, but from the surrounding soil on account of movements induced by differential loadings. Lateral loads develop bending moments and shear forces which the foundation must withstand. Besides the designer must also estimate the lateral deflection and the slope of the pile axis which influence the stability of the structure.

The pile behaviour in both the above cases chiefly depend upon the deformability of the pile, the deformability of the soil, the ultimate

resistance of the soil and the ultimate resistance of the pile material. The deformability of the pile in turn would depend on its elastic modulus, length and moment of inertia. The deformability of soil would depend on the pile diameter, pile depth and the displacement level. The ultimate resistance of the soil is governed by the undrained or drained shear strength parameters depending on the method of analysis used.

Behaviour of piles in sand and clays under lateral loads of short and long durations have already been studied in a fair detail by Vesic (1956), Matloc and Reese (1960), Broms (1964 a, b), DeBeer (1977), Jamiolkowski (1977) and others. The analysis are generally based on the theory of elasticity. It is assumed that the soil behaves identical to a series of closely spaced independent elastic springs (Winkler's assumption) and pile is treated as a beam on an elastic foundation.

Whether the pile is short and rigid, or long and flexible is yet another major consideration. For rigid piles, analytical methods have been put forward by Brinch Hansen (1960), Broms (1964 a, b), Meyerhof and Ranjan (1972) and others. Of these, the method put forward by Broms is simpler and it accounts for both rigid and flexible piles.

Despite the considerable advances in the theoretical treatment of the problem, pile load test continues to be only reliable method of assessing lateral load carrying capacity of a pile or group of piles. IS : 2911 part (iv)—1979 stipulates this. A load test is usually carried out by introducing a hydraulic jack between two piles. The load increments of about 20 per cent of the estimated safe load are provided using the hydraulic jack. The safe load, from the load displacement behaviour, is estimated as the least of the following.

1. Fifty per cent of the final load corresponding to a total displacement of 12 mm.

2. Final load at which total displacement corresponds to 5 mm.

3. Load corresponding to a specific permissible displacement.

Most of us understand what I have just said in respect of the lateral load on piles. The questions which still remain unanswered are.

1. What orders of lateral displacement and tilts a designer can permit for different types of structures? If the assigned limits of the lateral displacement or the tilt is exceeded what are the implications?

2. How the magnitude and the pattern of lateral load distribution on a pile gets altered as the soil-pile displacement takes place? What influence the method of installation has on the lateral capacity?

3. How much is the contribution of a pile cap in resisting lateral forces? If significant, as it seems, what is the rational method by which the contribution of a pile cap could be evaluated.

4. How is the lateral capacity of a single pile related to that of a pile group?

The answers to none of the above questions are simple and therefore deserve the fullest attention of a research scientist. My Institute has looked

into some of these problems particularly in relation to single and multi-bulb, under-reamed and bored compaction piles, CBRI Hand Book 1978 ; Soneja and Garg (1980). The fact that the pile cap adds considerably to the lateral load carrying capacity of a pile group was utilised by the CBRI while producing designs for the foundations of a 132 KVA transmission line towers in Dubai.

Research done at my Institute also make it possible to compare the results of load tests on 30 cm diameter, 3.5 m long single and double under-reamed piles ($D_u = 75$ cm) in silty sand ($N = 10$) with the predicted behaviour using theories due to Matloc and Reese (1960), Broms (1964) and Poulos (1971), Figure 11.

## Pile Load Tests

There is a large variation in the method of conducting the pile load test and its interpretation. The Indian Standard (IS : 2911 (Part IV)—
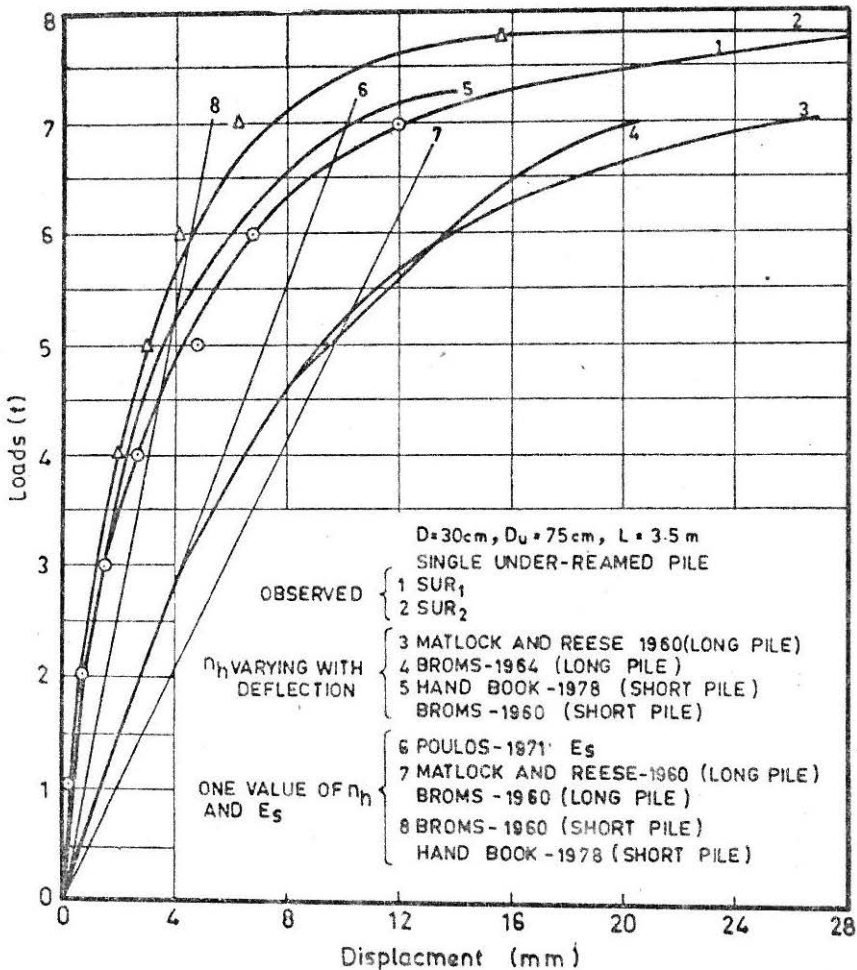


FIGURE 11 : Observed and Computed Lateral Load Versus Displacement Curves for Isolated Single Under-Reamed Pile

(1979) specifies a loading platform or a reaction frame and a hydraulic jack of adequate capacity to transfer the load to the pile head in stages. A cyclic load test is specified where it is desired to separate end bearing and side friction, which is needed for bridge foundations on piles subjected to scour. A load settlement curve is then plotted and the safe load is worked out from the criteria discussed earlier.

In a normal load test, it is rare for a pile to be tested to failure hence the effort of most piling firms is to show that the total settlement of the test pile remains within the allowable limit.

In clayey soils, the constant rate of penetration test developed by Whitaker and Cooke (1965) may be usefully adopted. The rate of penetration is kept constant (about 0.75 mm per minute) by working the jack in a suitable manner till the failure load is reached. The safe load of the pile is then worked out by using an appropriate factor of safety.

The CBRI has also developed an improved load test procedure called the equilibrium method of load test (Dinesh mohan and Jain, 1967) where instead of maintaining the load at each stage constant, the jack is operated a little beyond the desired load at each stage, further pumping of jack stopped, and settlement noted when it comes to an equilibrim.

The kentiledge required for pile load test also presents a problem specially when it is large diameter pile and the load bearing capacity exceeds 1000 KN. A simple method called the boot strap method of testing can be used in this situation. The method consists of jacking down against a reaction frame anchored to the ground at a distance of 3.5 $d$ away from the pile on either side. One such test was conducted by CBRI in Indore on an under-reamed pile in black cotton soils. (Dinesh Mohan and S.P. Gupta, 1979). The reaction frame had 4000 KN capacity. The stem diameter of the pile was 45 cm, base diameter 112.5 cm and length 7.6 m. Twelve under-reamed piles driven at 30° incline in a circle round the test pile acted as anchors and load was applied by a 5000 KN capacity jack, in stages, Figure 12. It was noted that the computed value of ultimate bearing capacity of the pile (by analysing soil properties) agreed well with those obtained by load test.

## Defects in pile and integrity testing

In the broadest sense, the ultimate aim of all on-going refinement in the technology of pile construction is to be able to construct a pile true to the design, at the maximum of speed and the minimum of expenditure. Some times, however, constructions do end up with piles of questionable integrity due to defective pile toe; necking and honey-combing along the pile shaft; use of poor quality of concrete materials and mix; and adoption of unscientific methods of construction. The problem gets compounded when a particular piling system is inappropriately thrust on a given situation, a proper construction sequence is not followed and rudimentary principles of quality control are thrown to winds.

## Defective Pile Toe and TV Scanning

A pile without a sound toe and without a proper seat into the virgin ground should prove disastrous particularly if required to act in point-bearing. Even otherwise, such piles could lead to high orders of detri-
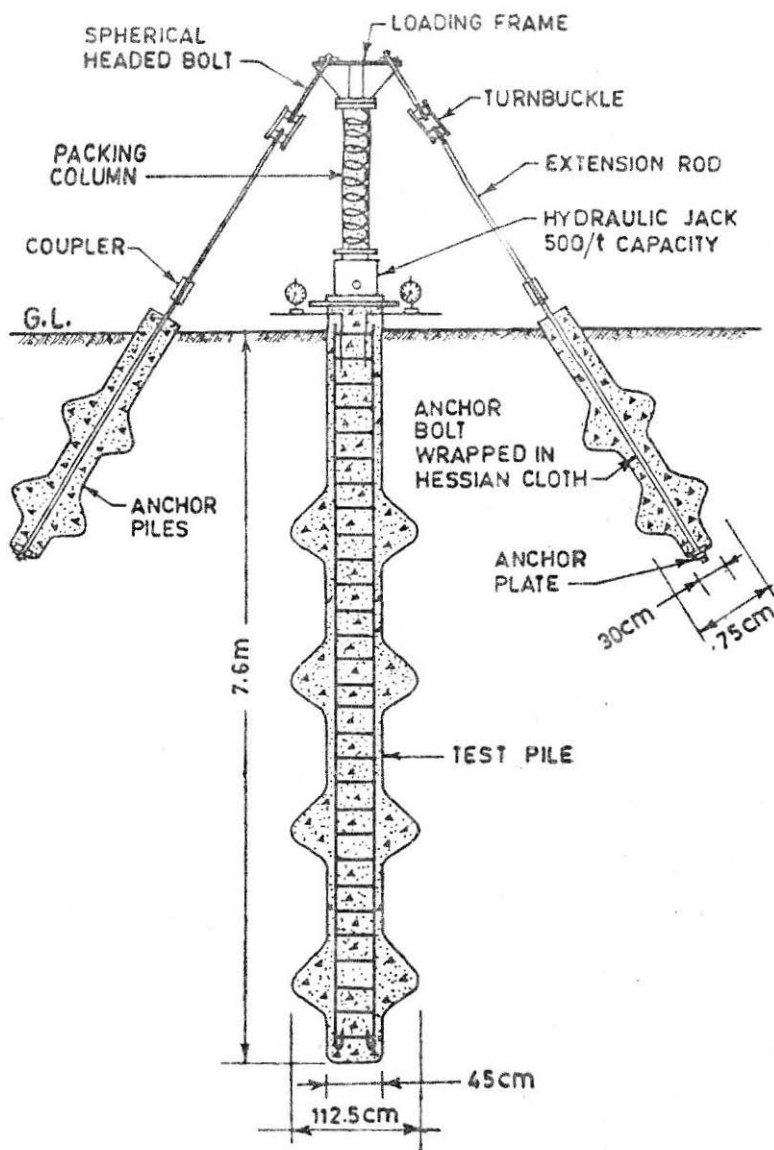
**FIGURE 12   Sectional view of the pile load test set-up**

mental settlements.   The piles of various kinds exhumed after construction
have provided direct evidences to defective pile toes.   Dastidar (1974)
referred  to  a  number  of  sites  in  different parts of India where bored piles
were found to have between 8 cm and 23 cm of bare reinforcement sticking
out at the bottom with soft and loose soils  filled into the space  between
the  underside  of  the  concrete shaft and the founding strata.   In early
seventees on a project in UK, toes of 400 bored piles, constructed  to  rest
on  a  rocky  stratum,  were examined.   Intrusions of soft soil, upto about
0.45 m thick, were found at the toes of the pile practically without exception
(Dastidar *loc. cit.*).

A classic example of the 'missing toe of a driven, cast-in-situ pile' is reported by Kulkarni (1979). Driven cast-in-situ piles 50 cm diameter and 20 m deep designed to carry 1180 KN of safe load in alluvium on the bank of river Ganga. During pile load testing, the observed load settlement curve was found to be linear upto about 150 per cent of the working load without any abnormal change in the slope of the curve. When the test load attained a value of 2360 KN, a sudden failure was noticed. The entire pile was, therefore, exhumed. On inspection, the 90 cm of the pile toe was found to be without concrete (Figure 13). One of the plausible explanation was that 20 m hydrostatic head of ground water acting at the casing bottom generated uplift force far in excess of that countered by 4 m charge of concrete placed in-side the pile casing. As a remedy, 10 m charge of concrete was, subsequently used in the rest of the construction. Dastidar (1980) has mentioned many similar bad experiences related to driven, cast-in-situ piling in and around the Calcutta region.

The other known case of ending up with a defective pile toe is the inefficient cleaning of pile hole and consequent retention of loose debris near the pile base. Piles constructed under bentonite are particularly
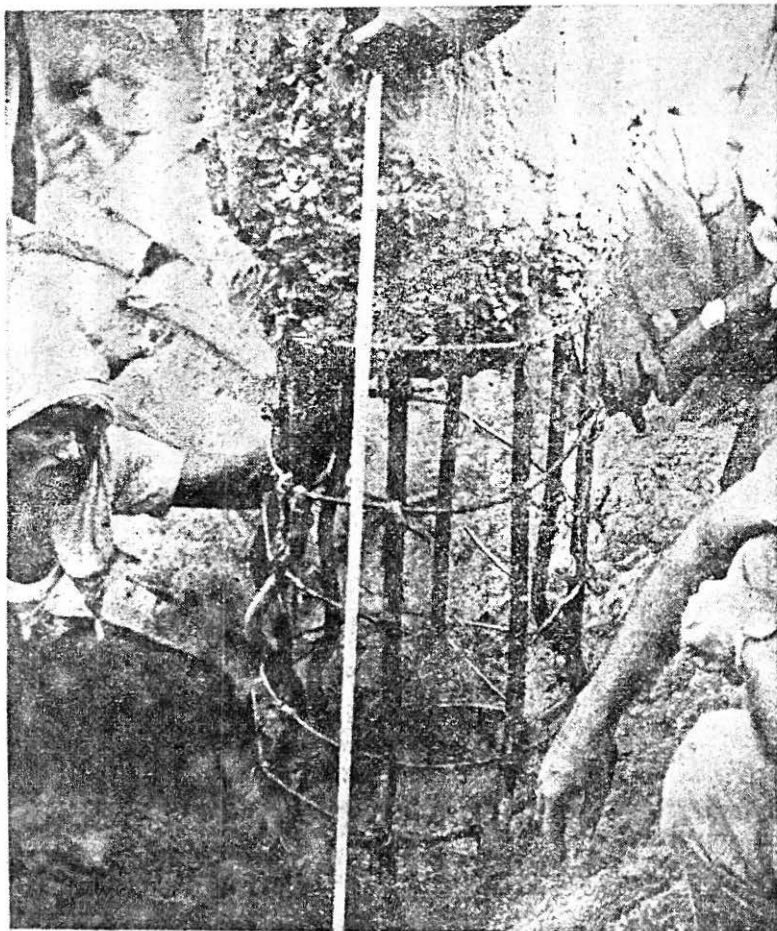


FIGURE 13   A cast-in-situ pile without toe concrete

prone to such a defect.    Earlier, Dastidar (1974) had pointed out that by using a bailer with a heavy flap about 7.6 cm above the cutting edge, it is not possible to clean the last 7.6 to 15.2 cm of the pile hole.

The hazards associated with defective pile toes have led to a great deal of caution, particulary in dealing with large diameter piles.    Some codes, for example AS 2159-1978, stipulates that

(a) each pile, if exceeding 75 cm in diameter shall be inspected to the base of the pile prior to the placement of concrete.

(b) For piles in which concrete is placed by tremie through drilling fluid, a sample of fluid shall be taken from the base of the bored hole by means of an *approved sampling device, and concrete plac-* ing shall not proceed if the density of fluid exceeds 1250 kg/m³. The sand content shall not exceed 10 per cent.

(c) Where the density of the fluid is in the range of 1150 kg/m³ to 1250 kg/m³ and the sand content is between 5 per cent and 10 per cent, concrete placement shall proceed only if it can be shown that satisfactory displacement of the drilling fluid can be achieved by the concrete mix.

The practicing geotechnical engineer in the light of the above facts and rather stringent construction requirements, seem fairly uneasy particularly when he finds that his theroetician counterpart is not fully alive of the reality in assuming higly idealised failure patterns under a pile point which is often defective. It would be desirable therefore to work out the likely devations in the pile point bearing capacities that may arise for whatever practical reasons and educate the practicising engineers of the ensuing implications.

On the other hand, recourse should be taken to the development and use of pile toe inspection devices. Holden (1980) has described a pile-socket inspection device utilising a colour TV camera enclosed in a waterproof casing, mounted on the top of a diving bell (Figure 14). Compressed air displaces the drilling fluid from the diving bell and enables the camera to clearly view, and record on a video-tape, any material on the socket base.

Future lies in the development of such or simpler devices in India as also perfecting the methods of advancing and cleaning of pile holes.

## Defective Pile Concrete and Non Destructive Testing

The defects in the cast-in-situ piles are often the consequence of faulty concreting. Use of deleterious concrete materials, improper mix design, faulty concrete placement and subsequent ground disturbances due to the construction of adjoining piles may combine to produce a poor quality of pile. It is therefore the responsibility of the field engineer to ascertain that concrete is dense, homogeneous, sufficiently plastic to flow around the reinforcement and well compacted. Poor concrete in the pile makes it vulnerable to attack of the aggressive ground environment. While concreting under drilling fluid, incorrect consistency of concerete may fail to displace the drilling fluid efficiently thereby leaving behind the intrusions or pockets of foreign materials in the pile concrete.
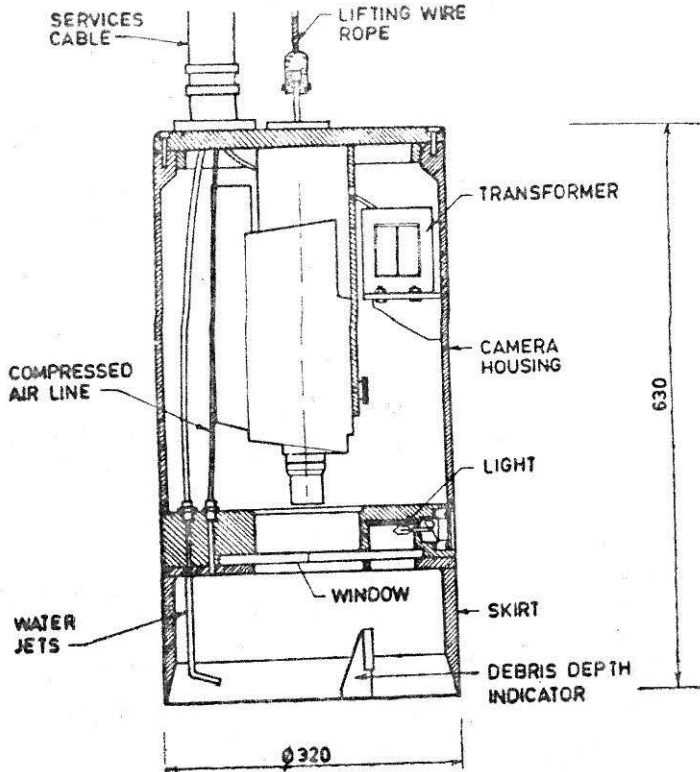
FIGURE 14   Pile Bottom inspection device (Holden, 1980)

It would, therefore, pay handsomely if a quick non-destructive method of testing the pile is devised where defects in concrete along the length of the pile could be estimated to a fair degree of accuracy. A sonic sound'ng method has been developed by Paquet (1969) and it was used to test some cast-in-situ piles in London. Preiss and Caesermann (1971) have used a radiation method. Since both these methods are cumbersome and costly, the CBRI has developed ultrasonic pulse method based on pulse reflecting technique (Rajagopalen et. al. 1975). In this method, two galvanised iron pipes of 2.5cm diameter are embedded in a pile during casting in diametrically opposite positions (Figure 15). These pipes can even form part of the pile reinforcement. Prior to testing, the pipes are filled in with water and transmitting and receiving probes are lowered in them simultaneously. An ultrasonic tester is used to observe the pulse and measure their time of transit through the concrete. It makes use of the principle that when the ultrasonic pulse trains passes through a medium, it undergoes modifications which depends on the medium. The equipment consists mainly of an electronic pulse generator, a triggering stage for activating the transmitting transducer, amplifier stage for amplifying the pulses received and a cathode ray tube for displaying the pulses. By observing the time of transit, pulse amplitude and frequency, the defective part of the piles can be detected. The method can be used for checking piles up to 75cm diameter and 20m deep.
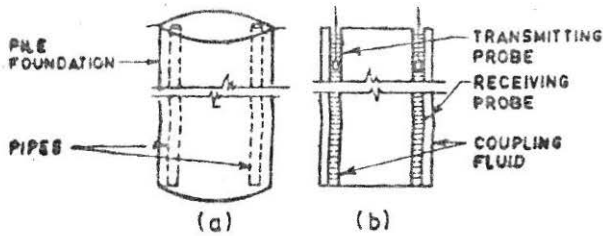
TESTING A PILE FOUNDATION
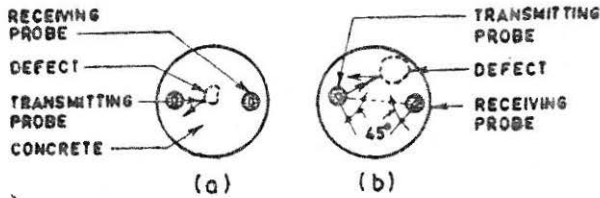


DETECTION OF DEFECTS IN DIFFERENT POSITIONS
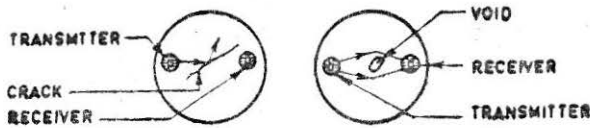


(LEFT) REFLECTION AND (RIGHT) DIFFRACTION OF PULSES



(a) 1:2:4     (b) 1:3:6     (c) 1:4:8
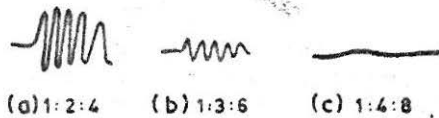
PULSES FOR DIFFERENT MIX PROPORTION OF CONC.



(a) SMALLER AGGREGATES (b) LARGER AGGREGATES

PULSES WITH DIFFERENT SIZE AGGREGATES

FIGURE 15   Pile testing by pulses

### Ground Heave, Necking and Strangulation of Piles

Any pile driving operation is invariably accompanied with volume displacement of the ground. The volume displacement has the beneficial effect of compaction, if one considers precast piles driven in soils of poor relative density. In the event of the soil not being susceptible to compaction by

pile driving, the already driven piles may get lifted-up when adjoining piles are driven. The piles, then, may not carry the design load unless driven back to their original depths. Besides additional time required and other practical difficulties, repetitive driving may lock in un-known magnitudes of residual stresses in piles, the implications of which are yet to be realised and constitutes an excellent topic of research.

Pile driving leads to build up of detrimental lateral forces in clayey soils When a driven, cast-in-situ pile is constructed in such soils the freshly cast adjacent piles always bear the risk of 'necking' or even 'strangulation' due to the associated ground movements. If the pile concrete is not fully set and if pile section is unreinforced, the squeezing ground may even shear off the pile section. For a site in Haldia, Dastidar (1980) has reported build up of lateral ground pressures to the tune of 2 kg/cm² which led to ground displacements and consequent oozing out of the water from the top of some driven, cast-in-situ piles. Butterfilled and Johnston (1973) have also shown that radially stress develops symmetrically on pile soil interface varying in magnitude from $4 C_u$ (undrained shear strength) to $8 C_u$ ; the upper limit occurring near the pile head during extraction.

Necking may also be the result of squeezing ground behind the pile casing as it is withdrawn while concreting. The possibility of such an occurrence increases if the concrete is harsh and of low slump. When there are delays in pulling out the casing, the suction effect at the casing bottom my compound the problem, particularly if withdrawal of casing is sudden and uncontrolled. There are a number of publications which draw attention to failures on account of necking (Snow, 1947 ; Hobbs, 1957). My Institute has recently documented a failure case record where cast-in-situ piles had shown considerable evidence of deshaping, (Figure 16). Some of the Franki piles exposed after construction were also found to be deshaped. (Figure 17).
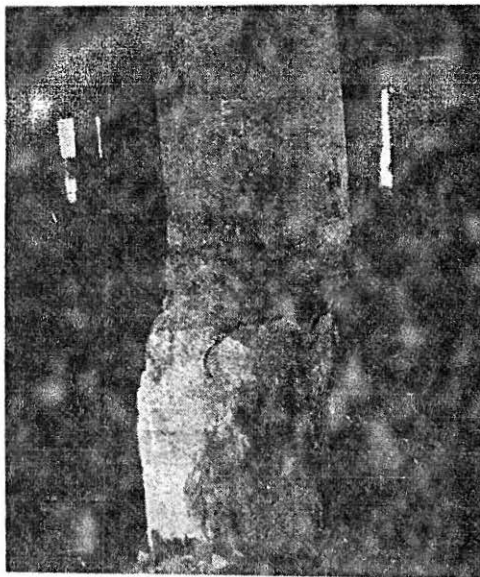


FIGURE 16  Deshaped cast-in-situ pile
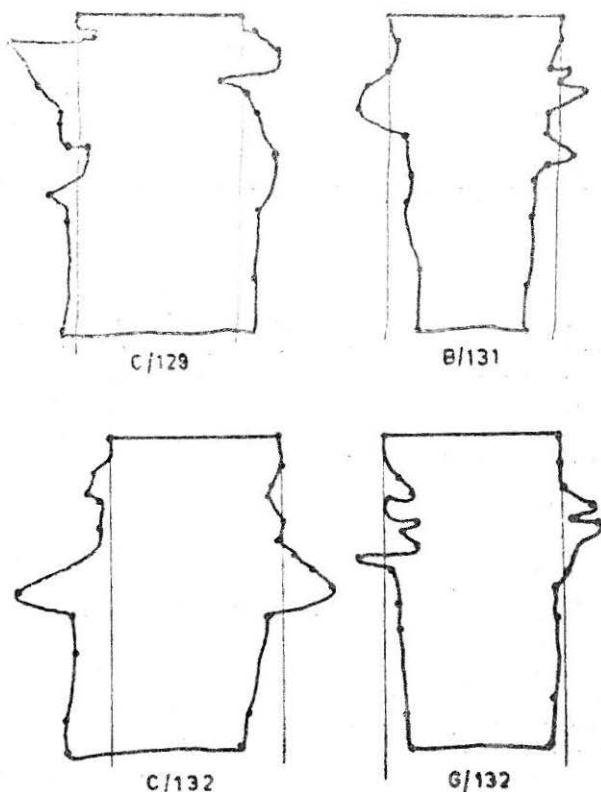
C/129   B/131

C/132   G/132

FIGURE 17  Deformed shape of pile

To get over part of the risk, pilling codes of a few countries recommend that during the setting period, no casing should be permitted to be driven or sunk nearer than six diameters to a pile already installed unless it is atleast 8 hours old. One of the Code (AS 2159-1978, clause 4.4.3) explicitly states that piles which are installed without a permanent casing shall be reinforced for their full length with steel bars such that,

(a) the cross-sectional area of longitudinal reinforcement shall be not less than 0.50 per cent of the cross-sectional area of the pile.

(b) the longitudinal bars shall be tied into a prefabricated cage with steel ties or spiral binding of not less than 5mm diameter and 150 mm minimum pitch, prior to installation, and shall protrude a minimum of 60mm above the top of the piles when installed,

(c) the minimum cover to reinforcing steel shall be 50mm, and

(d) joints in the longitudinal bars shall be permitted and shall be such that the strength of the bar is effective across the joint and that there is no relative displacement of the reinforcement duriug subsequent operations in the construction of the pile.

The above points have been discussed at the various Foundation Engineering Sectional Committee meetings of the Indian Standards Institution.

It has been finally agreed that all piles will now have minimum of 0.4 per cent of reinforcement to the full depth. However, none of the Indian Piling Firms seem ready to accept the restriction by way of the minimum distance between the piles or minimum waiting periods of eight hours to allow setting of pile concrete, in view of their difficulty of moving rigs backward and forward as also for the reasons of added cost on account of the restriction. Sooner we find an acceptable solution to this problem, surer the geotechnical engineer would be about the quality of construction.

## Pile Coring

Coring is the traditional method to check a suspect pile, by drilling using a diamond drill. Collection of cores is expensive and time consuming process but provides a direct evidence of the concrete quality. A much faster way to drill hole would be to use percussion equipment and then to examine the inside of the (50mm-100mm) hole physically by lowering a televison camera and watching the transmission on a screen.

Panthanky (1980) has reported an interesting application of the coring technique in detecting defect in Ziessel driven, cast-in-situ piles for the Central Labour Institute at Sion. It is reported that within a few months of construction, the structure started settling and cracking, and some of the column foundations settled by as much as 30 cm. This baffled the engineers particularly because during installation, requisite number of piles had been tested. At that time, in an adjoining plot, another specialist firm was installing bored piles which failed on load testing. Attempts to take continuous cores from the concrete failed because the concrete had not set. On a detailed soil investigation conducted jointly by the CPWD and the HCC, it was found that the soil and water contained a very high percentage of sulphates.

## Vibration Testing

Vibration testing of the piles has been developed by the Central Experimental de Recherches et d'Etudes du Batiment et des Travaux Publics (CEBTP) of France and is described by Gardner and Moses (1973), Davis and Dunn (1974). It's applications have been reported in UK.

The principle behind the method is that a generater supplies a sinusoidal current of frequency $f$ which can be varied from 0 to 1000 Hz. An electro dynamic motor of Mass $M$ installed at the pile head is operated by this current. The vertical vibration of the motor exerts a force $(F)$ on the pile head given by

$$E = F_o \sin wt = M \gamma_o \qquad \qquad \ldots(9)$$

where $\gamma_o$ represents the acceleration given by $\gamma_o \sin wt$. $\gamma_s$ is held constant as the signal from the motor is fed into a regulator. Since the head of the pile vibrates at the same frequency as the motor, the force $F_o$ can be determined by measuring the amplitude of $F$ at a known frequency $f$. If the instantaneous velocity of the pile head is measured continuously, for any frequency, the velocity constant $V_o$ can be found using

$$V = V_o \sin wt + K \qquad \qquad \ldots(10)$$

and $\dfrac{V_o}{F_o}$ is recorded as the modulus of mechanical admittance.

During the test $20 \leqslant f \leqslant 1000 \ H_z$ and an automatic plotter records $\dfrac{V_o}{F_o}$ versus $f$. From the shape of the resulting curve follows

(a) The pile length, or depth to first major discontinuity

(b) The weighted average pile cross-section

(c) The mass of the pile

(d) The damping effect of the soil surrounding the pile

(e) The apparent stiffness of the pile.

The above method does not require any special provision to be made in the pile at the time of casting. About 30 piles can be tested per day and testing is no hindrance to the progress of work.

In view of extensive piling work being done in the country, there it a very good case for trying out the vibration testing equipment.

Among all the various methods of integrity testing, some of which are outlined above, the pile load test remains the most popular, despite several disadvantages (Davis and Dunn, 1974).

### Negative Drag on Piles

In India, many foundation failures or their unsatisfactory performances are often explained in terms of either unexpected or under-expected, added axial loads on the piles on account of the negative drag. The designer faces a dilemma with the widely varying estimates of negative drag for the same situation depending on the method of calculation he follows.

In a classical situation, negative drag develops on a point bearing pile when the ground in which it is installed settle relative to the pile. Such settlement may take place due to (a) the under-consolidated nature of the foundation soil (b) remoulding of the soil that occurs during pile driving (c) a freshly placed fill around a pile foundation and (d) sustained ground water lowering. The impact of negative drag is reflected in that it increases the axial load in piles and at the same time lowers the over-burden pressure along the pile shnft and the pile tip. Consequently, the positive shaft friction as well as the point bearing component of the pile capacity drop, if the lower part of the pile happens to be in a bed of sand.

The additional loading due to negative drag may be so large as to cause over-stress of the pile material or may lead to unacceptable settlements, or even failure, in the underlying supporting soil. The load transferred to the pile depends on the material of which the pile is made, the type of soil into which it is installed, the manner of installation and the amount and rate of relative movement between the soil and the pile. It appears that relative movement of the order of 10 mm is all that is necessary for full negative drag to mobilize.

It is an important unknown for the designer to determine the depth of the soft clay strata that would contribute to the negative drag. He therefore attempts to find the depth at which the relative displacement between

the soil and the pile attains a minimum value necessary to develop full negative drag. The depth at which the relative settlement between the soil and pile becomes zero, the so called neutral condition exists such that the positive skin friction must act below the neutral point (Figure 18).

The fallacy of considering negative drag for the full depth of the pile can be demonstrated by the following example : Assume the coefficient of volume compressibility $(m_v)$ = 100 cm²/kg ; consolidation pressure $(\sigma_v')$ = 1 kg/cm² and the settlement of the pile tip $(S)$ = 2 cm.

Upon full consolidation, the thickness of soft clay $(H_1)$ which would produce settlement equal to that of the pile tip (2 cm) is given by :

$$H_1 = \frac{S}{\sigma_v' \cdot m_v} = \frac{100 \times 2}{1} = 2 \text{ m.} \qquad \ldots(11)$$

Assume further that a minimum of 1 cm of relative displacement between the soil and the pile is needed to develop the negative drag. This relative movement would occur if $H_2 = (100 \times 1)/(1) = 1$ metre of additional depth of clay is considered. In other words, only top 3 m of the clay deposit contributes to the negative drag and not the whole depth of the compressible soil that surrounds the pile.

Results of measurements of negative skin friction of piles have been presented by Gant, Stephens and Moulton (1958), Fellenius and Broms (1969), Endo Minau, Kawaski and Shibata (1969), Walkar and Darval (1973).

Theoretical expressions for the design of pile foundations taking negative skin friction into considerations have been developed by Buisson
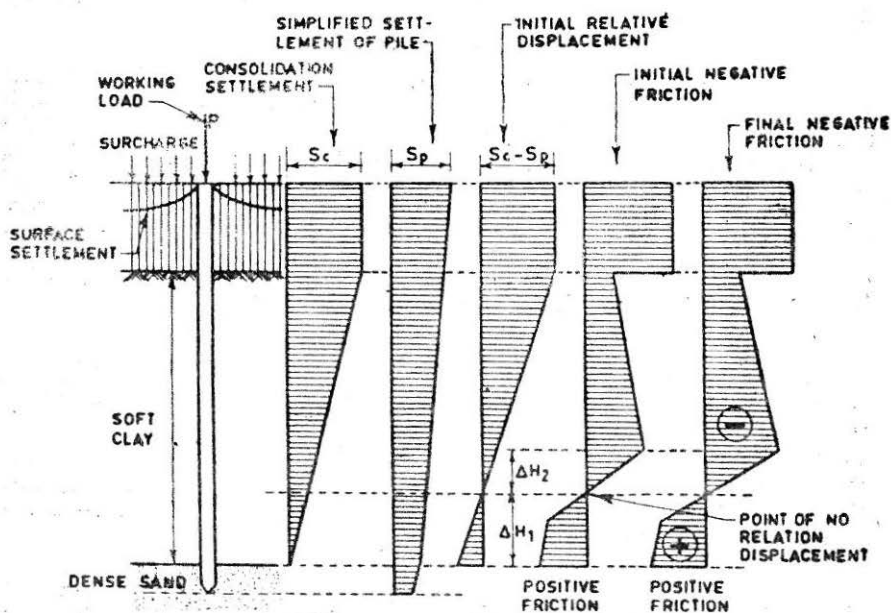


FIGURE 18 : Negative Drag on a Point Bearing Pile (Moretto 1971)

Ahu Habib (1960), Zeevaert (1960), Brinch Hansen (1968), Poulos and Mattes (1969) and Sawaguchi (1971).

Negative drag can be reduced by (a) using precast piles with shafts of smaller cross-sectional area compared to the points (Plantema and Nolet 1957) (b) by driving piles inside a casing with a space between the pile and casing filled with a viscous material and casing withdrawn (Golder and Willeumier, 1964) (c) by coating the piles with bitumen.

Without going into the details of the state-of-the-art on the above aspects, I would like to focus attention to some of the fundamental issues which deserve the attention of the researchers.

1. Designers take the piles deeper as the probability of the magnitude of negative drag becomes higher, without appreciating that their action, in itself, would generate higher negative drag by making foundation truly point bearing. On the other hand, if the settlement sustaining capacity of the structure is exploited and it could be allowed to settle with the soil without permitting detrimental settlements, the problem of negative drag could be minimised or even eliminated, in some cases.

2. It is often feared that pile driving in soft clays (a) remoulds the clayey strata in the zone of thickness equal to one diameter (b) generates lateral pressures in the foundation soil and (c) increases hydrostatic pressures around the pile. No serious studies on these topics have been made in this country. Without such studies those who 'know' would always be highly conservative in their designs and those who know not, that they don't know, would invariably end up with problematic designs.

3. The influence of the pile diameter and the pile cap, on the negative drag is yet to be fully understood. The general impression that the relative movements of the order of 10 mm between the soil and the pile is all that is necessary indicate that the problem is identical if pile is as small as 20 cm diameter or as big as 200 cm and whether pile cap stands free or rests on the ground.

4. If a pile having an ultimate capacity ($Q_u$) is subjected to 'negative drag' ($Q_n$) the factor of safety ($F$) is, by some, calculated using the following relationship.

$$F = \frac{Q_u}{Q_a + Q_n} \qquad \qquad ...(12a)$$

or

$$Q_u = FQ_a + FQ_n \qquad \qquad ...(12b)$$

In which $Q_a$ stands for the axial load due to structure. If $Q_u = 400$ ton, $Q_a = 100$ ton, and $Q_u = 100$ ton, $F$ will work out to be 2.

According to another school of thought, however, if negative drag is already conservatively determined, it need not once again be multiplied by the factor of safety in Equation 12.

Thus,

$$Q_u = FQ_a + Q_n \qquad \qquad ...(13a)$$

or

$$F = \frac{Q_u - Q_n}{Q_a} \qquad \qquad ...(13b)$$

If Equation 13 is used, $F$ will work out to be 3 as against 2 calculated earlier.

Designers continue to use approach 1, it being conservative. However, much depends upon the meaning a designer attaches to the so calculated factor of safety. Terzaghi and Peck's (1967) approach, which keeps away from the above ambliguity, seems more rational. It states that if $Q_q$ is the axial load per pile driven through soft clay into a sand stratum the pile tip will ultimately receive a load $Q_t = Q_a + Q_n$. If this load is greater than the point resistance of the pile, the settlement of the foundation will be excessive, regardless of what ultimate capacity a load test may indicate. If the load on the pile does not exceed two thirds of the point resistance, the settlement of the foundation will be relatively unimportant. This approach further eliminates the ambiguity of deciding the value of $Q_u$ and would reduce the problem to estimating the point resistance of the pile corresponding to the negative drag situation.

My Institute has launched an ambitious programme of evaluating magnitude and distribution of negative drag on a fully instrumented, 66 cm diameter and 28 m long, bored pile in soft clay of Visakhapatnam. The pile rests on a hard stratum. Accelerated consolidation of the soft clay with respect to the pile is being achieved by means of sand drains and pre-loading. The magnitude and distribution of negative drag is being monitored at each stage of loading by means of 20 load cells embedded in the pile at intervals of 2.5 to 4 m. Dissipation of pore water pressure and settlements of the clay stratum at different elevations are also being monitored by means of piezometers and magnetic settlement gauges developed at the CBRI. This full scale study has been taken up after field trials on a pilot pile 43 cm in diameter and 5 m long instrumented with load cells. The soft clay area around the pile was loaded to 30 KN/m² in equal steps and subsequently unloaded in similar steps. Magnitudes of load at three locations along the pile shaft were monitored with time. The final reading on complete unloading indicated a reversal in trend set up at initial stage of loading. The drag was registered more towards the pile toe. Piezometers installed in the loading area have responded to the excess pore pressure due to loading and have registered its dissipation with time.

The pile was also load tested in compression. The following conclusions were drawn from the study.

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 $\alpha$ and $\beta$ increase with increase in preload. (Mohan et al. 1981)

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

## Requirements of Cement and steel in Concrete Piles

Unsuitable concrete mix and lack of adequate reinforcement has many a time resulted in honeycombing, necking and shearing of the piles, resulting in reducing their load carrying capacity to a considerable extent. The concrete mix is therefore to be designed to suit the type of the pile and manner of concreting. It is evident that fluidity of concrete placed under water or through a tremie has to be different from concrete placed in a dry borehole. The British Federation of Piling have recommended the specification for concrete as given in Table 3

TABLE 3    Specification for Concrete

| Piling Mix | Slump (mm) | Manner of use |
|---|---|---|
| A | 125 | Poured into water-free bore. Widely spaced reinforcement leaving ample room for free movement of concrete between bars. |
| B | 150 | Same as above, but reinforcement not spaced widely enough to allow free movement of concrete between bars. |
| C | 175 | Concrete placed by tremie under water or bentonite slurry. |

The concrete should flow easily without any seggregation and should be self compacting. My Institute has examined the question of minimum quantity of cement for underwater concreting (Bhandari et.al. 1978) and concluded that:

(a) For small diameter bored piles and thin cut-off walls constructed to relatively small depths of 10 to 15 m a cement content of about 350 kg per m³ of concrete is sufficient for tremie concreting. For large diameter deep-bored piles or thick diaphragm walls and other substructures at deeper depths the present minimum cement content of 400 kg per m³ of concrete is justified till more data become available.

(b) Where a higher strength of concrete is desired, a richer concrete mix with greater cement content may be designed.

It is hoped that work will be done to obtain a greater number of cores from underwater and from bentonite displaced concretes and their strengths compared with laboratory samples. Such an exercise will give the necessary confidence which cannot be obtained by any other means. Until such time, there would appear to be no alternative but to err on the safe side.

There is also some controversy about the function of reinforcement in the lower part of the pile. Most firms do not provide reinforcement beyond 8m from the top, whilst some stipulate a minimum of 0.8 per cent steel for the pile length. The IS code calls for 0.4 per cent only. It is now being felt that it is worthwhile to reinforce the pile all along its length since reinforcement provides resistance to shearing forces generated due to adjacent pile driving and it is also a barrier against necking or general reduction of pile

driving and it is also a barrier against necking or general reduction of pile diameter in difficult ground conditions.

## Some Special Piling Systems

### Granular Piles

Granular piles, as the name implies, are made of stone aggregates and sand, compacted in long cylindrical holes. They have been successfully used either singly or in small groups, to support isolated footings, large rafts or embankments for improving strength and deformation characteristic of weak soils. Thorburn and MacVicar (1968); Baumann and Bauer (1974); Rathgeb and Kutzner (1975).

Granular piles or stone columns were well known in France in 1830s. They were used first for heavy foundations of Iron works at the Artillery Arsenal in Bayonne. The piles were about 20 cms in diameter and 2m in length.

Today, the granular piles may be made up to 50-150 cms in diameter and 15m depth for a design load of 300 KN. Granular piles also improve ground by serving as drains for dissipation of excess porewater pressures. In addition to improving the deformation behaviour under the static vertical load, the improved performance of the combined mass of the weak soil-granular pile system under seismic loading has also been discussed in detail by Engelhardt and Golding (1975)

Large scale field studies have been conducted at the CBRI to study the stress-deformation behaviour of soil-granular pile systems in a soil deposit of SM-ML group, Rao and Bhandari (1979, 1980); Rao and Sharma (1980). Granular piles, 15cm diameter and 3 m long were cast using 18-20mm stone aggregate and coarse sand compacted with the impact of 1.25 KN internal hammer in layers of 30cm thickness. While casting the groups of 2,3 and 4, the granular pile spacing was maintained at 3 diameters. Stress behaviour of plain and skirted granular piles and then groups were studied. ground improvement, in terms of improved bearing capacity was for to be 110 per cent for single skirted granular piles, 280 per cent for groups of three and 375 per cent for group of 4. Without 'skirting' the corresponding figures were 60 per cent, 170 per cent and 280 per cent. The above comparisons have been made with respect to the bearing capacity of virgin ground.

Load test results also reveal that the granular piles, whether plain or skirted, single or in a group, mobilise their ultimate carrying capacity corresponding to a deformation ranging between 19 per cent and 28 per cent of the pile diameter.

My Institute advised on strengthening of the foundations of a 50,000KN capacity, 24m diameter, molasses tank for a sugar factory. Construction of 225 piles of 30cm diamster spaced at 3.5 diameters was executed under the technical surveillance of the Institute. Load tests on groups of 2 and 4 granular piles further confirmed the validity of design assumptions.

The process has been licensed through the NRDC.

### Pedestal Piles

Pedestal piles have been introduced by my Institute as an economical substitute of under-reamed piles for light structures and buildings. The pedes-

tal pile consists of precast reinforced concrete stem of 10 cm × 10 cm section with 30cm diameter and high concrete pedestal cast in equal diameter auger holes. Field tests carried on these piles in loose saturated sand have revealed that the piles can take about 22 KN in compression and 8 KN in uplift. Studies, extended to black cotton soil revealed safe loads in compression and uplift of about 35 KN and 17.5 KN respectively for a 3 m long pile. A few piles exhumed after testing provided testimony to the good quality of concrete.

*Bored Precast Piles*

Bored Precast Piles, as the name implies, refer to precast piles installed in pre-bored holes. Piles of this kind combine the advantages of bored and precast piles ; they can be treated against agressive environmental conditions and are not subjected to driving stresses. The first application of such piling system is reported by Punja and Rao (1976) for Mangalore Fertilizer Complex. Their development is reported to be as a result of research done by Cemindia Company Limited, Ketkar (1980). The method of construction described by Punja and Rao (1976) suggest that the pre-boring for receiving the precast piles can be done by any of the standard methods. A cased pile hole is usually advanced by percussion method until the founding stratum is reached. The stratum is visually examined and tested by means of a chisel and a 10cm deep socket is provided. On completion of boring, the precast pile is lifted by crane and lowered into the pile hole. Just before resting the pile in the hole, it is thoroughly cleaned by flushing with water under pressure. Immediately thereafter a cement sand grout $(1 : 1\frac{1}{2})$ is pumped to the bottom of the borehole through a central hole in the pile. The grout rises around the pile displacing the ground water. On completion of grouting the temporary casing is extracted leaving the pile in place and grout around it.

The main advantages of the above system of pilling are that suitable protoective coating can be applied to prevent sulphate attack, the quality of concrete in pile could be better controlled, required founding depth could be ensured and the pile is not subjected to driving stresses.

The efficacy of the borehole cleaning procedure and the evidence with respect to the quality of the pile toe are two aspects on which more information is required for developing a higher level of confidence in this promising piling system.

*Element Piles*

AFCONS have reported a number of jobs in which element piles were constructed for transferring heavy loads of multi-storeyed buildings and other industrial structures. For example, for the sky scraper at Bombay Central, 23 element piles with maximum capacity of the order of 6500 KN and 16 large diameter piles were executed which resulted in substantial savings in quantities of pile caps and time. The capacities of these piles, which were cast in different shapes, are reported to range from 4500 KN to 19000 KN. Their use is favoured in situations where the horizontal loads and moments on piles are high.

We should have some publications on analysis and behaviour of element piles so as to have a better idea of the potentiality of such piles.

### Bored Compaction Piles

CBRI introduced bored compaction piles which are ideally suited in loose sandy and silty soils. For their construction, a bore hole is first advanced with an auger and its bottom under-reamed with a hand operated underreaming tool as for normal under-reamed piles. The concrete is then poured in and the reinforcement cage (welded to a conical steel shoe at the bottom) is driven into the green concrete with a steel pile core. During the driving of the cage through the steel tube, the concrete is compacted and in turn compacts the surrounding soil also upto a certain distance. The steel tube is withdrown gradually and more concrete is poured through it simultaneousely to fill in the void left by the withdrawal of the tube. It has been found that load carrying capacity of the pile improves 75 to 100 per cent by this technique. An Indian Standard Code of Practice has been brought out both on plain under-reamed and compacted under-reamed piles. Recently, I had the privilege to edit a book on these piles which was complied by my colleagues at the CBRI and which describes in detail all aspects of piling, e.g., planning, design, construction and testing (CBRI Hand Book, 1978).

### Micro Piles

A micro pile is a small diameter pile constructed by forming a bore hole and sealing into the same a steel tube with high strength grout injected under pressure through the tube itself. The grout injection forms bulbs along the shaft of the pile, through which the pile derives added frictional resistance. The method of execution, a patented Rodio process, has been developed and perfected to give a high quality pile with a small rig.

A micro pile is constructed by a small rotary-cum-percussion rig which can work in low headroom and small working space, using bentonite mud to retain the sides of the hole. Due to the specialised installation process, the vibrations and disturbances to the surrounding soil and structures are minimised. All the above features make the micro pile ideally suited for piling in restricted area and/or low headroom and close to the existing structures such as in case of under-pinning. The piling can also be used for dock floors, where tensile forces are to be carried (AFCONS Catalogue).

Feed back studies and case records describing application of micro piling would be welcome.

### Lime Piles

Lime piles were first developed in Sweden. In these, a mechanical auger makes a borehole in the ground and simultaneously dry unslaked lime is introduced through the centre drill rod mixing it thoroughly with the soil as the auger advances forward. The time improves the load carrying capacity of the column through ion exchange and makes it a lime piles. Lime piles having a diameter of 50 cms and a maximum length of 10 m have been constructed. These piles are capable of carrying relatively high loads and can be used to support light structures. Benefits due to sand drain action also been claimed (Broms and Boman, 1977).

## An overview of the piling problems

Piling research and associated design and development work involve the following areas of activity.

(a) Site investigation, Statistical Co-relations and choice of design parameters

(b) Model, Semi-field and field studies

(c) Pile Foundation Design

(d) Development of tools, equipment and piling technology

(e) Pile construction and implications of associated changes in the quality of ground

(f) Field testing

(g) Performance evaluation and Instrumentation

(h) Feed back and failure studies; Documention: Indentification of the problems of research.

Of these, I have attempted to discuss some of the more important aspects of pile design, construction and testing.

A designer is often concerned with calculations of structural loads; evaluation of soil reports, pile load test results and driving records for choice of design parameters; choice of safety factors in relation to permissible settlements; selection of piling system; and estimation of pile capacities with full appreciation of the group effect, pile cap contribution, settlement and possibilities of negative drag. Thereafter, he is required to work out structural designs and lay down specifications for concrete and steel. Finally, he recommends the number and locations of pile load tests. The pile test results provide data which might subsequently call for revision of designs.

The construction engineer then interacts to select proper piling equipment and materials of construction. It is his responsibility to ensure conscientious implementation of the design by minimising positional deviations to the extent practicable, by exercising strict control on the quality of construction, by maintaining proper construction sequence and by providing to the designer a feed back on any deviations in the soil conditions or construction details. He must exercise periodic checks on the quality of concrete mix and concreting procedure for cast-in-situ piles. For driven piles, he must ensure proper casting, adequacy of driving equipment, efficient protective coatings, splicing and driving.

Several innovations have taken place in tools, equipments, plants and construction technology because of the growth concern for speed and economy. The pace of development has hastened to meet the construction situations never encountered before. Greater focus is, however, required to develop inspection devices to check the accuracy of pile holes, quality of soil strata at the pile base and the integrity of piles as discussed earlier in this paper. Economical and innovative procedures should be dveloped for exhumation of piles in special cases.

Last but not the least, I would like to emphasize on the need for more and more of instrumentation and feed back studies and their documentation. Instrumentation is necessary to know the proportions of load transfer through shaft and point bearing, the sharing of load between piles in a group, the contribution of pile caps and the overall settlement behaviour of the piled foundation. During installation, instrumentation helps in estimating pore pressures due to construction, magnitudes of ground displacements and vibration levels.

It is only through instrumentation that we can hope to have a clear idea about the magnitudes and distributions of negative drag on piles and its dependence on pile diameter, soil characteristics, surcharge and time. I have earlier quoted the example of such a study currently being conducted by the CBRI.

Load transfer through a group of piles particularly when pile cap is directly supported on the ground or laid a little below it, is a matter of profound interest to the designer. The problem becomes more complicated when the advantage is sought to be derived from the rigidity of the pile cap. Instrumentation can help to provide data on significance of rigidity which could give us better insight and confidence in our designs.

## Concluding Remarks

In this talk today, I have tried to enumerate some of the important problems faced in the design, construction and testing of pile foundations and have tried to give the latest knowledge on various aspects of these problems. There is a good deal of empiricism in much of the design and it is only with extensive feed-back data from the field, case studies and instrumentation in pile foundations that we would be able to further rationalise the design and achieve innovations and economy in construction. Some of the new types and concepts in pile foundations are also noteworthy and I hope in India too we will innovate and use some of these piling systems with a view to introduce further economy, speed and efficiency in our pile foundations.

## Acknowledgements

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## Vote of Thanks

Mr. V.M. Sharma, Secretary of the Indian Geotechnical Society proposed the Vote of Thanks.

'On behalf of the Indian Geotechnical Society, it is my privilege to propose a Vote of Thanks to Professor Dinesh Mohan for delivering the Third Annual Lecture of the Society. Piles constitute one of the most important aspects of foundation engineering and Professor Dinesh Mohan with his enormous experience in this field has highlighted some of the important problems faced in the design, construction and testing of Pile Systems. Starting with the historical background, he touched upon various aspects of the problem such as skin friction, group action, pile caps, lateral loads, defects in piles, ground heave and negative drag, etc. bringing about the latest thinking on this subject. He stressed the need of feed-back data from the field, case studies and instrumentation so that the designs could be further rationalised. He also described some of the special piling systems developed recently and concluded by taking an overview of piling problems. Professor Dinesh Mohan has combined the theoretical considerations of piling problems with the construction aspects of the industry and given such a complete and thorough treatment to the subject which he alone is capable of. Gentlemen, on your behalf and on behalf of the Society, I once again thank Professor Dinesh Mohan for the excellent lecture. Let us give him a big hand to show our appreciation.

We are grateful to Professor Ranganatham who kindly agreed to preside over the function at a short notice. By gracing the occassion, he has lent additional granduer to the lecture.

On behalf of the Executive Committee of the Indian Geotchnical Society, I would like to convey our thanks to Professor R.K. Katti of Indian Institute of Technology, Bombay, Dr. N.V. Nayak, Organising Secretary and other members of the Organising Committee of GEOTECH-80, for providing excellent arrangements for this lecture.

I must thank you all for your participation in deliberations and making this function, a grand success.