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Observations on Pile Design and Construction Practices in India

Shailesh R. Gandhi[1]

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Abstract This paper reviews the design and construction practices on pile foundation being adopted in India observed over the last three decades. The factors that affect the choice of the pile have been discussed highlighting the necessity of initial field tests prior to the detailed design. A case study is presented where choice of the pile was made based on initial field tests on piles of promising types. The common problems faced during pile construction in each type of pile with possible remedial measures that can improve the performance are described. Various issues in pile design are reviewed with some of the experimental and numerical work carried out on lateral load on piles in slope, pile group effect under lateral load, negative drag force, rock-socketed pile, etc. Finally, the need for more such full-scale field tests and monitoring with instrumentation is emphasized to achieve an optimum pile design along with few case studies on such tests carried out. The review reveals remarkable improvements, both in the design and construction, comparable with that in other countries.

Keywords Pile design · Pile construction · Field tests · Group effect · Negative drag · Case study

✉ Shailesh R. Gandhi
srgandhi@iitm.ac.in

1 Department of Civil Engineering, Indian Institute of Technology Madras, Chennai 600036, India

Introduction

In early 80's, bored cast-in situ piles and driven cast-in situ piles were widely used. The technology for precast piling was picking up. Very few projects used precast piles with a limited size as the spliced pile technology was not in use. However, pile use was extensive due to lack of confidence in various ground improvement methods. The choice of pile type was based on availability of piling equipments. It was very rare that initial pile test on two or more promising pile type are carried out to arrive at the choice and corresponding pile capacity. The phenomenon of the effect of pile driving, negative drag force on pile, methods of reducing drag force, reduction in lateral capacity due to pile group effect was not known precisely. The concept of instrumented field test to verify the design parameters was picking up though with a difficulty in procuring the instrument at high cost. Considerable improvements have been visualized in the last three decades which has improved the confidence level in pile design as well as construction. Effective interaction between the client, geotechnical designer and the contractor is desirable to constantly review the field test results (preferably with suitable instrumentation) and accordingly modify the design. Many of the design parameters can be evaluated better based on field testing as they are highly dependent on the specific strata, equipment used and the method of construction.

During early 80's most of the designs followed the codes of practices which are based on past experience using conservative parameters. In several instances the field test results have shown much higher load capacity compared to the theoretically estimated capacity. However, in very few cases the design is modified as the field test results are generally available only after sizable work has been completed. Instrumented field tests which can provide vital information on actual capacity with individual share from different layers were particularly very rare due to the high cost of imported instruments. Also the clients generally have a notion that an instrumented test is more an academic exercise and does not benefit the project.

Bored cast-in situ piles and driven cast-in situ piles were the most commonly adopted as precast pile technology was limited. Only very few projects used precast driven piles, some used imported spun pipe piles and even large diameter pre-stressed pipe pile cast at site. Precast piles are still not available off the shelf in the market and a casting yard is essential in each site which sometimes do not permit use of this type due to lack of space or due to less number of piles justifying the setting up of a casting yard. The driven cast-in situ piles have limited diameter of about 550 mm and depth of 25–30 m below the ground level whereas in case of bored piles, large diameters of 1.5–1.8 m have been used. The technology available permits the bored pile construction to large depth exceeding 60 m, socketing the pile in any rock for several meters and installing it even through water in challenging marine environment.

As far as construction technology is concerned, most of the bored piles, even for major projects were installed by a conventional tripod with a chisel and a bailer or with direct mud circulation (DMC) method. The use of rotary piling rigs was available much later during 80's. The conventional method using a tripod is very slow and less effective in use of temporary liner to a deeper level, lowering of steel cage in parts with number of lap joints, concreting with a tremie pipe, etc. The rotary rig on the other hand, is much faster in installing a temporary liner to a large depth and for pile construction. However, one of the major problem in use of rotary rigs is lack of continuous circulation of the bentonite slurry (unlike in the case of a DMC method) which may result in gradual increase in the slurry density as well as viscosity if not checked and replaced. The increase in slurry density can result in suction pressure under the operating bucket resulting in reduced capacity of pile and difficulties in concreting, if not taken care adequately.

Pile design methods are well established but the selection of key parameters such as earth pressure coefficient K, adhesion factor α, etc. has been a real challenge. If a project schedule does not permit adequate initial field testing to evaluate some of the important parameters, the design is made with a conservative approach. A detailed analysis using p–y, t–z and q–z curves is rarely done for the land piles. The interactions effect among the piles under vertical load is taken care by providing 2.5D or 3D spacing if piles are terminated on rock or in soil respectively. However reduction due to the interaction effect under the horizontal loads has to be evaluated as this can be significant even if a spacing of 3D is provided.

The codes on pile design do not give adequate information on evaluation of the negative drag force on piles passing through settling soil layers, methods to reduce the drag force, etc. Different approach is observed in assessing the safe capacity when subjected to drag force. It may be noted that this parameter cannot be obtained based on load test as the settling layers will offer positive friction during the period of testing. Some of the above aspects have been discussed in detail in the following section based on either case study or detailed analysis carried out.

Choice of Pile Type

Ideally, choice of pile type is dependent on several factors such as strata conditions, availability of equipments, method followed for construction, loading requirements, etc. The most crucial parameter governing the pile performance is method of construction which is found to have large variations.

In case of bored pile (non-displacement pile) in cohesion less soil or in stiff clay, constant presence of bentonite slurry along the wall of the bore hole as well as formation of filter cake can reduce the frictional resistance. On the other hand, a driven pile (displacement pile) will densify the surrounding strata in the process of installation. Driving of the pile will normally result in densification of the surrounding soil and increase in the normal stress on pile surface resulting in higher frictional resistance. Similar increase is expected in the end bearing resistance also. In view of the above, the driven piles are known to have better performance compared to bored pile of similar size, except in sensitive clay stratum where driven pile can have reduced capacity. Other factors favoring use of precast driven piles are high grade of concrete with good quality control, neat site condition as no bentonite slurry is used, possibility of applying a slip layer to reduce the drag load, assessment of pile capacity during driving based on "set" value, higher lateral capacity, easy to install with a rake, faster construction, etc.

In sensitive cohesive soil, driving of a pile creates remolding of clay strata in the surrounding. Depending on the sensitivity of the strata there is corresponding reduction in the shear strength and hence reduction in capacity of the pile. In such case, bored piles (non displacement method) prove better.

When pile socketing is required in hard strata, either to carry higher compression load or for uplift load, driven pile cannot penetrate deep into such strata. For soft rocks such as chalk, mud stone, shale, etc. it is possible to drive the pile into such rock and derive higher capacity. For other variety of weather rock or hard strata, driving will result in high driving stress and possible damage to pile, and breaking of rock mass into smaller fragments surrounding the pile tip resulting in reduced capacity. On the other hand, boring through a hard stratum with a chisel will result in a very rough socket to which the in situ concrete gets strongly bonded resulting in higher friction. Whenever socketing in hard strata is required, bored cast-in situ pile is more appropriate. Some of the other factors favoring bored piles are less percentage of steel, less head room requirements, less noise and vibrations, etc.

Above factors do not help the design engineers in choice of pile type as the soil strata may comprises of several layers with varying properties. Initial field test on promising pile type therefore helps to get the following information:

- Choice of pile type (based on the actual performance at site).
- Establish that the proposed equipments/method can be used for installation of the required pile size to the required depth.
- To confirm the capacity and to optimize the pile design under vertical compression, pull out and lateral loads.
- Difficulty faced, if any during execution so that suitable remedial measures in the installation procedure can be included in the tender specifications.
- Structural designer can prepare the pile layout with more confidence and revisions in pile layout due to revision in capacity can be minimized.

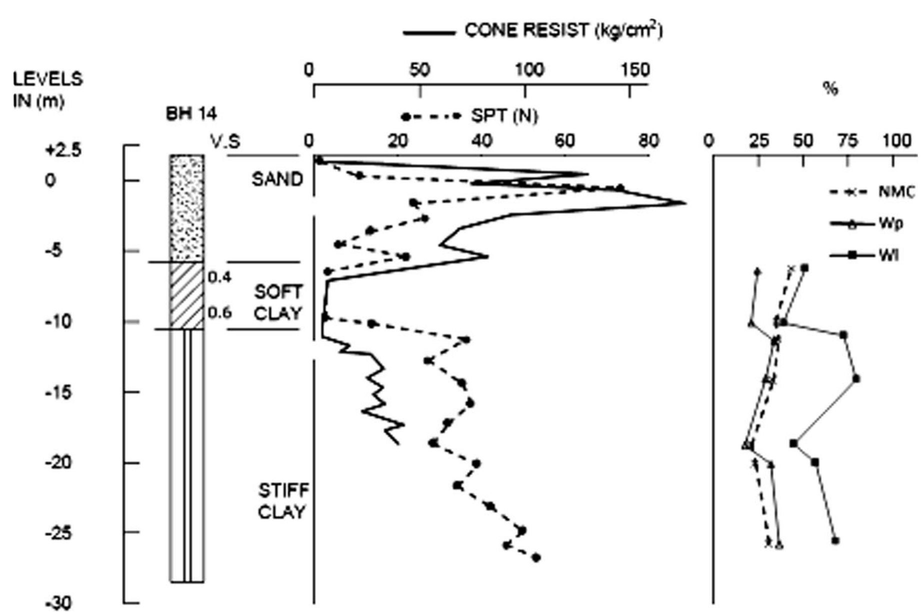
From execution point of view, conducting such initial tests prior to award of piling contract is difficult unless a separate tender is made exclusively for installing the test piles and for conducting the initial load tests. This will obviously have higher cost due to mobilization of the equipments for a limited number of piles. However, in view of the above benefits, the higher cost of a separate tender for few initial test piles can always be justified in the case of a major project. There are several instances where the initial tests are carried out only after the piling work has been awarded and the test results demand changes in either pile size or pile capacity resulting in revisions in drawings which implies several contractual issues and can delay the work.

Case Study on Pile Choice

Raju and Gandhi [18] have described the choice of foundation based on field tests for a major fertilizer plant on the east coast. Thousands of piles were installed with depth varying from 22 to 36 m. The strata comprised of a sand layer 4–8 m thick followed by very soft marine clay up to a depth of 6–16 m below the ground level. This is followed by a bearing layer of stiff clay. Typical soil profile is shown in Fig. 1.

While selecting the pile type, a major concern was the settlement of the soft clay layer under a site fill to raise the ground level. It was estimated that the surcharge due to the fill can create large magnitude of negative drag force on the piles. The drag force is anticipated not only from the soft clay layer but also from the surface sand layer which was expected to settle over 150 mm under the site fill up to FGL. In order to reduce the foundation cost, precast concrete pile was an obvious choice as the negative drag force can be considerably reduced. The effectiveness of the bitumen coat in reducing the drag force was verified experimentally (Kandasamy [11]). Series of pull out tests were carried out on model concrete piles of 100 mm

Fig. 1 Typical soil profile [18]



diameter and 600 mm long, with and without bitumen coat. Typical load displacement behavior under pull out load is shown in Fig. 2. According to this study, the drag force could be reduced by 60–90 %. Though precast concrete piles can be economical by reducing the drag force significantly, there were uncertainties about the availability of contractors for installing driven piles to such large depth. Also it was necessary to verify the performance of this pile type in the field due to the lack of experience in precast pile construction.

To decide on pile type, two promising pile types, namely precast driven piles and bored cast-in situ piles were selected for the initial tests. Three pairs of a precast pile and a bored pile of comparable dimensions were installed and load tested. The precast piles had a square cross section of 400 mm × 400 mm whereas bored cast-in situ pile had a diameter of 450 mm. The length of all 6 piles was about 22 m with nearly 6 m penetration in the bearing stiff clay layer. During testing, the friction contribution of the top sand layer which is likely to add a negative drag force was eliminated by providing an oversize casing around the pile up to the bottom of the sand layer and removing the sand in the annular gap between the pile and the casing. Each of these piles was loaded till failure using conventional kentledge method. The load test results are shown in Fig. 3. As can be seen, the precast driven piles have consistently better performance compared to the bored piles. All the three driven piles have higher ultimate resistance and much smaller displacement. Based on these field test results, finally precast driven piles were adopted for the project. Following criteria was adopted while selecting the pile length and capacity.

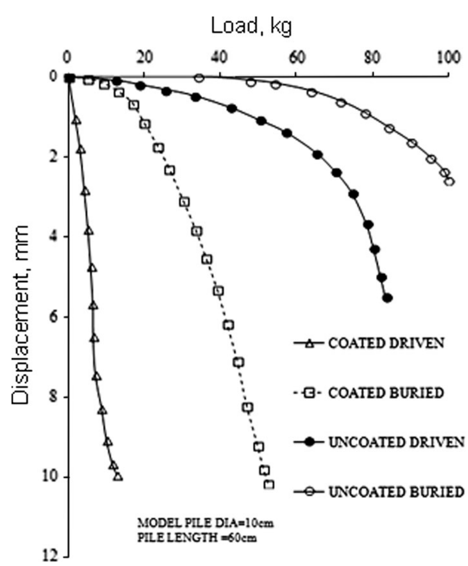


Fig. 2 Effect of bitumen coating on pull out resistance [18]

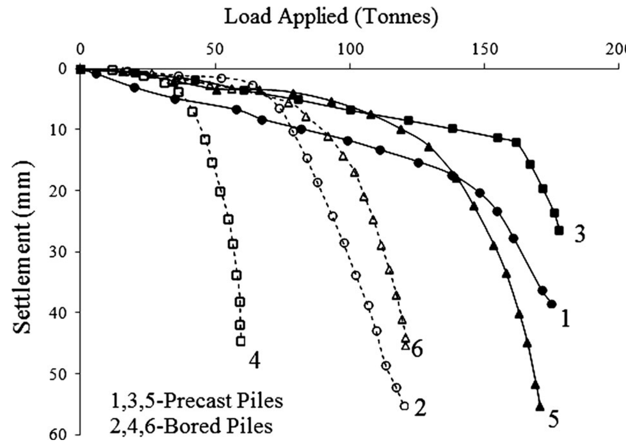


Fig. 3 Comparison of precast and bored piles [18]

- Structures with significant horizontal load, piles were used with 6 m penetration in the stiff clay with a design capacity of 900 kN as indicated by the test results. Though the structural capacity is not fully utilized, the number of piles required will be more to meet the lateral loads.
- Structures where the vertical load was governing the number of piles, pile length was increased up to 34 m with about 12–14 m penetration in stiff clay to achieve a higher capacity of 1100–1300 kN.

As the friction contribution from the top sand layer was anyway not to be considered due to the bitumen coat application, water jetting technique was used for the initial penetration of the piles up to the bottom of sand layer. Special water nozzles were attached to a pipe about 12 m long on the two diametrical opposite faces and water was jetted under pressure of about 1000 kPa. With jetting action, the pile was sinking in the ground very fast under its self weight itself. Once a penetration of 10–12 m was achieved, the water pipes were withdrawn and the pile was driven with drop hammer up to the founding level. This made the pile installation very fast and also reduced the noise/vibration level in the surrounding area.

During execution, 22 additional routine load tests were carried out to confirm the above capacity which showed satisfactory results. Each pile, after deducting the reduced negative drag could support a design load of about 800–1300 kN compared to 500–600 kN only estimated in the case of bored pile.

Difficulties in Pile Installation

While there has been considerable improvements in the piling equipments and know-how in pile construction, there are several possibilities of defects in the pile installation.

Based on the past experience, some of the common problems noticed in pile installation procedure are highlighted below. These problems are less where the piling contractor is experienced and has well trained staff to execute the works. The piling contractors may hire locally available technicians who may not have adequately experience for this specialized job. It is also noticed often that the piling equipments are locally fabricated and do not have some of the essential features which can significantly affect the piles installed.

Precast Driven Piles

The main difficulty is in handling a larger size of pile in terms of cross section as well as length. Unless the number of piles to be installed is large and adequate space is available to establish a casting yard, this type cannot be adopted. Many times, the choice of this pile type is not considered due to above limitations. Another difficulty is in prediction of pile length which may vary if the strata have variations resulting in excess length projecting out which needs to be cut little above the cut-off level, if the hard stratum is found to be at higher level. On the other hand, if the hard stratum is at deeper level, it needs to be further driven with a follower section made of steel and later grown with in situ concrete. Segmental piles are being used which can address the above issues but the cost of the joint (some have been patented) is high.

The capacity of pile can be estimated during driving based on set value and a detailed pile drivability analysis. The set value is dependent on number of parameters such as weight of the pile cap, stiffness of the cushion, soil damping parameters, etc. which are not reflected in Hiley's formula. Narasimha Rao and Gandhi [16] carried out a detailed parametric study using the wave equation to find the influence of the above parameters on the resulting set value and driving stresses which can help to choose an optimum weight of the pile cushion for an efficient driving.

Many times, where stratum is cohesion less, water jetting technique can be used instead of driving with a hammer. While this reduces the time of installation, noise and vibration problems, it does loosen the surrounding strata and reduces the frictional resistance as well as end bearing resistance and the lateral capacity. The loosening of strata created by water jetting can be partly rectified if last few meters of the pile is driven with a hammer.

If above limitations are taken care of, this type has several advantages and provides better performance. Precast piles, including hollow pipe piles of large dimensions have been successfully used in the country in spite of above difficulties.

Driven Cast-In Situ Piles

Driven cast-in situ piles also have limitations on size. The maximum diameter for which the equipments are available is limited to 500–600 mm and the depth to which installed, even with a jointed casing pipe is limited to 30–35 m. The water tightness of the joint between the pile shoe and the casing is very important as the pile is driven several meters below the ground water level and any accumulation of seepage water through this joint will affect the quality of concrete at pile tip. Many times accumulated water is found at the bottom which needs to be checked and cleared before concreting. Another problem faced is about uplifting of the casing pipe due to buoyancy before the steel cage is lowered. If pile tip is resting on hard strata, such upward movement can reduce the effective end bearing capacity. Concreting of pile and withdrawing of the casing also need an experienced operator to maintain a minimum height of fresh concrete within the casing while withdrawing to ensure that the outside ground water/soil do not force into the casing. Stage wise pouring of concrete and adequately tapping the casing with driving hammer at the top is essential to ensure that the concrete is compacted and occupies the gap created by the thickness of the casing pipe and the projected portion of the pile shoe. Driving of a pile adjacent to a freshly installed pile can be avoided as this may affect the concrete which has not adequate strength to withstand the displacement/vibrations. Unless adequate spacing exist between the piles, it is better to install alternate piles.

Bored Cast-In Situ Piles

It is often seen that small piles of diameter 400 mm or under-reamed pile of diameter 300 mm is being used. These will have severe constraint of flow of concrete through a small size tremie pipe which shall be of minimum 200 mm internal diameter.

Soil collapse during the boring operation is a common problem, in spite of bore hole stabilization with bentonite slurry, particularly in a loose cohesion less saturated soil. The temporary liner used has generally a limited depth of 2–5 m below the ground level as the withdrawal of the casing pipe with conventional tripod rig is difficult. For a good quality concrete, it is preferable to use a liner to its full length and withdraw the same after concreting. This is possible with rotary rigs which have a special attachment for installing the casing. Such casing pipes are with joints and can be installed or withdrawn by applying the static force plus torque by the machine. This will ensure good quality of pile and ensure no extra consumption of concrete. In many situations, sacrificial permanent steel liners have been used to a large depth which is very expensive. In case of rock socket, it shall be ensured that the liner, if used do not penetrate in the rock socket but it is terminated just below the rock surface. This is to ensure that there is no gap between the rock cut surface and the casing in which the concrete cannot flow and the skin friction from rock can be affected. In the absence of a liner, concrete gets strongly keyed to the rough rock surface and provide better frictional resistance.

Even if a liner is used, it is important to stabilize the bore with proper bentonite slurry with proper density and other properties throughout the pile construction. Properties of the bentonite used, ratio of water to bentonite adopted, density, pH values, etc. are to be regularly checked and the slurry shall be replaced if it does not meet the requirements.

Piling by rotary rigs and chisel/bailer method do not have a continuous circulation of the bentonite. Many times it is noticed that the bentonite slurry level in the bore hole is not maintained up to the top of the bore hole and it keeps going down every time the bucket is withdrawn. This can result in serious inward seepage of ground water into the bore hole and loosening of the soil strata around the pile shaft. Use of polymer slurry instead of bentonite slurry is being used increasingly though the cost is high due to several merits.

Suction pressure created while withdrawing the operating tool from the bore hole is often a problem which can result in serious damage to the soil strata prior to completion of the pile. In rotary rigs particularly, where continuous bentonite circulation is not provided and the machine is powerful to withdraw the tool at high speed, there are more chances of suction effect. The operating tool shall have adequate provision of vertical openings within the tool which permit free flow of the bentonite slurry through the tool while it is withdrawn.

Cleaning the bottom of the bore hole before concreting also has limitations, particularly in DMC method where the upward flow velocity of the slurry is not always adequate to carry the cut soil particles, unless there is no large gravel size particles or stones. Inadequate cleaning of the bottom results in soft toe and reduces the pile capacity. The problem is more when depth of the pile is large and in such case use of air lift system to flush out the sediments is essential.

Marine pile construction in locations with high tidal variations above 5 m in certain area such as Kandala, Hazira, Dahej, etc. requires special precautions. During boring operation through the steel liner, when the tide level is lowest, the level of slurry or sea water in the casing may remain up to the top and can create a piping failure due to high seepage flow under the tip of the casing if the penetration beyond the seabed bottom is not adequate. Similarly, inward flow can also occur when the tide level is high and the slurry level in the casing pipe is not maintained above the tide level. Both the above situations can result in considerable loosening of the strata around the pile and reduction in frictional capacity. This can be eliminated by increasing the liner penetration adequately to limit the exit gradient to a safe limit.

Concreting with a tremie pipe requires several checks such as minimum diameter of the pipe, ensuring that bentonite slurry in the borehole do not get mixed with the fresh concrete, maintaining minimum penetration of the pipe in the fresh concrete while withdrawing the tremie pipe, pouring extra concrete above the cut-off level to ensure that the contaminated concrete at the interface between the concrete and the slurry is eliminated, etc. Unless the rig operator is experienced, ensuring the above is difficult and often leads to a defective pile.

While preparing the pile head, there are different methods used to eliminate the contaminated concrete above the cut-off level. The usual practice is to chip-off the concrete manually with chisel and hammer to a reasonable finish at the cut-off level. This has to be done after the concrete gains the required strength to ensure that the concrete below the cut-off do not get damages during the chipping-off operation. Sometimes very heavy hammer is used to remove this extra concrete and that can damage the pile concrete below the cut-off. Use of a hydraulic system which has number of radial jacks can also be used which can crush the concrete little above the cut-off without impact under the static jack force is better and faster. Another practice is to remove the contaminated portion above the cut-off level immediately after casting of the pile. The removal can be either manual if the cut-off is closer to the ground level or using a special tool which permits scoop out the contaminated concrete from dipper level. Adequate care shall be taken to ensure that concrete is not removed below the cut-off level.

Precast Pre Bored Pile

This type is used very rarely though it derives the benefits of both precast and bored pile. Casting of these piles requires a grout pipe to be centrally placed which can make the needle compaction difficult particularly for smaller section. Also, if the length of the pile is more, and splice joint needs to be adopted, making a splice joint after lowering one piece which is still suspended in bore hole and providing the grout pipe through the joint is difficult. The method is not suitable where a rock/hard stratum is not available to terminate the pile. This is because the frictional resistance is reduced due to the presence of grout around the pile surface.

Pile Design Issues

Vertical Compression Load

As far as design of pile foundation is concerned, the formulae to arrive at the vertical compression load carrying capacity based on the known soil parameters are well established in the code [10]. These recommendations are generally in line with the other codes. However, certain key parameters which govern the capacity are often debatable as discussed below:

- Restricting the effective vertical stress for estimating the skin friction also to 15–20 times the pile diameter, as in the case of estimating the end bearing resistance. The necessity of restricting this is not well understood and it is often observed that if restricted, the pile length predicted is very high and driving becomes difficult or the load test shows a very conservative design.
- The co-efficient of earth pressure K, which is recommended to be 1–2 for a bored pile and 1–3 for a driven pile, is highly dependent on the method of installation. In case of large projects, this shall be evaluated by back calculating from the field test results preferably on instrumented test pile for an optimum design. However, some of the consultants do recommend a very conservative value and do not even permit to use the value based on load test results.
- In the case of rock socketed pile, [9] recommends not to consider any frictional resistance from the overburden soil. This is reasonable as the tip resting in rock may not have any significant settlement and mobilization of shear in overburden layers with limited relative movement between the pile and soil cannot be justified. Therefore it is preferable to derive the entire resistance from the rock socket only.
  The formula based on compression strength test results is conservative due to the fact that the intact rock cores do not represent the joints, its spacing, thickness, etc. The formula based on pressure meter test has main limitation of evaluating the limit pressure as most of the rock stratum is hard to achieve the limiting pressure during the pressure meter test. The formula based on shear strength is similar to the method suggested by Cole and Stroud [4] and the scale of shear strength based on SPT(N) values can be used to assess the shear strength, though the same is not included in the code.
- In case of a piled raft, the skin friction mobilization in the portion immediately under the raft where the relative displacements between the pile and the soil is very low is negligible. Clear guidelines on estimating the pile capacity, particularly the depth to which the skin friction is low is not available. Maharaj and Gandhi [13] discussed the load transfer based on non-linear finite element analysis of piled raft to evaluate this depth.
- The permissible settlement is not well defined and this also depends on the pile group size and the spacing between the piles. 3D spacing for friction piles and 2.5D spacing for an end bearing piles takes care of the vertical group effect but it may influence the resulting settlement as well as horizontal capacity which needs to be analyzed.

Horizontal Loads

Pile capacity under the horizontal load is governed mainly by the surface layers and not the bearing layer as most of the piles have elastic behavior due to its large length compared to its rigidity in lateral bending. The pile design code [10] provides the guidelines to estimate the fixity depth and to estimate the pile deflections under a given lateral load for both free head and fixed head condition. However, the code does not provide the procedure to find limiting lateral capacity as given by Broms [3] or the variations in deflections, shear, soil pressure and bending moment with pile depth as suggested by Matlock and Reese [14] based on elastic analysis of pile. However, above references are commonly used in pile design.

Vertical piles installed are often subjected to lateral drag either due to an unstable slope of the excavation or due to area surcharge on the adjacent area on a weak ground. In a typical berthing structure where the piles are being installed, the land reclamation behind the berth is also taken up simultaneously. If the ground is poor and the rate of fill placement is high, the weak layer may yield towards the sea slope and result in large lateral drag on the freshly installed piles. These piles might not have been cured and could be in free head condition as the deck work construction may not be in progress. This can result in large lateral displacements of the piles toward the sea, unless suitable steps have been taken. Muthukumaran et al. [15] have described a detailed monitoring of pile supported berthing structure where the lateral deflections in pile as well as the diaphragm wall behind the berth were measured using an inclinometer embedded in the pile/diaphragm wall. The actual lateral movement measured at the top of the berth (+4 m above MSL) was about 17 mm for the diaphragm wall at the rear and 30 mm for the pile closer to the berthing face.

Sivapriya and Gandhi [19] have described a detailed study to evaluate the lateral capacity of piles on slopping cohesive ground including the group effect. Based on a parametric study with experimental and numerical analyses, design charts have been prepared to estimate the pile capacity depending on the pile position from the crown of the slope and spacing between the piles. Sivapriya and Gandhi [20] have evaluated an excavation scheme where pile supported peripheral basement slabs of a commercial building in a top down excavation is subjected to lateral load due to the earth pressure on the diaphragm wall. The adequacy of the piles to support the slab and its lateral deflections were evaluated using a 3D PLAXIS code. Also the diaphragm wall was instrumented with an inclinometer to verify the lateral deflections during execution of the excavation and the measured lateral movement was compared with that based on PLAXIS analysis.

As far as group effect is concerned, it may be noted that piles even with a higher spacing up to 5D can have a group effect and reduce the lateral capacity. Wherever the number of piles is governed by the lateral load, it is better to check this while designing the spacing. Adequate pile spacing for no group effect can lead to larger size of the pile cap with corresponding increase in thickness and reinforcement in the pile cap. Many times the space available is also a constraint and in such case we have to accept the reduced lateral capacity of the pile on this account.

Gandhi and Selvam [8] carried out a detailed experimental study on the group behavior of aluminum model pipe piles of 18.2 mm external diameter under lateral load in a test tank of sand. 1 g model tests were carried out in a test tank 0.7 $\times$ 0.7 $\times$ 0.6 m deep filled with dry river sand. To create a fixity condition at the top of the piles, the pile cap, even in case of a single pile, was provided with two parallel arms hinged with the pile cap and at sufficient height above as shown in Fig. 4. Compared to the lateral displacement of the model pile, the height of the arms is much larger and hence the upward movement of the pile cap due to small lateral displacement is negligible. This arrangement ensured no rotation of the cap, even for a single pile. 21 model tests were carried out for piles with different configuration and center to center spacing. The spacing was maintained 3D in the direction perpendicular to load direction whereas in the load direction the spacing was varied from 4D to 12D.

Typical result of load versus displacement for tests on two pile groups is shown in Fig. 5. The load shared by each pile is calculated with the assumption that the front pile behaves in a manner similar to an individual pile and the rest of the load applied on the group is taken by the rear pile. The reduction in the load shared by the rear pile is evaluated as a load factor, α which is a non dimensional parameter defined as the ratio of the load taken by a rear pile in the group to the load taken by the front pile at same displacement. The load factor arrived as above is presented in Fig. 6 for different pile spacing s which is normalized by the relative stiffness factor, T. The results of similar work

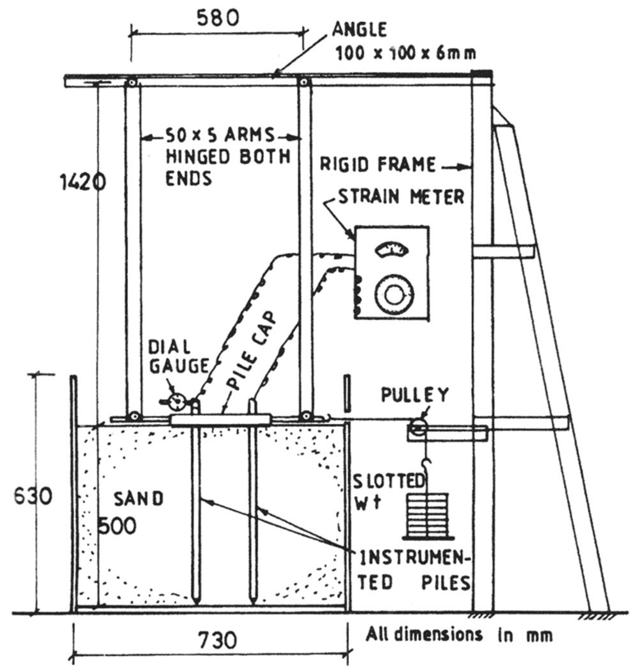


Fig. 4 Experimental setup [8]

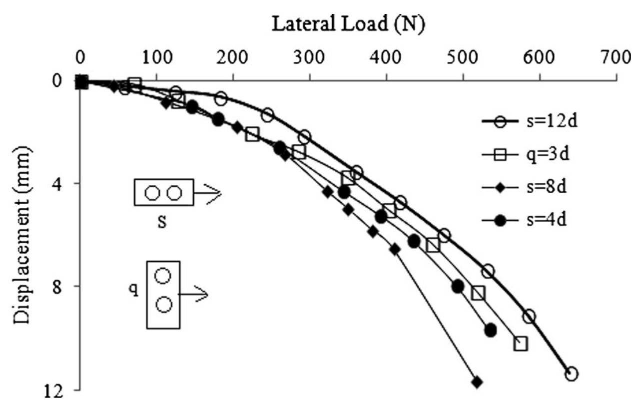


Fig. 5 Displacement of two-pile groups [8]

carried out by Franke [6] are also included and found comparable.

Based on further analysis it was concluded that the reduction factor α increases linearly with increase in pile spacing to stiffness ratio(s/T) up to a value of 2.0 and beyond s/T of 2.0, the value of α remains constant with a value close to 1.0, meaning no reduction in the capacity of the rear pile. It is also concluded that optimum spacing between the piles in the direction of load for a maximum group capacity is about two times the relative stiffness factor T.

A non-dimensional pile multiplication factor C_m which is defined as the ratio of lateral load on a single fixed head pile to that of a single free headed pile (for a same deformation) has been evaluated and plotted against the non-

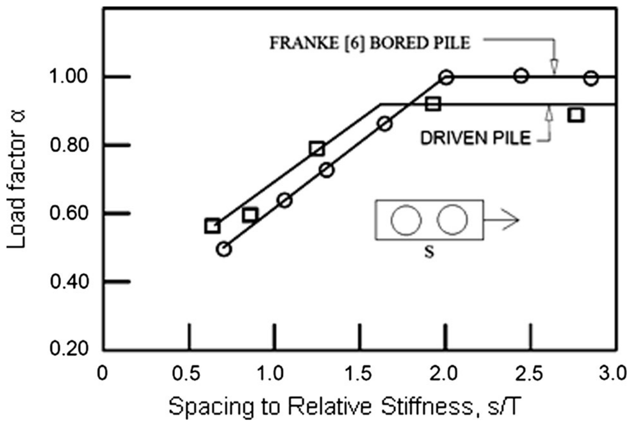


Fig. 6 Effect of spacing on load factor α [8]

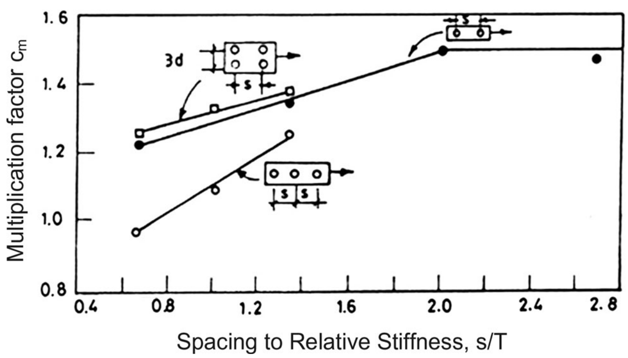


Fig. 7 Variation of multiplication factor with spacing [8]

dimensional spacing to stiffness ratio as shown in Fig. 7. As can be seen, the factor C_m in case of a two pile group increases linearly with the pile spacing up to a spacing equal to about two times the relative stiffness factor T. Beyond this spacing it remains constant.

Negative Drag Force on Piles

Negative drag force on pile can significantly reduce the pile capacity. Though the piling code [10] cautions the designer adequately about the reduction to be applied on this aspect, there are no clear guidelines on estimating the drag force and the factors that need to be considered in assessing the drag force. Both the laboratory and field tests have shown adequately that the drag force can be reduced significantly by applying a slip layer such as a bitumen coat on the pile surface over the zone in which the soil layers are undergoing large settlements. The application is possible only in use of precast driven piles or steel piles. Following factors shall be kept in mind while considering the drag force.

- No negative drag need to be considered if there is no surface loading triggering the settlement in the settling layer. Many times, negative drag is applied because there is a soft clay layer, even if the pile foundation is for a bridge pier in a river bed where no surcharge loading is expected.
- Settlement of the pile with depth and the settlement of the soil strata with depth shall be estimated based on the load on pile and the surface load on the surrounding soil. This is required to determine the neutral plane where both the settlements are equal. The negative drag is to be applied only for the portion above the neutral plane. All the soil layers above neutral plane will exert negative drag on the pile.
- Individual piles and piles in smaller group will have large negative force compared to inner piles in a large pile group which can be limited to a maximum value of the area represented by the pile times the surface load intensity applied which triggered the settlement of soil layers.
- The drag force can be reduced considerably by 60–80 % after applying a slip layer such as bitumen coat over the portion above the neutral plane.
- As the drag load is the limiting shearing resistance the ground can offer on to the pile, it does not require a factor of safety. The ultimate load capacity of pile shall be therefore taken as ultimate resistance in end bearing plus ultimate resistance in skin friction for all the layers between the pile tip and neutral plane minus the estimated drag force. This shall be divided by a desired factor of safety to arrive at a safe load on pile.

Khare and Gandhi [12] made a detailed experimental study on frictional characteristic of different pile material with and without different slip layers. Direct shear tests were also carried out on model test block representing the pile surface as bottom half of regular direct shear test apparatus and the upper half portion was filled with sand at required density to measure interface friction for different thickness of bitumen coat. Typical result for uncoated and coated specimen is shown in Fig. 8. As can be seen, there is significant reduction in the shear stress mobilized for a bitumen coated specimen compared to an uncoated specimen.

In addition, detailed model pile tests were carried out on piles in a sand bed with different slip layer thickness and surcharge condition. Typical results are shown in Fig. 9. As can be seen, compared to an uncoated pile, the shear stress developed on coated pile surface is far less. The pile L/D ratio was varied to check the scale effect. The t–z curved developed based on these tests have been compared with the published result by Coyle and Sulaiman [5] as shown in Fig. 10 and found comparable. It was observed that for a given L/D ratio, the t–z relationship is nearly free from scale effect. Based on this study it is concluded that in

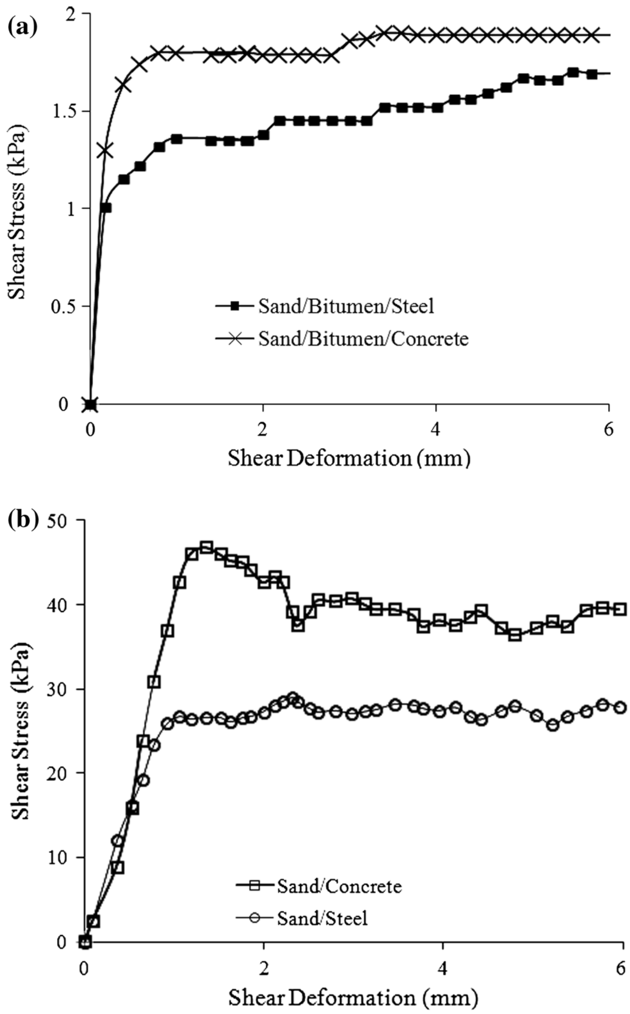


Fig. 8 Interface shear of sand with: **a** uncoated blocks; **b** coated blocks. Normal stress = 75 kPa [12]

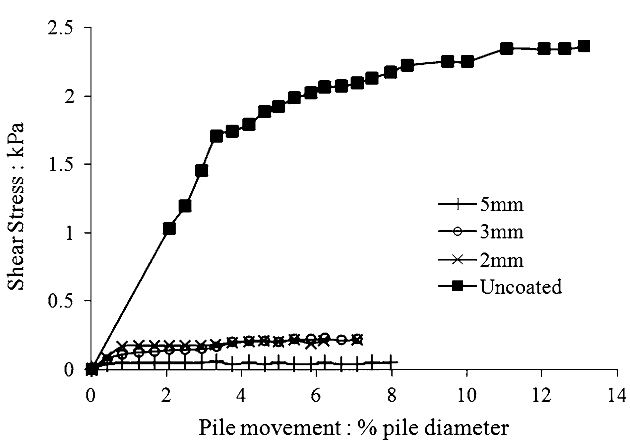


Fig. 9 Effect of coat thickness on shear stress [12]

case of uncoated piles, the relative movement of 4–6 mm is adequate for full mobilization of drag force whereas in case of a bitumen coated piles, relative movement of only

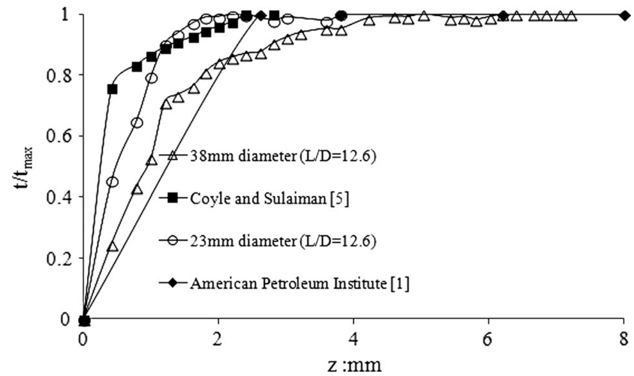


Fig. 10 Scale effect on t–z curve [12]

0.5–1 mm is sufficient indicating that failure will always occur in the bitumen even if the relative movement is very small justifying full reduction of the drag force even for very low settlement in the ground.

Field Tests and Monitoring

In view of the various limitations of the geotechnical investigations, laboratory testing, theory to predict the pile capacity, etc., field testing is considered to be most reliable. Field tests can be planned not only to get the pile capacity but to evaluate several design parameters which will help the designer to optimize the foundation. Unfortunately, detailed field monitoring is carried out only in few projects for various limitations. Use of special instruments and other devices have been used for different purpose as given below:

(a) Strain gauges or load cells for axial load distribution in a pile at various depth. This enables to find the end bearing resistance and the frictional resistance from each of the layers. It is also possible to analyze the data further to evaluate the t–z as well as q–z curves, Negative drag force, optimum pile depth, etc.
(b) Inclinometer tube embedded in pile shaft to record the lateral displacement in pile at various elevations below the ground to check whether the pile has flexible behavior or rigid behavior under the lateral load.
(c) Load cell or soft toe at pile tip to measure directly the end bearing load and thereby separating the frictional and end bearing resistance of the pile.
(d) Pull out tests on strain gauge instrumented short pile installed up to the bottom of settling layers to evaluate the likely drag force from different layers.
(e) Elimination of soil friction from certain surface layers to estimate the capacity of the deeper layers by providing an oversize casing pipe around the pile up

to a depth to which the friction is to be eliminated. This is used either to account for the negative drag or in case of marine piles, it represent the condition prevailing after dredging to the required depth. While an oversized casing eliminates the frictional contribution of these layers, the presence of these layers during load test does increase the frictional resistance of the layers below due to the surcharge applied which needs to be accounted appropriately.

Sundaravadivelu et al. [21] monitored a cargo berth shown in Fig. 11 where a diaphragm wall in the front of the berth was provided with lateral support using tie rods connected to a deadman diaphragm wall. The shear contribution of the two pile rows behind the diaphragm wall was not considered on a conservative side and the tie rods were designed to support the wall. In order to verify the contribution of the two rows of vertical piles, three load cells were fabricated and installed to monitor the tie rod force as the dredging depth was increased. The measured tie force was much less (73–129 kN) than the theoretical prediction which indicated that considerable lateral load was transferred to the two rows of piles though it was neglected in the design. Based on this study, another berth which came up subsequently was designed without tie rods transferring all the lateral force to the piles. This was possible because the structure required two more rows of piles to accommodate a fertilizer conveyor gallery. The elimination of the tie rods and deadman wall resulted in considerable savings in cost and completion time of the berth.

Gandhi et al. [7] conducted a lateral load test on a 1300 mm diameter pile of the above structure to check its adequacy to support a lateral load of 700 kN. In order to take the benefit of the minimum axial load on the pile, a combined vertical and lateral load test was conducted as shown in Fig. 12. The vertical load of 1400 kN was applied with a hydraulic jack supported on pile top using steel rollers to prevent any friction while undergoing a lateral deflection. It was found that in the presence of the axial load, the lateral deflection was considerably less (0.8 mm under a lateral load of 350 kN against about 5 mm under the same lateral load without axial load).

Raju and Gandhi [17] presented results of a field trial fill constructed measuring 30 m × 30 m × 5 m high as shown in Fig. 13 to measure the possible settlement and lateral movements in the ground due to the presence of a soft marine clay layer. The maximum vertical stress on top of soft clay layer due to this fill was estimated to be 80 kPa. This was to represent high surface loading over a large area that was anticipated from the floor of urea silo. The settlements due to the fill were measured at five different points as shown in Fig. 14 and lateral movement below the ground were monitored using inclinometer as shown in Fig. 15. The study helped to find the extent of lateral movement in the ground which was up to 20 mm and can increase the bending moment in the neighboring piles if not controlled adequately.

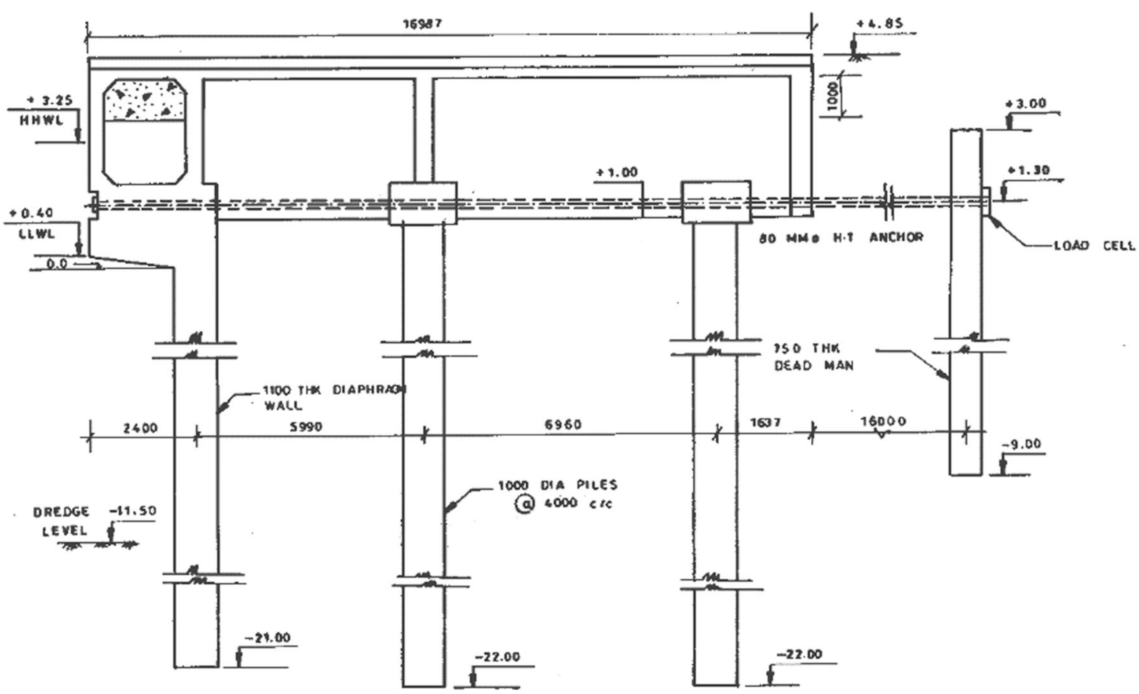


Fig. 11 Typical cross section of third general cargo berth [21]

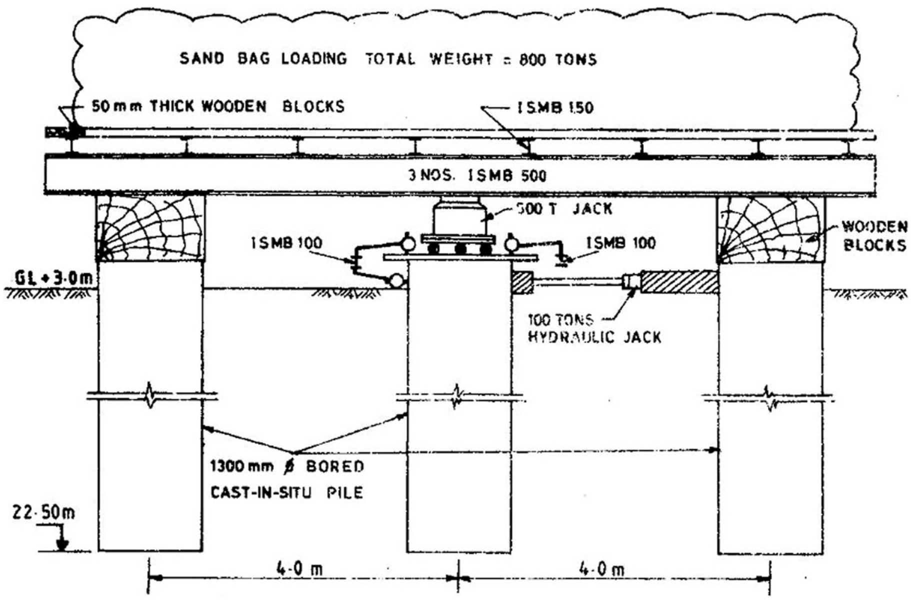


Fig. 12 Load test arrangement [7]

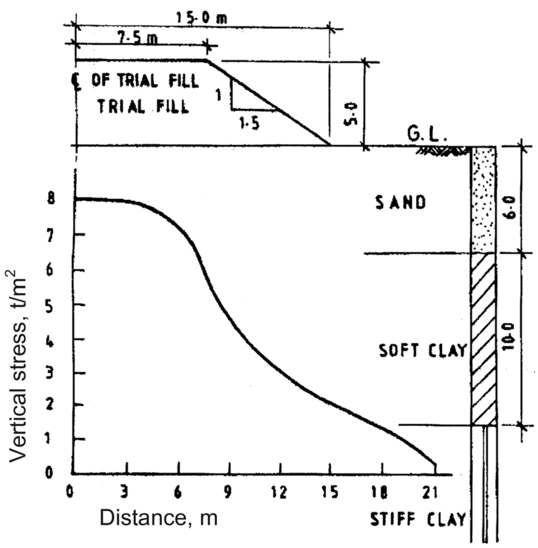


Fig. 13 Vertical stress distribution below the trial fill on top of soft clay [17]

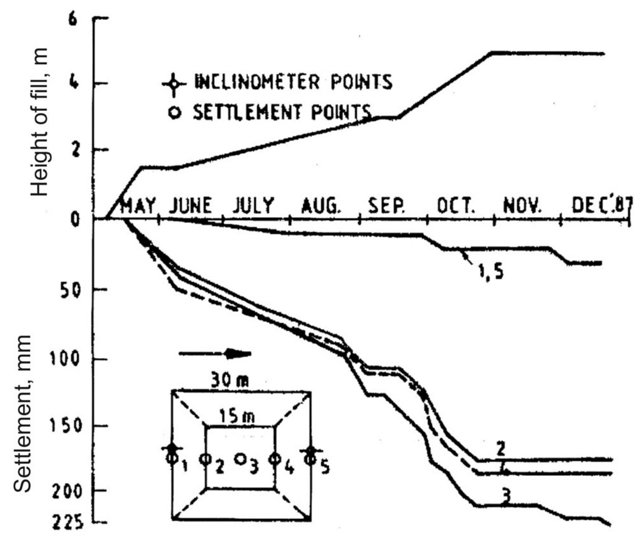


Fig. 14 Settlement below trial fill [17]

Role of Instrumentation in Foundation Optimization

Geotechnical instrumentation has been used successfully in pile foundation mainly for the following purpose.

Axial Load

Measurement of axial strain/load is very useful to evaluate the axial load distribution along the pile depth which enables to arrive at the frictional resistance offered by individual soil layers. In view of the heterogeneity of the pile material (particularly RCC piles) converting the measure strain to the axial load is a complex problem. Number of researchers suggested different methods for evaluating the axial load. In steel pile sections it is relatively simple to arrive at the axial load from the strain readings.

Use of bonded electrical resistance type strain gauges with adequate water proofing have been used to measure the strain in pile reinforcement. The axial load is arrived at based on the elastic modulus of the steel and concrete. Arun Prakash [2] carried out detailed experimental and numerical analysis to evaluate the axial load in RCC column based on strain readings. He also arrived at actual load a column of a hostel building was subjected to by embedding the gauge in column during construction. With easy availability of strain gauges of different types such as

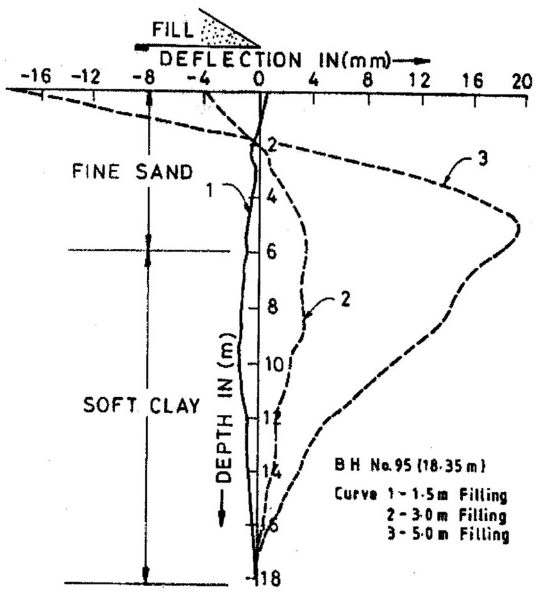


Fig. 15 Horizontal displacements measured under toe of the trial fill [17]

vibrating wire, embedment type, weldable gauge, etc. the instrumented pile load test have been used extensively.

Raju and Gandhi [18] instrumented a full scale precast driven pile of 400 × 400 mm square section, 22 m deep with electrical resistance type strain gauges specially fabricated at IIT Madras. The gauges were calibrated in the laboratory and welded to the pile reinforcement at site before concreting. During the load testing, the strain readings were taken as each of the load cell location for each increment of load of about 200 kN. The measured axial strain from the load cell was converted to axial load in pile at corresponding level. The variation in axial load obtained is shown in Fig. 16. As can be seen the axial load distribution clearly indicate the friction contribution of each of the layer and the end bearing load mobilized during each increment. This data can be further analyzed to get t–z curves and q–z curves, if required.

Lateral Deflections with Depth

An inclinometer tube, normally used for lateral movements in the ground has been used successfully to measure the lateral deflections of pile or even diaphragm walls subjected to lateral loads. Suresh [22] measured lateral deflections of a precast pre bored pile under lateral load. The deflected profile helps to understand the behavior of the pile (flexible or rigid behavior) and also permit to work back the modulus value of the surface soil.

Load Cells

Direct measurement of axial load in piles by embedding load cells or pressure pads covering the entire area of cross section of pile have been used successfully. While this method directly gives the axial load (eliminating complex analysis), it has difficulty in installation in cast-in situ piles as flow of concrete below the load cell is not possible.

Use of the above instrumentation with little additional cost enables the designer to evaluate parameters more precisely and optimize the foundation. The number of projects where pile instrumentation is used is increasing due to several benefits the measurements offer in optimizing the foundation.

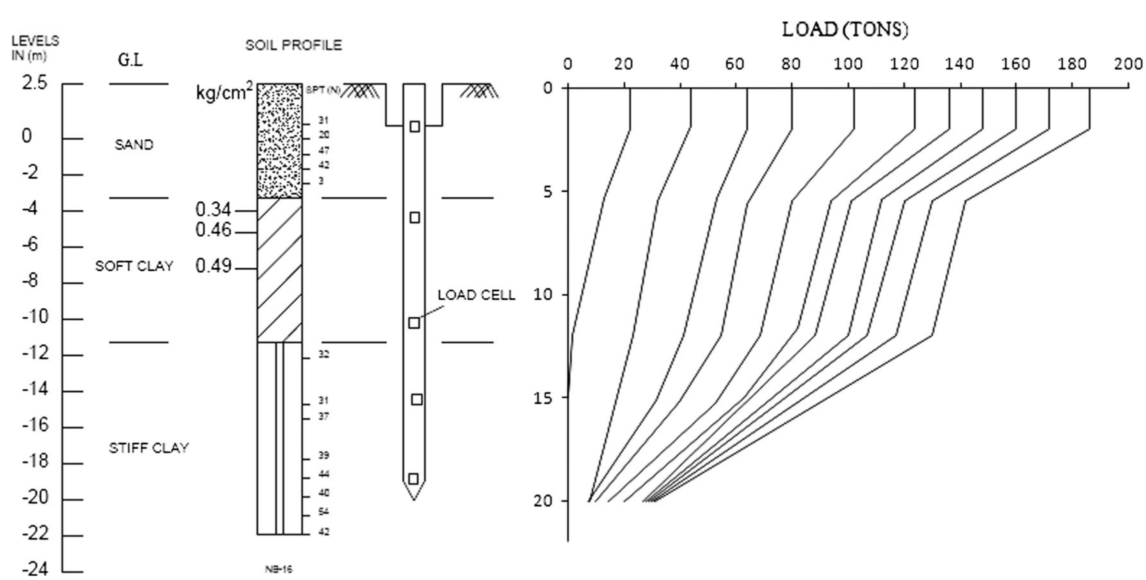


Fig. 16 Load distribution curves for instrumented pile [18]

Summary

The paper highlights the observations made over the last three decades on various aspects related to pile design, installation and testing. There have been considerable improvements in the piling equipment, construction procedures and field test methods. However, the present codes of practice do not cover a detailed guide lines on several aspects related to precautions in pile construction, evaluation of the negative drag with effective methods to reduce the same, pile group effect under horizontal load, evaluation of field test results to account for group effect or pile head fixity condition, etc. However, such field tests shall be carefully planned, executed and evaluated to account for the difference in the test condition and the actual field condition. The limitations in geotechnical investigation as well as in theory for pile design leads to more and more use of the instrumented field load test which shall be encouraged for an optimum design. In near future, it is expected that with availability of more field test data, the design procedure is improved with more realistic parameters.

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References

1. American Petroleum Institute (2000) Recommended practices for planning, designing and constructing fixed offshore platforms: working stress design, 21st edn. API recommended practice 2A-WSD (RP 2A-WSD), Washington, DC
2. Arun Prakash M (2011) Evaluation of axial load based on strain measurements in pile/RCC columns through instrumentation. M. Tech Dissertation, Department of Civil Engineering, IIT Madras
3. Broms B (1960) The lateral resistance of piles in cohesive soils. J Soil Mech Found Div ASCE 90(2):27–63
4. Cole KW, Stroud MA (1976) Rock socket piles at Coventry Point, Market Way, Coventry. Geotechnique 26(1):47–62
5. Coyle HM, Sulaiman IH (1967) Skin friction for steel piles in sand. J Soil Mech Found Div ASCE 93(6):261–278
6. Franke E (1988) Group action between vertical piles under horizontal loads. In: Van Impe WF (ed) Proceedings of 1st international geotechnical seminar on deep foundations on bored and augur piles. A. A. Balkema, Rotterdam, pp 915–918
7. Gandhi SR, Suresh PK, Raju VS (1988) Lateral load tests on large diameter bored pile and analysis. In: Proceedings of Indian geotechnical conference 1988 held at Allahabad, vol I, pp 373–377
8. Gandhi SR, Selvam S (1997) Group effect on driven piles under lateral loads. J Geotech Geoenviron Eng ASCE 123(8):702–709
9. Bureau of Indian Standards (IS): 14593 (2008) Design and construction of bored cast-in situ piles founded on rocks: guidelines, New Delhi
10. Bureau of Indian Standards (IS): 2911 (2002) Code of practice for design and construction of pile foundations, New Delhi
11. Kandaswamy MK (1979) Bitumen coating to reduce skin friction in piles. M.Tech. Dissertation, IIT Madras
12. Khare MG, Gandhi SR (2009) Shear resistance of bitumen-coated piles in sand. Proc Inst Civ Eng-Geotech Eng 162(GE6):303–310
13. Maharaj DK, Gandhi SR (2004) Non-linear Finite Element Analysis of piled raft foundations Geotechnical Engineering. Proc Inst Civ Eng 157(GE3):107–113
14. Matlock H, Reese LC (1960) Generalized solutions for laterally loaded piles. J Soil Mech Found Div ASCE 86(5):63–91
15. Muthukumaran K, Sundaravadivelu R, Gandhi SR (2007) Effect of dredging and axial load on a berthing structure. Int J Geo-Eng Case Hist 1(2):73–88
16. Narasimha Rao G, Gandhi SR (1995) Parametric study on pile drivability-selection of hammer and other components. Civ Eng J, The Institution of Engineers (India) 76(1):19–24
17. Raju VS, Gandhi SR (1988) Foundation problems on soft clay for a fertilizer plant. In: Proceedings of international conference on engineering problems of regional soils, August 11–15, 1988, Beijing, China
18. Raju VS, Gandhi SR (1989) Ultimate capacity of pre-cast driven piles in stiff clay. Indian Geotech J 19(4):273–289
19. Sivapriya SV, Gandhi SR (2013) Experimental and numerical study on pile behavior under lateral load in clayey slope. Indian Geotech J 43(1):105–114
20. Sivapriya SV, Gandhi SR (2014) Evaluation of pile behaviour on sloping clayey soil in supporting a diaphragm wall for building excavation. In: Proceedings of international conference TC207 on soil-structure interaction and retaining walls, June 16–18, 2014, Saint Petersburg, Russia
21. Sundaravadivelu R, Idichandy VG, Gandhi SR, Raju VS (1990) Tie-rod forces measurement in cargo berth. J Waterw, Port, Coast Ocean Eng ASCE 116(1):43–56
22. Suresh PK (1989) Analysis of piles subjected to lateral loads. M.S. Dissertation, IIT Madras

Prof. S. R. Gandhi obtained his B.Tech. Civil from Saurashtra University in 1977, M.Tech. in Soil Mechanics from IIT Madras in 1979 and Ph.D. on topic "Piles socketed in weak rock" from IIT Madras in 1985. He worked as faculty in Ocean Engineering Centre from 1982 and since 1990, he is a faculty member in Geotechnical Engineering Division in Department of Civil Engineering at IIT Madras. His research area includes design of pile foundation, ground improvement, field instrumentation and monitoring and design of ash pond. He has carried out three major Sponsored Projects funded by Department of Science and Technology on Densification of Fly Ash and Reinforced Earth Embankment. He is an active

Foundation Consultant to various Government organizations and Private industries for design and construction of foundation. He has served as Executive Member of IGS Delhi for two terms. He was the organizing chairman for the IGC 2006 and DFI India conference 2012 held at IIT Madras. He served as Chief Editor of Indian Geotechnical Journal during 2005–2010 and presently he is associate editor of SADHANA Journal.