INTERNATIONAL JOURNAL OF RESEARCH
In Advanced Engineering Technologies
Volume 6, Issue 2 AUG 2017

A STUDY OF PILE FOUNDATION TO ENHANCE SOIL BEARING CAPACITY FOR THE STRUCTURE

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Abstract: ***This paper presents a case study of piling foundation and testing of a Commercial Project in the south region of India. The overall pile driving works involved more than 1600RC piles of a total length over 25.000 m. Assumption that the bearing capacity of a pile driven into cohesive soil may increase significantly in time (set-up effect), was the reason for the contractor to take the risk to accelerate the testing procedure. Usually, when the load test result indicates insufficient bearing capacity, the testing procedure may be repeated after a period required by the codes of practice. The possible later increase of pile bearing capacity adds up to additional safety margin for the design.***

In the case of sandy soils, reported by Jardine et al (2006) values of capacity increase amounting to app. 20% do not affect much pile bearing capacity and the design procedure. It is important to state that some authors have observed an opposite effect called relaxation, which can appear in silty soil. The authors of the paper, however, have never noticed this effect. On the contrary, the numerous static and dynamic testing of foundation piles designed for Auchan Commercial Centre in Raciborz (Poland) have proved a significant time-dependent increase of bearing capacity of piles driven in silt (reaching app. 67%).

Keywords: pile load test, set-up.

1.1 INTRODUCTION

Pile foundations consist of piles that are dug into the soil till a layer of stable soil is reached. Pile foundations with unstable upper soil that may erode, or for large buildings

Pile foundations are used extensively for the support of buildings, bridges, and other structures to safely transfer structural loads to the ground and to avoid excess settlement or lateral movement. They are very effective in transferring structural loads through weak or compressible soil layers into the more competent soils and rocks below. A "driven pile foundation" is a specific type of pile foundation where structural elements are driven into the ground using a large hammer. They are commonly constructed of timber, precast pre stressed concrete (PPC), and steel (H-sections and pipes).

LIMITATIONS OF THE STUDY

In foundation practices, the main point of concern is bearing capacity of soil. Bearing capacity can be defined as the maximum load that can be carried by the soil strata. When the soil is strong enough that it can carry the whole load coming on it, then we use shallow foundation. Shallow foundations are usually used where hard soil strata is available at such a depth that construction of foundation is not too costly.

If hard soil is available at deeper levels of earth, then there is a need of some source that can transfer the load of the structures on the deep hard soil strata. This source can be said to be as the deep foundation. Pile foundation is a type of foundation in which pile is usually used as the source to transfer the load to deep soil levels. Piles are long and slender members that transfer the load to hard soil ignoring the soil of low bearing capacity. Transfer of load depends on capacity of pile. There is a need that pile should be strong enough to transfer the whole load coming on it to underlying hard strata. For this purpose, pile design is usually given much consideration.

OBJECTIVE OF THE PRESENT STUDY

The overall objective of this report is to document the lessons learned from the installation of driven piles in project. This includes review and analysis of pile design criteria and specifications, pile driving equipment and methods, issues encountered during construction, dynamic and static load test data, and cost data for different pile types and site conditions.

SCOPE OF THE PRESENT STUDY

1. Introductory and background information about the contracts where significant pile driving occur
2. The criteria and specifications used for pile design and construction in project.
3. The equipment and methods used for pile driving. Major construction issues encountered during driving, such as pile and soil heave, are also discussed.
4. Pile load tests performed on test piles using static and dynamic test methods, including a discussion of axial capacity, dynamic soil parameters, and pile driving criteria.
5. The unit costs for pile driving and preaugering for the different pile types used, as identified in the original construction bids.
6. Summarizes the important findings of this study.

LITERATURE REVIEW

Zeevaert et al., (1957) He was one of the earliest to prepare the concept of using deep foundation elements particularly piles to reduce the raft settlement. Such a combination of raft with piles was termed as piled raft. A notable early example of the use of settlement-reducing piles in foundation design was given by Zeevaert (1957) for La Azteca office building in Mexico City constructed during 1954-57. The foundation comprised of 41m × 16m raft with 83 concrete piles of length of 24m and diameter 0.4m. Partially compensated friction-pile foundation was adopted in the design. The calculated immediate raft settlement was about 200mm, compatible with the observed settlement at the end of construction and the predicted differential settlement was about 30mm. A comparison of observed and computed settlements was given in an attempt to predict the future behavior of the foundation of the building**.**

Whitaker et al., (1957 and 1961)conducted series of tests on 1g models of free standing pile groups and pile groups with rigid caps (piled raft) resting directly on a soil bed and examined the influence of number of piles, length of piles and spacing between piles on bearing capacity and settlement of pile group. The rigid cap of piled raft system showed block failure and efficiency of pile group was higher than 100% particularly for wider pile spacing. Since the failure was essentially block failure, a method was proposed for estimating bearing capacity of piled raft based on it. No differential settlement was reported since the raft used was relatively rigid**.**

Weisner and Brown et al., (1978) This research performed tests on piled raft models installed in over consolidated Kaolin clay bed and demonstrated the applicability of linear elastic continuum theory for predicting behavior of piled raft with both rigid and flexible rafts. In these experiments settlement of raft and strains in the longitudinal and transverse directions of the raft were measured at selected points using displacement transducers and semi conductor strain gauges. The settlements and moments thus measured for various vertical loads were compared. Satisfactory agreement were reported between the experimental results and results of elastic analyses extended from Hain (1975). Though the tests were conducted on rafts of two different shapes and different soil-raft stiffness (both flexible and rigid rafts), the role of pile group was not analyzed. Concentration was more on differential settlement and variation of moment in the raft rather than load sharing between the raft and piles.

Cooke et al., (1986) He conducted elaborate model tests on rafts (unpiled), free standing pile groups and piled rafts on over consolidated clay bed. The tests were conducted till the settlement was of the order of 1% of the width of the raft. Cooke established that very little advantage could be obtained by designing the piled raft with spacing lesser than 4d and also indicated that the block behavior occurred at even much wider spacing (i.e. 6d to 8d) than what was being traditionally accepted for piled raft design. Hence fewer piles also could produce considerable settlement reduction. However this observation needs to be strengthened either through detailed model studies or observations on real structures supported on small number of piles. Cooke also established that the ratio of length of the pile to width of the raft influenced the behavior. He indicated that the length of the pile must be such that the shaft friction should influence the settlement than the tip resistance, meaning that the settlement reducing piles must be friction piles.

PILE FOUNDATION DESIGN AND CONSTRUCTION

In actual construction, first pile load test is performed on the soil to verify soil strength that whether it can take the load of pile or not. Factors which affect the selection of pile are as under:

1. Length of pile in relation to load and soil condition
2. Behavior of structure
3. Availability of material in locality of construction
4. Type of loading
5. Ease of maintenance
6. Availability of funds
7. Factors causing damage
8. Cost of piles

LOAD CARRYING CAPACITY OF PILES

Usually three methods are used for the calculating the capacity of pile:

1. Static formula (this formula is applicable to driven and insitu piles)
2. Dynamic formula (this formula is applicable to driven piles)
3. Load Test (this test is usually done two times the design load. If it is stable then pile is considered O.K. This test is quite expensive and time consuming. Load application is also very much difficult in this test)

Two more things which have importance in pile foundation design are:

1. Pile spacing
2. Negative skin friction

While materials for piles can be precisely specified, and their fabrication and installation can be controlled to conform to strict specification and code of practice requirements, the calculation of their load-carrying capacity is a complex matter which at the present time is based partly on theoretical concepts derived from the sciences of soil and rock mechanics, but mainly on empirical methods based on experience. Practice in calculating the ultimate carrying capacity of piles based on the principles of soil mechanics differs greatly from the application of these principles to shallow spread foundations

The conditions which govern the supporting capacity of the piled foundation are quite different. No matter how the pile is installed, whether by driving with a hammer, by jetting, by vibration, by jacking, screwing or drilling, the soil in contact with the pile face, from which the pile derives its support by shaft friction, and its resistance to lateral loads, is completely disturbed by the method of installation. Similarly, the soil or rock beneath the toe of a pile is compressed (or sometimes loosened) to an extent which may affect significantly its end-bearing resistance (Figure 1.1b). Changes take place in the conditions at the pile–soil interface over periods of days, months or years which materially affect the skin-friction resistance of a pile.

These changes may be due to the dissipation of excess pore pressure set up by installing the pile, to the relative effects of friction and cohesion which in turn depend on the relative pile-to-soil movement and to chemical or electro-chemical effects caused by the hardening of the concrete or the corrosion of the steel in contact with the soil. Where piles are installed in groups to carry heavy foundation loads, the operation of driving or drilling for adjacent piles can cause changes in the carrying capacity and load/settlement characteristics of the piles in the group that have already been driven.

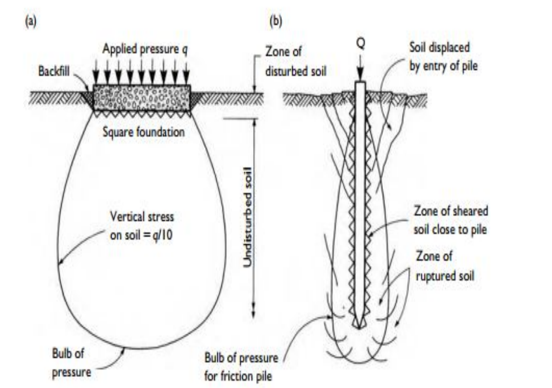


Figure 1.1 Comparison of pressure distribution and soil disturbance beneath spread and piled foundations (a) Spread foundation (b) Single pile.

The basis of the 'soil mechanics approach' to calculating the carrying capacity of piles is that the total resistance of the pile to compression loads is the sum of two components, namely shaft friction and base resistance. A pile in which the shaft-frictional component predominates is known as a friction pile (Figure 1.2a), while a pile bearing on rock or some other hard incompressible material is known as an end-bearing pile (Figure 1.2b).

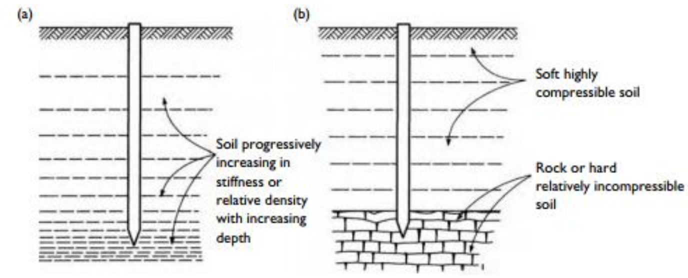


Figure 1.2 Types of bearing pile (a) Friction pile (b) End-bearing pile.

DYNAMIC PILING FORMULAE

The soil mechanics approach to calculating allowable working loads on piles is that of determining the resistance of static loads applied at the test-loading stage or during the working life of the structure. Methods of calculation based on the measurement of the resistance encountered when driving a pile were briefly mentioned in the context of history. Historically all piles were installed by driving them with a simple falling ram or drop hammer. Since there is a relationship between the downward movement of a pile under a blow of given energy and its ultimate resistance to static loading, when all piles were driven by a falling ram a considerable body of experience was built up and simple empirical formulae established from which the ultimate resistance of the pile could be calculated from the 'set' of the pile due to each hammer blow at the final stages of driving

Dynamic pile formulae 4 General principles and practices (a) (b) Soil progressively increasing in stiffness or relative density with increasing depth Rock or hard relatively incompressible soil Soft highly compressible soil Figure 1.

2 Types of bearing pile

(a) Friction pile

(b) End-bearing pile.

$$Q_{dy} = \frac{\alpha W_H H}{S + 0{,}5 S_e}$$

Where

$$S_e = \sqrt{\frac{2\alpha W_H HL}{AE}}$$

Qdy = ultimate dynamic bearing capacity of driven pile

∝=pile driving hammer efficiency

W_H = weight of hammer

H= Hammer drop

S = Inelastic set of piles, in distance prehammer blow

L=Pile length

A= pile end area

E=Modulus of elasticity of pile material

PILING EQUIPMENT AND METHODS

PILING FRAMES

The piling frame has the function of guiding the pile at its correct alignment from the stage of first pitching in position to its final penetration. It also carries the hammer and maintains it in position co-axially with the pile. The essential parts of a piling frame are the leaders or leads, which are stiff members of solid, channel, box, or tubular section held by a lattice or tubular mast that is in turn supported at the base by a moveable carriage and at the upper level by backstays. The latter can be adjusted in length by a telescopic screw device, or by hydraulic rams, to permit the leaders to be adjusted to a truly vertical position or to be raked forwards, backwards, or sideways.

The pile head is guided by a cap or helmet which has jaws on each side that engage with U-type leaders. The hammer is similarly provided with jaws. The leaders are capable of adjustment in their relative positions to accommodate piles and hammers of various widths. Self-erecting leaders on powerful hydraulic crawler carriages can be configured for a variety of foundation work. Initial erection and changing from drilling to driving tools can be rapidly accomplished and with the electronic controls now available the mast can be automatically aligned for accurate positioning. Some crawlers have expandable tracks to give added stability and can handle pile hammers with rams up to 12 tonne at 1:1 back rake.

Figure 3.1 ABI Mobilram with telescopic leader fully extended driving tubular pile (courtesy ABI GmbH).

Piling hammers

The simplest form of piling hammer is the drop hammer, which is guided by lugs or jaws sliding in the leaders and actuated by the lifting rope. The drop hammer consists of a solid mass or assemblies of forged steel, the total mass ranging from 1 to 5 tonne. The striking speed is slower than in the case of single- or double-acting hammers, and when drop hammers are used to drive concrete piles there is a risk of damage to the pile if an excessively high drop of the hammer is adopted when the driving becomes difficult. There has been a revival of interest in the simple drop hammer because of its facility to be operated inside a sound-proofed box, so complying with noise abatement regulations.

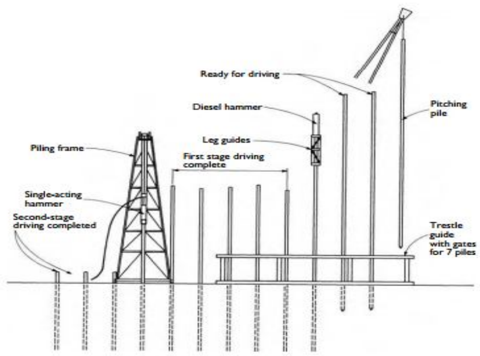


Figure 3.9 Driving piles in stages in conjunction with trestle guides.

PILE LOAD TESTING

Load tests on piles are conducted on completion of 28 days after casting of piles. Two types of tests namely initial and routine tests, for each type of loading viz. vertical, horizontal (lateral) pull out, are performed on piles.

INITIAL TESTS ON PILES

This test is performed to confirm the design load calculations and to provide guidelines for setting up the limits of acceptance for routine tests. It also gives an idea of the suitability of the piling system. Initial Test on piles are to be carried out at one or more locations depending on the number of piles required. Load applied for the initial (cyclic) load test is 2.5 times the safe carrying capacity of the pile. Loading for Initial Tests is conducted as per Appendix 'A' Clause 6.3of **IS-2911 Part IV**.

ROUTINE TESTS ON PILES:

Selection of piles for the Routine Test is done based on number of piles required subject to maximum of ½% of total number of piles required. The number of tests may be increased to 2% depending on the nature / type of structure. The test load applied is 1½ times the safe carrying capacity of the pile. **The Maintained load method** as described in Clause 6.2 of **IS-2911 (Part IV) – 1985** shall be followed for loading for the Routine Tests.

This test will be performed for the following purposes:

a) To ensure the safe load capacity of piles

b) Detection of any unusual performance contrary to the findings of the Initial Test.

The tests shall be performed at the cut-off level only.

VERTICAL LOAD TESTS ON PILES

This test will be carried out as stipulated in IS-2911 (Part IV) 1995. The maximum settlement at test load should not exceed 12 mm

Lateral Load Tests on Piles:

Fig: Horizontal load test on piles

The jack should be placed horizontally, between two piles. The load on the jack shall be the same on both the piles. The load will be applied in increments of 20% of the estimated safe load and at the cut off level. The load will be increased after the rate of displacement is nearer to 0.1 mm per 30 minutes. If the cut-off level is approachable, one dial gauge exactly at the cut-off level shall measure the displacement. In case the cut-off level is not approachable, 2 dial gauges 30 cm apart vertically, shall be set up and the lateral displacement of the cut-off level calculated by similar triangles

CALCULATING THE RESISTANCE OF PILES TO COMPRESSIVE LOADS

When a pile is subjected to a progressively increasing compressive load at a rapid or moderately rapid rate of application, the resulting load–settlement curve is as shown in Figure 4.1. Initially the pile–soil system behaves elastically. There is a straight-line relationship up to some point A on the curve and if the load is released at any stage up to this point the pile head will rebound to its original level. When the load is increased beyond point A there is yielding at, or close to, the pile–soil interface and slippage occurs until point B is reached, when the maximum shaft friction on the pile shaft will have been mobilized.

If the load is released at this stage the pile head will rebound to point C, the amount of 'permanent set' being the distance OC. The movement required to mobilize the maximum shaft friction is quite small and is only of the order of 0.3% to 1% of the pile diameter. The base resistance of the pile requires a greater downward movement for its full mobilization, and the amount of movement depends on the diameter of the pile. It may 140 Resistance of piles to compressive loads Resistance of piles to compressive loads 141 is in the range of 10% to 20% of the base diameter. When the stage of full mobilization of the base resistance is reached (point D in Figure 4.1) the pile plunges downwards without any further increase of load, or small increases in load produce increasingly large settlements. If strain gauges are installed at various points along the pile shaft from which the compressive load in the pile can be deduced at each level, the diagrams illustrated in Figure 4.2 are obtained, which show the transfer of load from the pile to the soil at each stage of loading shown in Figure 4.1.

Thus when loaded to point A virtually the whole of the load is carried by friction on the pile shaft and there is little or no transfer of load to the toe of the pile (Figure 4.2). When the load reaches point B the pile shaft is carrying its maximum frictional resistance and the pile toe will be carrying some load. At Point D there is no further increase in the load transferred in friction, but the base load will have reached its ultimate value.

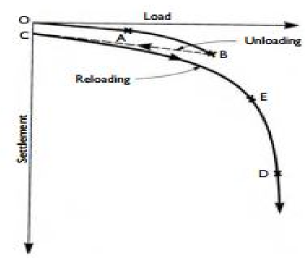


Figure 4.1 Load/settlement curve for compressive load to failure on pile.

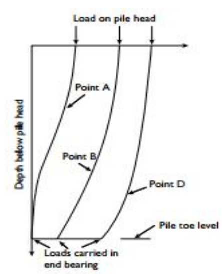


Figure 4.2 Load transfer from head of pile to shaft at points A, B and D on load/settlement curve in Figure 4.1.

The concept of the separate evaluation of shaft friction and base resistance forms the basis of all 'static' calculations of pile bearing capacity.

The basic equation is

$$Q_p = Q_b + Q_s - W_p$$

Where

Qp=ultimate resistance of the pile,

Qb = ultimate resistance of the base,

Qs = ultimate resistance of the shaft, and

Wp =weight of the pile.

6.1 DYNAMIC PILE LOAD TESTS .

This was carried out for the piles which had been installed from 4 to 6 days before. The measurements were repeated on the 55th day after the piles had been installed. Load capacity has been calculated on the basis of the pile penetration per blow and by means of dynamic load tests. Hammer energies used for the pile driving and for the dynamic load tests were very similar (the same hammer mass, falling height varying generally from 0.4 m to 0.6 m). The values of the load capacity of piles have been juxtaposed in Table 1.

Pile Number	Period	Pile length	Pile penetration per blow	Pile load capacity acc to sorensen and hansens formula	load capacity dynamic load tests
1	6days	12m	5t/0.5/8	720KN	616KN
2	6days	12m	5t/0.5/7	650KN	791KN
3	6days	12m	5t/0.5/5	402KN	776KN
4	6days	12m	5t/0.5/6	589KN	693KN
5	6days	12m	5t/0.5/6	589KN	817KN
6	6days	12m	5t/0.4/7	535KN	650KN
7	6days	12m	5t/0.4/9	658KN	612KN
8	6days	11m	5t/0.3/11	601KN	631KN
9	6days	11m	5t/0.4/9	666KN	654KN
10	6days	13m	5t/0.4/7	540KN	690KN

Figure 1: PILE TESTING IN 6 DAYS PERIOD

Pile Number	Period	Pile length	Pile penetration per blow	Pile load capacity acc to sorensen and hansens formula	load capacity dynamic load tests
1	55days	12m	5t/0.5/8	720KN	676KN
2	55days	12m	5t/0.5/7	650KN	842KN
3	55days	12m	5t/0.5/5	402KN	788KN
4	55days	12m	5t/0.5/6	589KN	889KN
5	55days	12m	5t/0.5/6	589KN	973KN
6	55days	12m	5t/0.4/7	535KN	795KN
7	55days	12m	5t/0.4/9	658KN	830KN
8	55days	11m	5t/0.3/11	601KN	645KN
9	55days	11m	5t/0.4/9	666KN	669KN
10	55days	13m	5t/0.4/7	540KN	863KN

Figure 2: PILE TESTING IN 55 DAYS PERIOD

LOAD CAPACITY INCREASE AND CORRECTION OF PILE DESIGN

The strata, which have the decisive impact on the pile load capacity and the change of the pile load capacity in time, were sands and gravels with the admixture of silts. The fill deposit above the bottom of the basin had no influence on the change of the pile load capacity in the considered time interval. Very soft aggregate muds at the bottom of the basin, with thickness of about 2.0 m, had no significant importance from the point of view of the pile load capacity and its change in time, either. The piles numbered from 1 to 15 show a gain of about 20% in the load capacity, in the period starting from about the 5th day until the 55th day after the piles had been driven in the ground. When we assume the formula given by Skov and Denver (1988), describing the change of the pile load capacity values in time, it is easy to predict the load capacities measured in the second series, 55 days after their driving.

$$Q/Q_0 - 1 = A \cdot \log_{10}(t/t_0)$$

Q – load capacity in time (t)

Q0 – the load capacity at the moment of the first test

A – the empirical constant

t – time elapsed from the moment of the pile installation

t_0 – the time of the first load capacity test.

CHECK OF PILE LOAD CAPACITY AFTER PILE DESIGN CORRECTION

On the lengthened piles and piles driven in silt (numbers 16–21), 13 days after installation, the load capacity was tested. The results have been presented in table below :

Pile Number	Period	Pile length	Pile penetration per blow	Pile load capacity acc to sorensen and hansens formula	load capacity dynamic load tests
11	45days	14m	5t/0.5/8	717	1291
12	13days	13m	6t/0.4/9	761	1459
13	13days	13m	5t/0.4/12	795	1189
14	13days	14m	6t/0.4/14	1002	1083
15	17days	14m	6t/0.4/6	558	1106
16	13days	14m	6t/0.4/7	628	838
17	5days	15m	5t/0.2/7	337	697
18	17days	15m	6t/0.2/8	374	786
19	45days	11m	5t/0.2/7	293	569
20	13days	12m	5t/0.5/8	733	1006
21	14DAYS	13m	6t/0.4/13	972	1369

Figure 3: results of piles in 13 days

The measured load capacity values varied between 570 kN and 1370 kN. The substantial discrepancy of load capacity values could be observed already at the time of pile installation. In line with Sorensen and Hansen's formula, the values ranging from 290 kN to 970 kN were obtained. No second series of measurements was carried out on those prolonged piles. It is possible, however, to venture predicting the pile load capacity, relying on the results obtained on the basis of pile penetration per blow values and the dynamic load tests capacity. Such prediction is by nature approximate, hence it was assumed, that the load capacity calculated on the basis of the values of the pile penetration per blow was, at the same time, the load capacity of a pile on the first day after driving. The exact analysis of the geology of particular piles was also abandoned. The average increase of the load capacity of piles number 16 to 21, in the period of 13 days after they had been installed, amounted to 68%. When we substitute A ¼ 0.6 in equation (1), taking into account the period from the first to the 13th day after the pile driving, we obtain a significant load capacity increase, reaching about 67%.

DESIGN OF PILE CAPS

DESIGN OF TWO PILE CAP

DATA:-

Pile Diameter	: 400 mm
Spacing of piles 2 hp	: 2 x 400 : 800 mm
Column Dimension B x D	: 300 x 450 mm
Factored Load	: 1072.8 KN
Factored Moment Mxu	: 51.29 KN.m
Safe Load on Single Pile	: 500KN
Concrete Mix	: M20
Steel Grade	: Fe 415

DESIGN : -

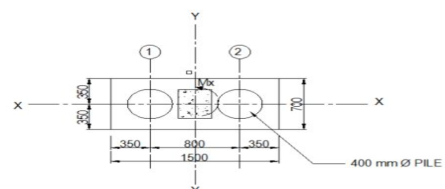
1. Pile Cap Dimension :

Breadth of Pile Cap = C/c of Pile + hp /2+ 150 +hp /2 + 150

= 800 + 400/2+ 150 +400/2+ 150 =1500mm

Width of pile cap = hp + 150 + 150 = 700 mm

Depth of Pile cap = 2 hp + 100 = 2 x 400 + 100 = 900 mm.



PLAN OF PILE CAP OF TWO PILE GROUP

Total factored axial compressive load

$$\frac{Pu}{n} \pm \frac{M_x y}{\sum y^2} \pm \frac{M_y}{\sum x^2}$$

Self weight of Pile Cap = (1.5 x0.7 x 0.9 x 25) x1.5

= 35.45 KN

Factored load from column Pu

= 1072.80 KN

Total Factored Load Pu = 1108.25 KN

No. of Piles along one side of axis = 2

y coordinate of Pile cap = 0.4 m

Mx = Moment about x axis = 51.29 KN.m

Compressive load in A1 & A2 about x – x axis

$= \frac{1108.25}{2} + \frac{51.29*0.4}{2*0.4^2}$

= 554.13 + 64.11

=618.24 KN

Design working load = 618.24 /1.5 = 412.16 KN < Safe Load on Pile i.e 500KN. O.K.

3. Bending Moment :-

Factored Moment in section Y-Y

$$Mu = 618.24 * \frac{(0.8 - 0.3)}{2} = 154.56 KN.m$$

4. Check for effective depth :

Mu = 0.138 fck bd² = 154.56 x 10^6

d required = √ (154.56 x 10^6) / 2.76 x700 =282.84 mm

D provided = 900 mm

d available = 900 – 60 -12- 6 = 822 mm > d required i.e. 282.84 mm

5. Check for Punching Shear (Two way shear) : -

Punching shear at a distance d/2 (i.e.822/2= 411mm) from face of column

= 1072.80 KN

The critical section of punching comes the centre of pile.

Hence the net load is to be taken. However the depth is checked for factored

axial load from column = 1072.80 KN

b= 700 x 822 mm

d= 822 mm

Perimeter of critical section = 2 (700 + 822) = 3044 mm

Punching shear stress $= \frac{1072.80*10^3}{3022*822} = 0.43\ N/mm^2$

Allowable shear stress for M20

= 0.25 √fck = 0.25 √20 = 1.12 N/mm²

Hence safe.

6. Main Reinforcement: -

Mu = 154.56 x 106 KN.m

K = Mu / bd2 = $\frac{154.56\ x\ 106}{700*822^2}$ = 0.33

Pt from Table 2 of Design Aid=0.11

Minimum Ast = $\frac{0.12}{100}$ x 700 x 822 = 690.48 mm²

Provide 7 Nos. 12 Φ RTS at bottom on both ways.

(Ast = 791 mm2 > 690.48 mm2)

Reinforcement at top :-

Minimum Ast = $\frac{0.12}{100}$ x 700 x 822 = 690.48 mm²

Provide 7 Nos. 12 mm Dia RTS at top .

(Ast = 791 mm² > 690.48 mm²)

Check for one way shear:-

Maximum Shear force at face of column = 618.24 KN

Shear stress = 618.24 x 103 = 1.07 N/mm2

700 x 822

For Pt = 0.20%

ζc from Table 61 of Design Aid to IS 456 -1978 = 0.33 N /mm2

Shear to be carried by stirrups shear

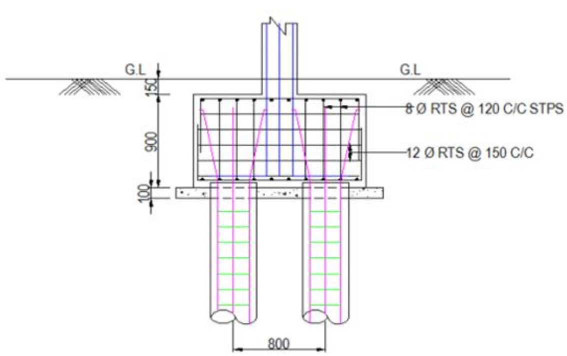
Vus =(1.07– 0.33) x700 x822 x 10-3= 425.80 KN.

Vus /d = 425.80 / 82.2 = 5.18 KN/cm

Provide 8 Φ RTS 4 legged stirrups @ 120 mm c/c.

(Vus /d =5.58 KN/cm > 5.18 KN/cm).

8. Sketch:



REINFORCEMENT DETAILS OF TWO PILE GROUP

CONCLUSIONS

1. The check of the mean values of pile load capacity made it possible to predict, with no great difficulty, the load capacity of the installed piles already at the time of their driving, relying on the values of the pile penetration per blow. Such rough and ready check, however, does not always bring about good results, therefore, it is indispensable to carry out control tests in order to identify the exact soil conditions, which is imperative from the point of view of the pile load capacity.
2. Here, the identification of soil conditions was essential for the proceeding of the pile work. Under the pressure of the pile work deadline, the values of pile load capacity were supposed to be predicted, in not well-characterized geological conditions and in a shorter time than required in regulations referring to pile load capacity tests.
3. At the same time, it turned out to be justified to estimate the lower values of load capacity increase, at least in the geological conditions which had not been identified with respect to the pile load capacity. Eventually, such all too optimistic estimation of pile load capacity led to the increase of the pile work cost.
4. It was required to enlarge the number of piles in places, where piles had been driven in before the dynamic load tests measurements. Due to the great number of piles according to the design, it was necessary to start the transportation of piles before the completion of the full geological survey. As a result, some of the piles had to be returned to the factory. Longer piles had to be delivered to the construction site, instead.
5. The observation of the values of pile penetration per blow, without load capacity measurements, was enough to fulfill the terms of the contract. Load capacity tests were carried out towards the end of the pile work, in order to comply with the formal pile work regulations.

REFERENCES

[1] Jardine R.J., Standing J.R., Chow F.C. (2006). ‘‘Some observations of the effect of time on the capacity of piles driven in sand.’’ Ge´otechnique, vol LVI, number 4.

[2] Skov R., Denver H. (1988). ‘‘Time dependence of bearing capacity of piles.’’ Proc. Third International Conference on the Application of Stress-Wave Theory to Piles. Ottawa, 25–27 May, pp 879–888.

[3] Australian Piling Standard, 1995. Piling design and installation. Standard AS2159-1995, Australian Council of Standards, Committee CE/18, 53 p.

[4] BUSTAMANTE, M. and GIANESELLI, L., 1982. Pile bearing capacity predictions by means of static penetrometer CPT. Proceedings of the Second European Symposium on Penetration Testing, ESOPT II, Amsterdam, May 24-27, A. A. Balkema, Rotterdam, Vol. 2, pp. 493-500.

[5] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, CFEM, Fourth Edition. BiTech Publishers, Vancouver, 488 p

[6] DERUITER, J. and BERINGEN, F. L., 1979. Pile foundation for large North Sea structures. Marine Geotechnology, 3(3) 267-314.

[7] ESLAMI, A. and FELLENIUS, B.H., 1997. Pile capacity by direct CPT and CPTu methods applied to 102 case histories. Canadian Geotechnical .Journal, 34(6) 886-904.

[8] FELLENIUS, B.H., 1984. Negative skin friction and settlement of piles. Proceedings of the Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18p.

[9] FELLENIUS, B.H., 1989. Tangent modulus of piles determined from strain data. The ASCE Geotechnical Engineering Division, 1989 Foundation Congress, Edited by F. H. Kulhawy, Vol.1, pp. 500-510.

[10] FELLENIUS, B.H., 2004. Unified design of piled foundations with emphasis on settlement analysis. Honoring George G. Goble — Current Practice and Future Trends in Deep Foundations, Proceedings of Geo-Institute Geo TRANS Conference, Los Angeles, Edited by J.A. DiMaggio and M.H. Hussein. ASCE GSP 125, 253-275.

[11] FELLENIUS, B.H., 2006. Results from long-term measurement in piles of drag load and downdrag.

Canadian Geotechnical .Journal .43(4) 409-430.