DESIGN OF PILE FOUNDATION FOR MULTI-STOREY BUILDING

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DEPARTMENT OF CIVIL ENGINEERING

MAY - 2009

Submitted in partial fulfillment of the Degree of Bachelor of Technology

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JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

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CERTIFICATE

This is to certify that the work entitled, " **Design of Pile Foundations for Multi-storeyed Buildings"** submitted by **Adarsh Thakur** and **Siddharth Singh**, in partial fulfillment for the award of degree of Bachelors of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under our supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

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Acknowledgement

We would like to take this opportunity to thank our reputed **Jaypee University of Information Technology**, Waknaghat, Solan, Himachal Pradesh and Honorable Dean **Prof. (Dr.) Rudra Mani Vasan** and respected teachers for giving us the opportunity to do project on DESIGN OF PILE FOUNDATION FOR MULTI-STOREY BUILDING so as to develop our understanding of foundation subject.

I would then like to thank **Dr. Subodh K Jain** (Project Guide) and **Mrs. Poonam Dhiman** (Project Co-Guide) for supervising and helping us during our project work & without whose able guidance, support and continuous motivation, the project would not have been carried to perfection.

The successful compilation of final year project depends on the knowledge and attitude inculcated in the total length of the course. So we also express our sincere gratitude to all the faculties who taught us during the four years of B.Tech.

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ABSTRACT

In many areas of engineering, the conditions of the soil and strata sometimes do not allow the laying of foundation by conventional methods. In such cases and if economy allows, pile foundations are the best solution. In the present work, pile foundations have been designed for a four storey building standing on a sandy stratum. The foundation design has been accomplished by IS code method, as provided in the *Indian Standard Code of Practice 2911* for *Design & Construction of Pile Foundation* part 1 Concrete Piles. The deflection in piles has been calculated in accordance with the *method of fixity*.

List of Figures and Tables

List of Abbreviations and Symbols

Symbol	Meaning
DL	Dead load
LL	Live Load
EL	Earthquake load/seismic load
WL	Wind Load
A_{st}	Area of tension reinforcement
f_y	Characteristic strength of steel
f_{ck}	Characteristic compressive strength of concrete
e_x	Eccentricity along X axis
e_z	Eccentricity along Z axis
Q_u	Ultimate load bearing capacity of a pile
Q_{pu}	Point load bearing capacity of a pile at the base
Q_f	Friction load bearing capacity of a pile
A_p	Cross-sectional area of the pile toe in cm^2
γ	Effective unit weight of soil
$N\gamma$ & N_q	Bearing capacity factors depending on the angle of internal friction
R	Depth of fixity
Q_m	Load on the end pile

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CHAPTER -1
INTRODUCTION

1.1 Role of foundation engineering

Foundation engineering is concerned directly with the design of the substructure such as resulting soil stability and estimated deformations are tolerable and within codal provisions.

From practice consideration following points have to be considered carefully:

a) Forces and moments acting on the superstructure and their transfer to the substructure elements.
b) Fixity conditions and various tolerance (i.e. settlements, lateral displacement and tilt) keeping in view the behavior of the superstructure.
c) Identification of soil/rock parameters need in their design, their evaluation from an adequate field exploration and testing programme.

1.2 General Requirements of Substructure

1. The substructure should be properly located with respect to any future influence which could adversely affect its performance.
2. The substructure elements must be safe against overturning, rotation, sliding or soil rupture.
3. Total and differential settlements should be tolerable for both substructure and superstructure elements.

Interpretation of the data obtained through soil exploration generally provides information to:-

a) The type of foundation
b) Safe load capacity of the foundation
c) Settlement of the foundation
d) Location of the ground water
e) Cause of settlement, tilt and cracks in the existing structure.
f) Identification and solution of the excavation problems

1.3 Pile Foundation

The first type of deep foundation is a pile foundation, which consists of long, slender, prefabricated structure members driven or inserted into the ground. Engineers use piles both on land and in the sea to support many kinds of structures. Piles are made from variety of materials and in different diameters and length according to the needs of each project.

Pile transfers the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity.

When to use Pile Foundation

Pie foundation is generally used in place of shallow foundation when :-

a) Upper soils are weak, structure loads are high.
b) Upper soils are subjected to scour or undermining.
c) Foundation must penetrate through water.
d) Need large uplift capacity.
e) Need large lateral load capacity.

1.4 History

Pile foundations have been used as load carrying and load transferring systems for many years. In the early days of civilization, from the communication, defense or strategic point of view villages and towns were situated near to rivers and lakes. It was therefore important to strengthen the bearing ground with some form of piling. Timber piles were driven in to the ground by hand or holes were dug and filled with sand and stones. In 1740 Christopher Polhem invented pile driving equipment which resembled to days pile driving mechanism. Steel piles have been used since 1800 and concrete piles since about 1900. The industrial revolution brought about important changes to pile driving system through the invention of steam and diesel driven machines.

More recently, the growing need for housing and construction has forced authorities and development agencies to exploit lands with poor soil characteristics. This has led to the development and improved piles and pile driving systems. Today there are many advanced techniques of pile installation.

1.5 Purpose of Pile Foundation

The purpose of pile foundation is to transfer a foundation load to a solid ground and to resist vertical and uplift load acting on the foundation.

A structure can be founded on piles if the soil immediately beneath its base does not have adequate bearing capacity, if the results of the site investigation show that the shallow soil is unstable and weak or if the estimated settlement is not acceptable, a pile foundation may be considered. Further a cost estimated may indicate that a pile foundation may be cheaper than any other compared ground improvement costs.

In case of heavy construction, it is likely that the bearing capacity of shallow soil will not be satisfactory and the construction should be built on the pile foundation. Piles are convenient method of foundation for works over water, such as jetties or bridge piers.

1.6 Type of Piles

Most piles are now made from timber, concrete or steel. Each material has its own advantages and disadvantages and is best suited for certain applications.

Timber Piles

Used from earliest record time and still used for permanent works in regions where timber is plentiful. Timber is most suitable for long cohesion piling and piling beneath embankments. The timber should be in a good condition and should not have been attacked by insects. For timber piles of length less than 14 meters, the diameter of the tip should be greater than 150 mm. If the length is greater than 18 meters a tip with a diameter of 125 mm is acceptable. It is essential that the timber is driven in the right direction and should not be driven into firm ground. As this can easily damage the pile. Keeping the timber below the ground water level will protect the timber against decay and putrefaction. To protect and strengthen the tip of the pile, timber piles can be provided with toe cover. Pressure creosoting is the usual method of protecting timber piles.

Concrete Piles

Pre cast concrete Piles or Pre fabricated concrete piles : Usually of square, triangle, circle or octagonal section, they are produced in short length in one meter intervals between 3 and 13 meters. They are pre-caste so that they can be easily connected together in order to reach to the required length. This will not decrease the design load capacity. Reinforcement is necessary within the pile to help withstand both handling and driving stresses. Pre stressed concrete piles are also used and are becoming more popular than the ordinary pre cast as less reinforcement is required.

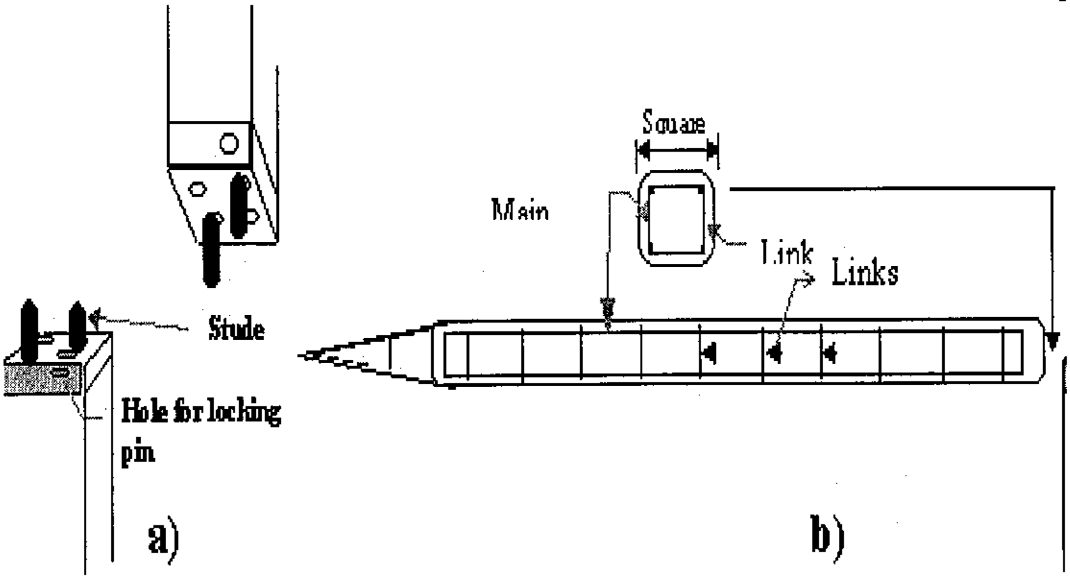


fig. 1.1 concrete piles

Steel Piles

Steel piles: Steel/ Iron piles are suitable for handling and driving in long lengths. Their relatively small cross-sectional area combined with their high strength makes penetration easier in firm soil. They can be easily cut off or joined by welding. If the pile is driven into a soil with low pH value, then there is a risk of corrosion, but risk of corrosion is not as great as one might think. Although tar coating or cathodic protection can be employed in permanent works.

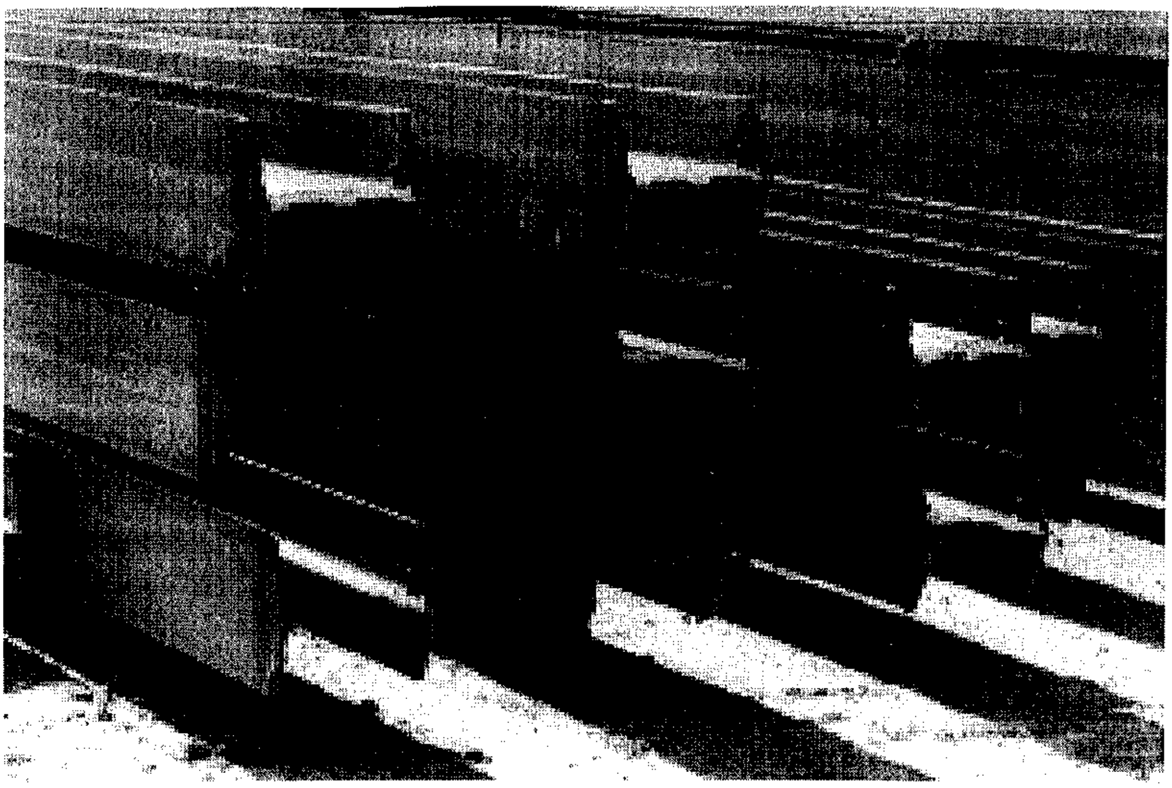


fig. 1.2 steel piles

Composite Piles

Combination of different materials in the same of pile. As indicated earlier, part of a timber pile which is installed above ground water could be vulnerable to insect attack and decay. To avoid this, concrete or steel pile is used above the ground water level, whilst wood pile is installed under the ground water level

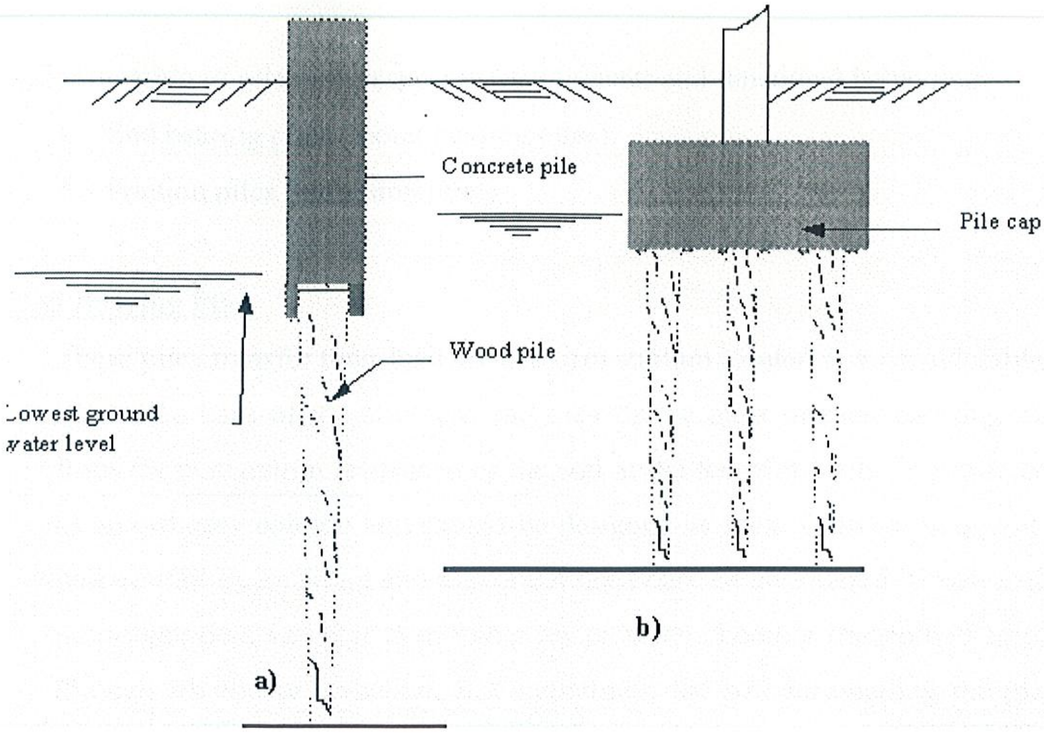


fig. 1.3 composite piles

Protecting timber piles from decay.

a) By pre-cast concrete upper section above water level.

b) By extending pile cap below water level.

Following factors are also considered while the selection of pile types such as:

a) The Applied Load: Some piles, such as timber are best suited for low to medium loads whereas other such as steel, may be cost effective for heavy loads.

b) The Required Diameter: Most pile types are available only in certain diameter.

c) The Required Length: Highway shipping regulations and practical pile driver heights generally limit the segments to about 18m (60ft). therefore longer piles must consist of multiple segments spliced together during driving.

d) The Local Availability of each pile type: Some pile Types may be abundant in certain geographical areas whereas other may be scarce. This can significantly affect the cost of each pile type.

e) The durability of pile material in a specific environment

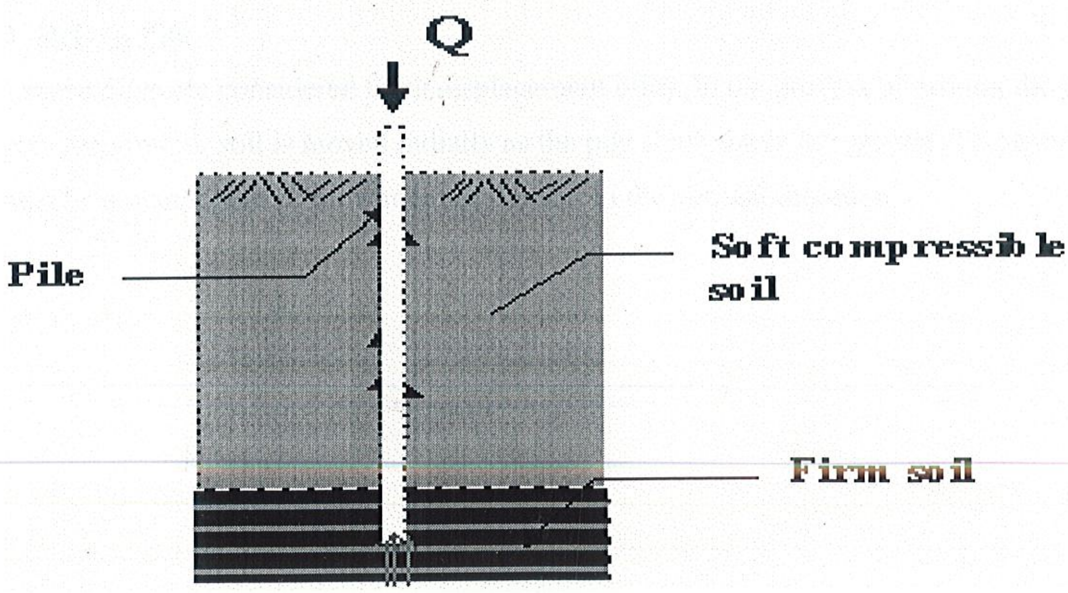
f) The anticipated driven conditions: Some piles tolerate hard driving while others are more likely to be damaged.

Classification of pile with respect to load transfer and functional behaviour:

- End bearing piles (point bearing piles)
- Friction piles (cohesion piles)

End Bearing Piles

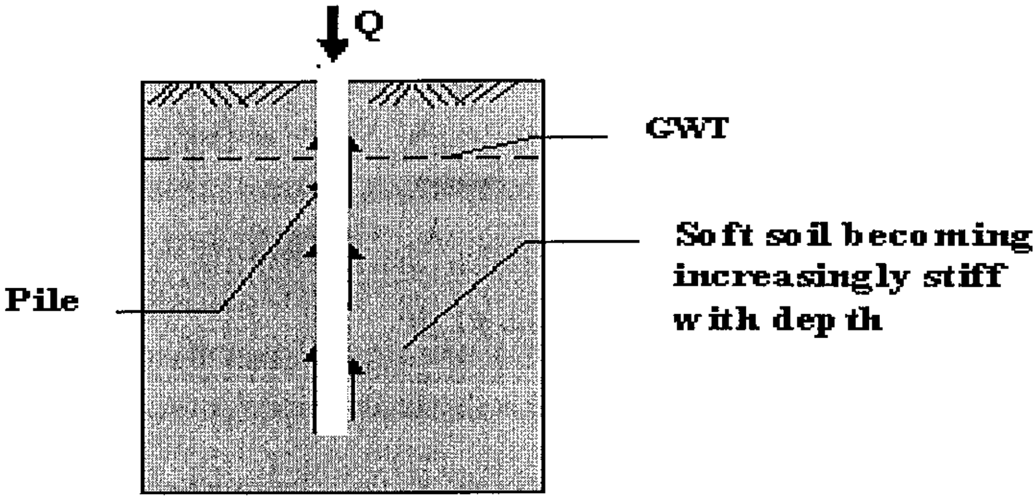
These piles transfer their load on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile. The pile behaves as an ordinary column and should be designed as such. Even in weak soil a pile will not fail by buckling and this effect need only be considered if part of the pile is unsupported, i.e. if it is in either air or water. Load is transmitted to the soil through friction or cohesion. But sometimes, the soil surrounding the pile may adhere to the surface of the pile and causes "Negative Skin Friction" on the pile. This, sometimes have considerable effect on the capacity of the pile. Negative skin friction is caused by the drainage of the ground water and consolidation of the soil. The founding depth of the pile is influenced by the results of the site investigate on and soil test.



1.4 End Bearing Pile

Friction Pile

These piles also transfer their load to the ground through skin friction. The process of driving such piles does not compact the soil appreciably. These types of pile foundations are commonly known as floating pile foundations



1.5 friction Pile

Classification of pile with respect to effect on the soil

1. Driven Pile

Driven piles are considered to be displacement piles. In the process of driving the pile into the ground, soil is moved radially as the pile shaft enters the ground. There may also be a component of movement of the soil in the vertical direction.

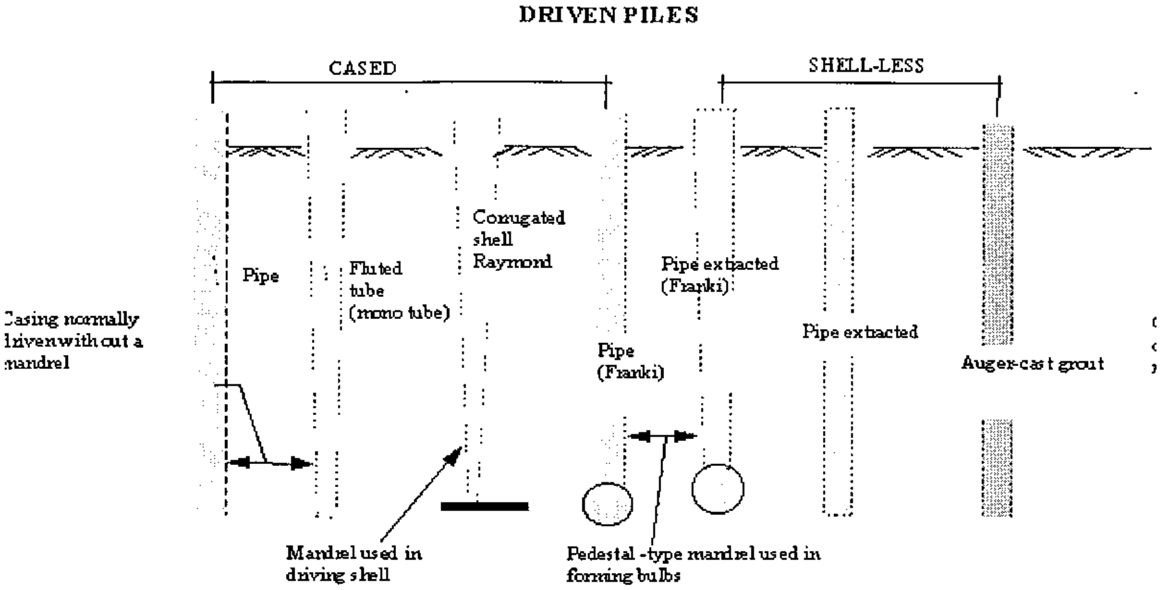


fig 1.6 driven piles

2. <u>Bored Pile</u>

Bored piles (Replacement piles) are generally considered to be non-displacement piles a void is formed by boring or excavation before piles is produced. Piles can be produced by casting concrete in the void. Some soils such as stiff clays are particularly amenable to the formation of piles in this way, since the bore hole walls do not requires temporary support except cloth to the ground surface. In unstable ground, such as gravel the ground requires temporary support from casing or slurry. Alternatively the casing may be permanent, but driven into a hole which is bored as casing is advanced. A different technique, which is still essentially non-displacement, is to intrude, a grout or a concrete from an auger which is rotated into the granular soil, and hence produced a grouted column of soil.

1.7 Grouping of Piles

Most of pile foundations consist not of a single pile, but of a group of piles, which act in the double role of reinforcing the soil, and also of carrying the applied load down to deeper, stronger soil strata.

The pile group is often joined at the ground surface by a concrete slab such as pile cap sufficient spacing is providing while grouping the piles. A minimum number of three piles is used under a column of triangular patter even if the load does not warrant the use of three piles. The spacing of piles depend upon many factors such as:-

- Overlapping of stresses of adjacent piles
- Cost of foundation
- Efficiency of pile group
- Type of Soil

1.8 Pile Cap

The pile cap distributes the applied load to the individual piles which in turn, transfer the load to the bearing ground.

The stiffness of pile cap will influence the distribution of structural loads to the individual piles. The thickness of pile cap must be at least four times the width of an individual pile to cause a significant influence on the stiffness of the foundation. The individual piles are spaced and connected to the pile cap or tie beams and trimmed in order to connect the pile to the structure at cut-off level

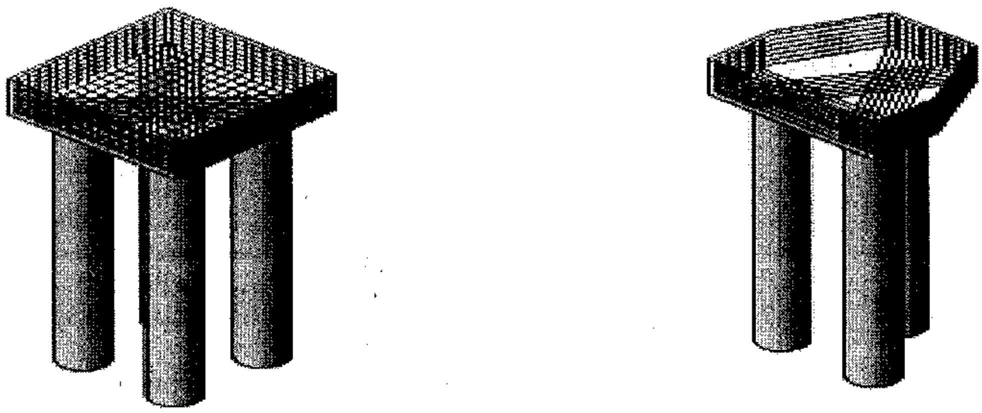


fig.1.7 pile cap

CHAPTER-2

PROBLEM FORMULATION

General

In this chapter, the buildings data have been described. The loads taken from various parts of IS-875 have been imposed on the building and ultimate loads coming on beams from slabs are calculated. Seismic coefficient method will be applied to calculate base-shear due to earthquakes, as per IS-1893:2002. Wind loads are also applied separately as per IS-875-part-III.

2.1 Building Parameters

No. of storey	=	4
Slab Thickness	=	115 mm
Dimensions	=	(15m × 15 m)
Clear cover: Slabs	=	20 mm
Beams	=	25 mm
Grade of concrete to be used	=	M25
Steel used :	=	Fe 415 (IS-1786:1985)
Beams :		
Dimensions:	=	(300mm × 450mm)
No. of Beams	=	96
Length of Beams	=	5 m
Columns :		
Dimensions:	=	(600mm × 600mm)
No. of Columns	=	64
Height Of column	=	3.5 m

Flooring finish = 25 mm thick flooring

Type of foundation desired: = straight pile foundation

Grade of concrete used in pile foundation = M25

2.2) METHODOLOGY

- For the four-storey building, the analysis will be performed and the design will be done for the following loads:
  1. Dead load
  2. Live load
  3. Earthquake load
  4. Wind load

- The dead load will be worked out by taking the provided thickness for the slab and then the actual thickness will be accordingly provided after calculating the required value. The load due to the flooring – screed, finishes, tiles etc. will be given due consideration and an allowance will be made for future erection of partitions.

- The live loads considered will be due to the imposed loads in case of educational buildings, as per the specifications of IS-875 part II - 1981 for Design loads (live loads) .

- Due to increased emphasis being laid on the design of earthquake resistant structures nowadays, the earthquake forces will be estimated with the help of the provisions of the revised Seismic Code (IS:1893).

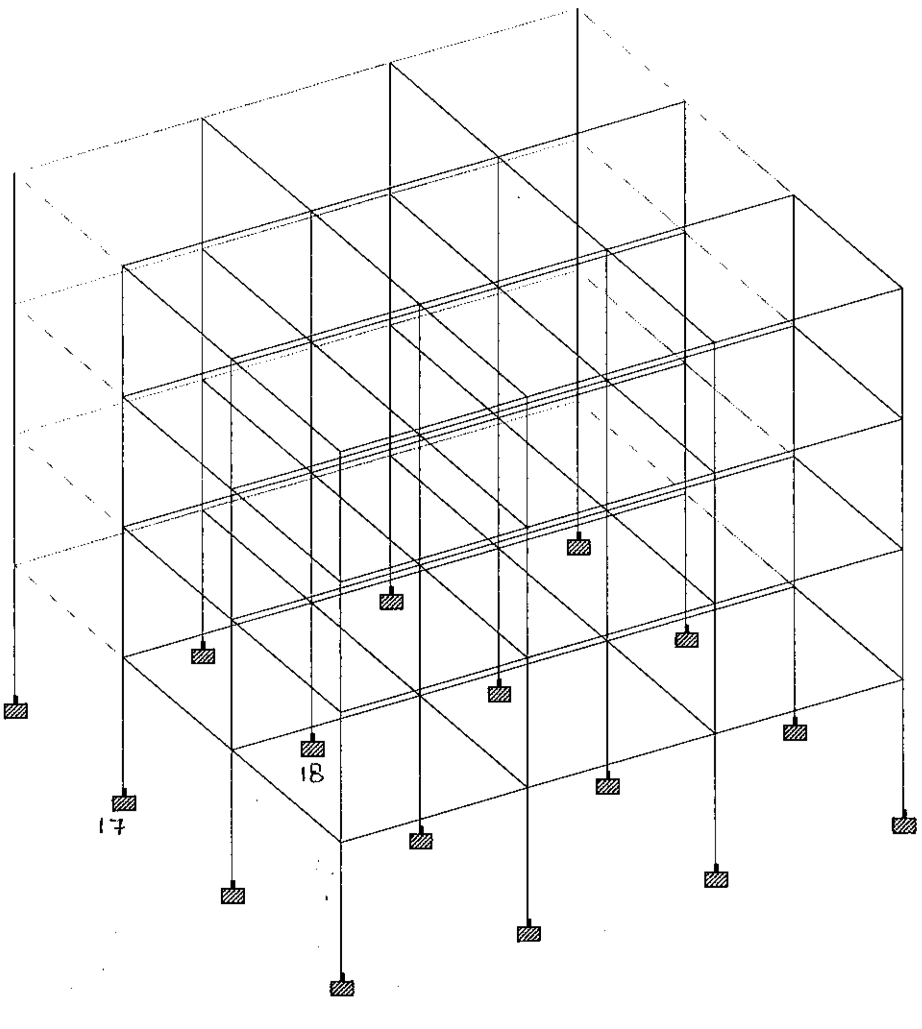
- The proposed building will lie in Zone IV. The value of the importance factor assigned to the entire structure was 1.5.

- The load will initially applied to the slabs and through trapezoidal distribution it will be transmitted to the columns via beams (longitudinal and transverse), and consequently to the foundations.

- The designing will be done after analyzing the structure for the above-mentioned loads – individually and for different load combinations recommended in the code.

The members will be designed by the Limit State method, according to the guidelines prescribed by IS: 456-2000.

CHAPTER-3
CALCULATION OF LOADS



3.1 Structure for which foundation is to be designed

3.1 Calculations for Dead Load

Thickness of the roof slab = 115mm

Dimensions = 15m x 15m

Total volume of reinforced slab = 115x15000x15000

$= 2.5875 \times 10^{10}$ mm^3

$= 25.875$ m^3

unit wt. of r/f cement concrete with 5% steel = 26.50 kN/m^3

[refer table-1 of [1]]

hence load of the roof slab = 25.875 x 26.50

= 685.68 kN

1) Thickness of the floor slab = 125 mm

Dimensions = 15m x 15m

Total volume of reinforced slab = 125x15000x15000

$= 2.5875 \times 10^{10}$ mm^3

$= 28.125$ m^3

unit wt. of r/f cement concrete with 5% steel = 26.50 kN/m^3

[refer table-1 of [1]]

hence load of the roof slab = 28.125 x 26.50

= 745.3125 kN

hence load for 3 floor slabs = 745.3125 x 3 = 2235.9375 kN

2) Total number of columns = 64

Dimensions = 0.6m x 0.6m

Height of the column = 3.5 m

Volume of single column = 1.26 m^3

unit wt. of r/f cement concrete with 5% steel = 26.50 kN/m^3

[refer table-1 of [1]]

hence load of a single column = 1.26 x 26.50

= 33.39 kN

hence load of 64 columns = 64 x 33.39

= 2136.96 kN

3) Total number of beams = 96

Dimensions = 0.3m x 0.45m

length of the beam = 5 m

Volume of single beam = 0.675 m^3

unit wt. of r/f cement concrete with 5% steel = 26.50 kN/m^3

[refer table-1 of [1]]

hence load of a single beam = 17.8875 kN

hence load of 96 beams = 96 x 17.8875 kN

= 1717.2 kN

Total dead load = 685.68 + 2235.9375 + 2136.96 + 1717.2

=6775.78 kN

3.2 Calculations for imposed loads

Dimensions of the roof slab = 15 x 15 m

Dimensions of the floor slab 15 x 15 m

Imposed floor loads = 2.5 kN/m^2 [refer table-1 of [2]]

Imposed roof loads =1.5 kN/m^2 [refer table-1 of [2]]

Load on the roof slab = 15 x 15 x 1.5

= 337.5 kN

Load on the floor slab = 15 x 15 x 2.5

= 562.5 kN

Load on the 3 floor slab = 3 x 562.5

= 1687.5 kN

Total imposed loads = 337.5 + 1687.5

= 2025 kN

3.3 Calculation for wind load

Design wind speed (V_z)

$V_z = V_b \, k_1 \, k_2 \, k_3$

Basic wind speed V_b

k_1 = probability factor

k_2 = terrain, height and structure size factor

k_3 = topography factor [refer article-5.3 of [3]]

value of k_1 for V_b (39m/s) is = 1 (for all general buildings and structures)

[refer, table-1, fig.-1[3]]

k_2 for category terrain for class A structure of height 14m = 0.96

[refer, table-2 of [3]]

k_3 for slope greater than 17° = 1.36

[refer, appendix C, article C-2 of [3]]

$V_z = 39 \times 1 \times 0.96 \times 1.36$

= 50.92 m/s

Design wind pressure

$P_z = 0.6 \, V_2^2$

= 1555.71 N/m^2 [refer, article-5.4 of [3]]

Design wind pressure = 1555.71 N/m^2

3.4 Calculation of earth quake loads

Design seismic base shear: the total design lateral force or design seismic base shear (V_b) along any principle direction shall be determined by the following expression:

$V_b = A_h W$ [refer, article-7.5.3 of [4]]

$A_h = Z\, I\, S_a/2\, R\, g$ (design horizontal seismic coeff.)

[refer, article-6.4.2 of [4]]

W = seismic weight of the building

Zone factor Z = 0.24 (zone 4) [refer, annex-E of [4]]

I = 1 [refer, table-6 of [4]]

R = 5 (for SMRF, ductile detailing of r/f to be done as per IS:13920 after design) [refer, table-7 of [4]]

$S_a/g = 1/T$ (for rocky foundation) [refer, fig-3 of [4]]

$T = 0.075\, h^{0.75} = 0.5428$ (for rocky foundation & h= 14m)

$A_h = 0.04421$

Seismic weight W of the building

For middle frame $W_{mf} = 2082.805$ kN

$(V_b)_{mf} = A_h W_{mf} = 0.04421 \times 2082.805$

$= 92.08$ kN

For end frame $W_{ef} = 1396.525$ kN

$(V_b)_{ef} = A_h W_{ef} = 0.04421 \times 1396.525$

$= 61.74$ kN

Q_i = design lateral force at floor i

$Q_i = (V_b W_i h_i^2)/\Sigma W_j h_j^2$

W_i = seismic weight of floor i

h_i = height of the floor measured from the base

n = number of storeys in the building

Table 1

LATERAL FORCES ON MIDDLE FRAME			
floors	h_i (m)	W_i (kN)	Q_i (kN)
ground	0	0	
1^{st}	3.5	554.085	3.52
2^{nd}	7	554.085	14.09
3^{rd}	10.5	554.085	31.72
4^{th}	14	420.55	42.77

Table 2

LATERAL FORCES ON END FRAME			
floors	h_i (m)	W_i (kN)	Q_i (kN)
E ground	0	0	
1^{st}	3.5	370.655	2.34
2^{nd}	7	370.655	9.39
3^{rd}	10.5	370.655	21.14
4^{th}	14	284.56	28.85

Table 3: Maximum support reactions for pile cap and piles

	node	L/C	horizontal FX(kN)	Verticle FY(kN)	Horizontal FZ(kN)	moment MX(kNm)	MY(kNM)	MZ(kNM)
Max FX	2	1.5(DL-WL)	43.240	366.257	-0.789	-0.997	1.668	79.799
Min FX	2	0.9DL+ 1.5WL	-40.308	168.193	-0.311	-0.273	-1.665	76.360
Max FY	1	1.5(DL+ LL)	0.489	$1.96*10^3$	0.489	0.534	0	-0.534
Max FY	1	EL in X direction	-15.879	-37.008	-0.470	-0.530	0.002	55.199
Max FZ	17	1.5(DL-WL)	-0.789	366.257	43.240	79.799	-1.668	0.997
Min FZ	17	0.9DL+ 1.5WL	-0.311	168.193	-40.308	-76.360	1.665	0.273
Max MX	18	1.5(DL-WL)	0.024	368.702	28.095	80.061	-0.765	-0.058
Min MX	18	0.9DL+ 1.5WL	0.014	222.719	-28.056	-79.970	0.765	-0.034
Max MY	2	1.5(DL+ EL)	-14.175	295.240	-0.657	-0.751	2.578	56.833
Min MY	3	1.5(DL+ EL)	-14.175	295.240	0.657	0.751	-2.578	56.833
Max MZ	17	1.5(DL+ EL)	-28.610	335.012	1.835	2.165	1.814	88.252
Min MZ	33	1.5(DL-EL)	28.610	335.012	1.835	2.165	-1.814	-88.252

3.4 Design of pile cap

L = B = S + D + 2 x assumed overhang of 150mm

For the pile driven in sandy soil, the group efficiency can be 100%

$100 = [2(m+n-2)S + 4D] / \pi D\, m\, n \times 100$

Spacing of piles

S = (3.14 x 250 x 4 -4 x 250)/ 4

= 250 x 2.14

= 435 mm $< S_{min}$

< 3d or 900mm = 750mm

(whichever is greater)

Hence S = 900mm

L = B = 900 + 250 + 300

= 1450 mm

Depth of pile cap

(a) Depth of pile based on moment

$Bd^2 = M_{z\,max} / [0.36\, f_{ck}(x_u/d)(1 - 0.416x_u/d)]$

$M_{z\,max} = P_u\,(s-1)/2$

P_u = load factor [(column load + assumed self wt. of 0.1P of pile cap)/ no. of piles + $MZ_c/\Sigma Z_c^2$]

=210 kN

$M_{z\,max} = 2 \times 210 \times (0.9 - 0.6)/2$

= 63 kN

x_u/d = 0.48 for steel of grade Fe 415

$1450\, d^2 = 63 \times 10^6 / [0.36 \times 20 \times 0.48\, (1 - 0.42 \times 0.48)]$

d = 125.38 mm

(b) Depth of pile based on one way slab

$V_{max} = \tau_{uc}\, bd$

$d = V_{max} / \tau_{uc}\, b$

$V_{max} = 2 \times 210 = 420$ kN

$d \quad = 420 \times 1000 / (0.36 \times 1450)$

$\quad = 804.5$ mm

(c) For 2-way slab

$V_{max} = 1.5 \times 335.01$ (based on punching of column)

$\quad = 502.518$ kN

$V_{max} = 210$ (based on punching of piles)

$\tau_{uc} = 0.25 \times \sqrt{f_{ck}} = 0.25 \sqrt{25}$

$\quad = 0.25 \times 5 = 1.25$ N/mm^2

$B = 4 \times (600 + d)$ (for punching column)

$B = 4 \times (250 + d)$ (for punching pile)

$d = (502.518 \times 1000) / [1.25 \times 4 (600 + d)]$

$d = 615$ mm (for punching column)

$d = 365.05$ mm (for punching pile)

therefore $d = 650$ mm

3.5 Ultimate load bearing capacity of an individual pile

The strata are poorly graded sand till a depth of 6m & homogeneous dense sand for the next 4m.

The base of the pile is founded at a depth of 10m from the ground surface.

The bearing capacity of the single pile is as follows:

$Q_u = Q_{pu} + Q_f$ [refer, appendix A of [6]]

$Q_{pu} = (0.5\, D\, \gamma\, N_\gamma + P_d\, N_q) A_p$

$Q_f = \Sigma^{n}_{i=1} K\, P_{Di} \tan\delta\, A_{si}$

Where

A_p = cross-sectional area of the pile toe in cm^2

D = stem dia. in cm

γ = effective unit weight of the soil at pile toe in kg/cm^3

N_γ & N_q = bearing capacity factors depending upon the angle of internal friction ø at the toe cap.

P_d = effective over burden pressure at the pile toe in kgf/cm^3

$\Sigma^n_{i=1}$ = summation for n layers in which pile is installed

K = coefficient of earth pressure

P_{Di} = effective over burden pressure in kgf/cm^2 for the ith layer where I varies from 1 to n

δ = angle of wall friction between the pile & soil, in degrees

A_{si} = surface area of the pile stem in cm^2 in the ith layer where I varies from 1 to n

D = 250 mm

γ = 16 kN/m^3 & 19.5 kN/m^3

N_γ = 30 (for ø = 32°) [refer, table 1 of [5]]

P_d = 16 x 5 + 19.5 x 3

= 138.5 kN/m^2

N_q= 40 [refer, fig-1 of[6]]

q_{pu} = 5613.125 kN/m^2

*the value of q_{pu} be limited to 11 x 10^3 kN /m^2 [refer, pg-559 of [7]]

$Q_{pu} = q_{pu} \times A_p$

$= 5613.125 \times (0.25)^2 \pi/4$

= 275.53 kN

$Q_f = \Sigma^n_{i=1} K P_{Di} \tan\delta A_{si}$

$= (1 \times 16 \times 5 \times 0.625 \times \pi \times 0.25 \times 5) + (2 \times 19.5 \times 3 \times 0.625 \times \pi \times 0.25 \times 3)$

= 196.35 + 172.30

= 368.7 kN

$Q_u = Q_{pu} + Q_f$

= 275.53+ 368.7

= 644.18 kN

Q_{safe}= 644.18/2.5

= 257.7 kN

(K=1 for loose sand, 2 for dense sand) [refer, pg-561, table 16.3 [7]]

3.6 Loading of piles under boundary columns

Node under consideration – 17

Load on the pile group = 335.012 kN

Dead load of pile cap = 33.50 kN

Total verticle load on the pile = 368.512 kN

Moment in x direction = 2.165 kNm

Moment in z direction = 88.252 kNm

Eccentricity in z direction e_x = 240 mm

Eccentricity in x direction e_z = 5.9 mm

Total number of piles = 4

The axial load in any pile m, at a distance x & y from the centroid with eccentricity e_x & e_y is given by:

$Q_m = Q_g/N \pm (Q_g.e_x)x/\Sigma x\text{^}2 \pm (Q_g.e_y)y/\Sigma y^2$

= 92.12 + 49.13 + 1.20

= 142.45 kN < 257.7 kN (hence the pile is safe)

Deflection in the pile

$EI = 4224\ kNm^2$

Soil subgrade reaction $K = 10 \times 10^3\ kN/m^3$

$T = (EI/K)^{1/5}$

$T = (4224\ /10 \times 10^3)^{1/5}$

= 0.842 m

L/T = 9.5 > 5 hence it is a long pile

$L_1 = 0$ $L_e = 8$ m

$L_1/\ T = 0$

Hence $L_f / T = 1.95$

$L_f = 1.64$ m

Calculation for deflection

$Y = Q (L_1 + L_f)^3/12\ EI$

Q = the horizontal load acting on the pile. = 28.61 kN

$Y = (28.61 \times 1.64^3)/(12 \times 4224)$

= 0.002489 m

= 2.489 mm

3.7 Loading of piles under internal columns

Node under consideration – 18

Load on the pile group = 368.702 kN

Dead load of pile cap = 36.21 kN

Total verticle load on the pile = 404.91 kN

Moment in x direction = 80.061 kNm

Moment in z direction = 0.058 kNm

Eccentricity in z direction $e_x = 0.00014$ mm ≈ 0

Eccentricity in x direction $e_z = 197$ mm

Total number of piles = 4

The axial load in any pile m, at a distance x & y from the centroid with eccentricity e_y is given by:

$Q_m = Q_g/N \pm (Q_g.e_y)y/\Sigma y^2$

= 145 kN < 257.7 kN (hence the pile is safe)

Deflection in the pile

$EI = 4224$ kNm^2

Soil subgrade reaction $K = 10 \times 10^3$ kN/m^3

$T = (EI/K)^{1/5}$

$T = (4224 / 10 \times 10^3)^{1/5}$

$= 0.842$ m

$L/T = 9.5 > 5$ hence it is a long pile

$L_1 = 0$ $L_e = 8$ m

$L_1 / T = 0$

Hence $L_f / T = 1.95$

$L_f = 1.64$ m

Calculation for deflection

$Y = Q (L_1 + L_f)^3 / 12 EI$

Q = the horizontal load acting on the pile. = 28.095 kN

$Y = (28.095 \times 1.64^3)/(12 \times 4224)$

$= 0.00244$ m

$= 2.44$ mm

3.8 Reinforcement detailing in pile

- A reinforced pile is designed as a column, considering it fixed at one end and hinged at other end, the effective length of the pile is taken as 2/3rd the length embedded in the soil.
- The longitudinal reinforcement in a pile usually varies from 1.25% to 2% of the gross sectional area of the pile, depending on its length.
- For a length 30 times the least width of the pile, the longitudinal steel is usually 1.25% and the longitudinal steel is increased to 2% for a length above 40 times the least width of the pile.
- Lateral R/C with at least 5mm diameter bars have to be provided in the form of links, and the amount should not be less than 0.2% of the gross volume of the pile and the centre to centre spacing should not exceed half the least width of the pile.
- At ends of the pile for a distance of three times the least lateral dimension, lateral reinforcement should not be less than 0.60% of the gross volume.
- For piles penetrating hard soil, lateral reinforcement at the top of the pile for a distance of three times the width should be in the form of helix.
- The cover to reinforcement including binding wire shall not be less than 40mm and for piles exposed to seawater or other corrosive content this has to be increased to 50mm.

Reinforcement detailing in pile

Pile depth = 8m

Diameter of pile = 250mm

L/D = 32>12 (the pile behaves as long column)

Reduction coeff. $C_r = 1.25 - L_{ef}/48D = 0.53$

Design load for column $P_u = 1.5 \times 335/0.53 = 949$ kN

Load carrying capacity of the column is given by:

$$P_u = 0.4\, f_{ck} A_g + (0.67\, f_y - 0.4\, f_{ck})\, A_{sc}$$

$$949 \times 10^3 = 0.4 \times 30\ (\pi \times 125^2) + (0.67 \times 415 - 0.4 \times 30) A_{sc}$$

$$360.86 \times 10^3 = 266.05\, A_{sc}$$

$$A_{sc} = 1356\ mm^2$$

min. r/f = 1.5% of cross section area of the pile = $(1.5/100)^* \pi r^2$

$= 735.94\ mm^2$

Hence we provide 6 bars of 16 mm dia in circular arrangement

$A_{sc,\ actual} = 1206.37\ mm^2$

Lateral r/f in the body of the pile should be provided in the form of links of not less than 5mm dia. wires

Adopting 8mm dia bars

Length of the lateral bar required = 162 mm

Hence volume of the bar = 25653 mm^3

Volume of r/f needed per mm length = $(0.2 \times 3.14 \times 125^2)/100$

$= 98.13\ mm^3$

Pitch = 25653/98.13 = 260 mm

Max. permissible pitch = 0.5 x 250 = 125 mm

Hence provide 8mm dia. bars at 125mm c/c throughout the length of the pile.

Lateral r/f near the pile head

Special r/f to be provided for a length of = 3 x 250 = 750 mm

Volume of r/f per mm length = $0.6 \times 3.14 \times 125^2/100$

$=295mm^3$

Adopting 8mm dia. spiral pitch = $\pi \times 162 \times 50.3/295 = 86$ mm

Hence provide 8 mm dia. ~~spiral~~ at 80mm pitch at the top of the pile for a length of 750 mm

Lateral r/f at the pile end

Volume of ties per mm length @ 0.6% gross volume = $295mm^3$

Volume of tie = 25653

Pitch = 25653/295 = 87 mm

Hence provide 8mm dia. ties @ 85mm c/c at the bottom of pile for depth of 750mm

3.9 Design of pile cap

Design for moment

$Mu_{max} = (P_{ua1} + P_{ua2})(S - b/2)$

$P_{ua1} = 1.5$ ((Column load P + self wt of pile cap) / No. of piles + $M_{uy}X/\Sigma X^2$)

$= 1.5\ ((368 + 75)/4 + 1000\times88\times450/2(450)^2)$

$= 1.5\ (110 + 98)$

$=312$ kN-m

$P_{ua2} = 1.5\ (\ 110 - 1000\times88\ 450/2(450)^2)$

$= 1.5\ (110 - 98)$

$= 18$ kN-m

$Mu_{max} = 330\ ((0.9 + 0.6) / 2)$

$= 250$ kN-m

$$A_{st} = (0.5 fck / f_y)\left[1 - \sqrt{1 - (4.6 M_{ux} / (f_{ck} bd^2)}\right] bd$$

$= (0.5\times25/415)[1-\sqrt{(}1-(4.6\times250\times10^6)/25\times450\times650^2)]$

$= 1447.8\ mm^2$

$\nless 0.85Bd/f_y$

$\nless 0.85\times1450\times650/415$

$\nless 1930.42mm^2$

$A_{st} = 1950\ mm^2$

Providing 10# 18mm Dia bars ($A_{st(actual)}$= 2540 mm^2) at bottom in both directions

Development length

$L_d = 0.87f_y Ø/(4\tau_{bd})$

$= 0.87\times415\times18/(4\times1.92)$

$= 846.21$ mm

$L_d = 850$ mm

Nominal reinforcement of 10 # 10 mm Ø bars.

CHAPTER 4

Results & Conclusion

After completion of the project, we have gained the knowledge of pile foundation and its design for a 4-storey building standing on a sandy strata. The building has been analysed with the help of STAAD-PRO calculating the loads coming onto the sub-structure i.e. on the pile cap and pile and then designed them within the safe bearing capacity of the soil which is 257.7 kN. After working on the project following results have been obtained:

- Soil conditions demand the use of friction piles.
- Safe load bearing capacity of the pile is 257.7 kN

The amounts of forces taken by different piles in a pile group under a column are different depending upon the eccentricity of the oncoming load.

- The load taken by an individual pile under a boundary column is 142.45 kN
- Deflection in pile under a boundary column is 2.489 mm
- The load taken by an individual pile under an internal column is 145 kN
- Deflection in pile under a internal column is 2.44 mm
- The pile foundation is safe against the oncoming dead loads, imposed loads, wind loads & seismic loads.

The piles and pile cap have been designed for resisting bending moment and shear forces and are designed in accordance with Limit State design method as per I.S. 456–2002 for plain and reinforced concrete.

Deflection for the pile has been calculated through method of fixity as according to the guidelines prescribed by I.S. 2911 (part III) – 1980.

Appendix-A

Project Oriented Practice-1

Pile cap design

Q: Design a pile cap for supporting a column of section, 400x400 mm, carrying an axial load of 1000 KN at service state. Pile cap contains a group of 4 friction piles each of 250 mm diameter for transfer of load from column to soil. Consider concrete of grade M-20 & steel of grade Fe-415 [ref-9, pg-672]

Sol:

1) Pile cap dimensions

Consider the pile cap as shown in the figure. Its dimensions are determined by

L = B = S + D + 2 x 150mm (150 mm assumed overhang)

S=> spacing of the piles. It can be fixed such that the group efficiency is 100%

$100= [2(m+n-2)S+4D]x100/ \pi Dmn$

$S= (\pi Dmn-4D)/2(m+n-2)$ *

(* S should not be less than S_{min} , where S_{min} = 3D or 900 mm, whichever is more)

3D = 3 x 250

=750 mm

Hence S_{min} = 900 mm

$S = (\pi x 250 x 2 x 2 - 4 x 250)/ 2(2+2-2)$

Hence S is taken as 900 mm

L = B = 900 + 250 + 2 x 150 = 1450 mm

2) Depth of pile cap

The depth of the pile cap may be governed either by the moment or the shear.

a) Depth of pile cap based on moment

The effective depth of the pile cap may be determined by considering singly reinforced concrete section as follows.

$Bd^2 = M_{u,\,max} / 0.36\, f_{ck}\, (x_{u\ l/d})\, (1 - 0.416\, x_{u\ l/d})$

Where $M_{u,\,max}$ = maximum moment at the face of column about x & y axes = $2P_{up}\,(S-l)/2$

P_{up} = ultimate reaction from each pile on the pile cap considering that the pile group is symmetrical and the applied load is concentric to the pile group.

= load factor x (column load p + assumed self weight 0.1P of pile cap)/no. of piles

= 1.5 x (1000 + 0.1 x 1000)/4 = 412.5 kN

$M_{u,\,max}$ = 2 x 412.5 x (0.9-0.4)/2 = 206.25 kNm

$x_{u\ l/d}$ = 0.479 for steel of grade Fe-415

$1450d^2 = (206.25 \times 10^6) / [0.36 \times 20 \times 0.479 \times (1 - 0.416 \times 0.479)$

d = 226.95 mm

b) Depth of the pile cap based on shear

The effective depth of the pile cap may be determined by considering that the shear is resisted without shear reinforcement as follows.

$V_{u,\,max} = \tau_{uc}\, bd$

$d = V_{u,\,max} / \tau_{uc}\, b$

$V_{u,\,max}$ = ultimate one way or 2-way shear

For one way shear

$V_{u,\,max}$ = maximum ultimate shear at the critical section at a distance of effective depth of pile cap from the face of the column

= $2\,P_{up}$ = 2 x 412.5 = 825kN

τ_{uc} = k, (taken as 1 for assumed total depth >= 300mm) x shear strength of concrete which may be taken as its middle value of 0.36 N/mm^2, on consideration that flexure reinforcement shall be small due to large depth of the pile cap based on shear considerations

=0.36 N/mm^2

b = B= 1450 mm^2

d = 825 * 1000/(0.36 * 1450) = 1580.46mm

For two way shear

$V_{u, max}$= maximum ultimate punching shear based on punching of column or pile on critical section at a distance of half the effective depth of pile cap from the face of the column or pile respectively

= 1.5 * 1000 = 1500 kN (based on punching of the column)

= 412.5 kN (based on punching of the pile cap)

$\tau_{uc}= 0.25(f_{ck})^{0.5}$

$= 0.25 * (20)^{0.5} = 1.118$ N/mm^2

b = 4 (400+d) (for punching of the column)

= 4 (250+d) (for punching of pile)

d= (1500 * 1000)/ 1.118 * 4 * (400 + d) = 412.72 mm (based on punching of the column)

d= (412.5 x 1000)/ 1.118 x 4 x (250+d) = 203.43 mm (based on punching of the pile)

Hence the effective depth of the pile cap is governed by one way shear. However the effective depth of the of pile cap based on flexure and two way shear is very small as compared to that based on flexure and one way shear. Hence the design of pile cap by considering effective depth based on flexure & two way shear being economical is made as follows

d = 412.72 mm

$D = 412.72$ + clear cover 40 mm + assumed dia. of 16 mm of r/f bars /2 = 460.72

Let $D = 475$ mm

$d = 475 - 40 - 16/2 = 427$ mm

3) Design for moment :

Area for steel for maximum ultimate moment as determined above at the face of the column about x & y axis are determined with the help of design aid as,

$A_{st} = 1297.44\ mm^2 \quad < A_{st,min}$

$< 0.85\ Bd/f_y$

$0.85 \times 1450 \times 477/415 = 1416.63\ mm^2$

Hence $A_{st} = 1416.63\ mm^2$

Provide 100 x 14 mm dia. bars ($Ast = 1539\ mm^2$) at bottom in both directions. Theses bars are bent up at 90° & extended to provide adequate development length,

$L_d = 0.87\ f_y ø/4\ \tau bd = (0.87 \times 415 \times 14)/(4 \times 1.92) \approx 660$ mm

Nominal r/f of 12 mm dia. bars is provided at the top in both directions for holding shear R/F

4) Design for 1 way shear:

Consider 8mm dia., 8 legged verticle shear stirrups ($A_{st} = 400\ mm^2$) at spacing S_v

$S_v = (0.87\ f_y A_{sv}) / (\tau_{uv} - \tau_{uc})B$

Where $\tau_{uv} = V_{u,max}/Bd = 825 \times 100/ 1450 \times 427 = 1.3325\ N/mm^2$

T_{uc} = k x shear strength of concrete in beam

$= 1 \times 0.36 = 0.36\ N/mm^2$

$S_{v,max} = 2.5\ f_y A_{sv}/B$ or $0.75\ d$ or 345 mm , whichever is less

$= 2.5 \times 400 \times 415 / 1450$ or 0.75×477 or 345 mm

$= 286.2$ mm

$S_v = 0.87 \times 415 \times 400 / (1.3325 - 0.36) \times 1450$

Provide 8 mm dia., 8 legged verticle stirrups at 100 mm c/c. nominal binding bars of 12 mm dia. at 150mm c/c are provided around outer piles.

Pop # 2

Solution:

The axial load in any pile m, at a distance x from the centroid is given by:

$Q_m = Q_g/N \pm (Q_g.e_x)x/\Sigma x^2$

e_x eccentricity of the load about y-y axis.

If the load is eccentric about both the axis

$Q_m = Q_g/N \pm (Q_g.e_x)x/\Sigma x\text{^}2 \pm (Q_g.e_y)y/\Sigma y^2$

e_y eccentricity of the load about x-x axis

N=12 $Q_g = 4$ mN

e_x =0.3 m $e_y = 0.4$ m

$\Sigma x^2 = 6 \text{ x } 0.5^2 + 6 \text{ x } 1.5^2 = 15\text{m}^2$

$\Sigma y^2 = 4 \text{ x } 1^2 + 4 \text{ x } 1^2 = 8 \text{ m}^2$

the maximum load occurs in pile 4

$Q_m = 4.0/12 + (4 \text{ x o}.3/15) \text{ x } 1.5 + (4 \text{ x } 0.4/8) \text{ x } 1$

$= 0.6533$ mN

$= 653.3$ kN

Pop # 3

Solution:

Let dia. Of the pile = 500 mm

Depth = 18 m

Cut off level = 1 m below GL

Q_{ult} per pile:

1) Brownish grey clay with fine sand

$C_u = 35\ kN/m^2$ $\alpha = 0.7$

Skin friction resistance = $\alpha\ C_u\ A_s$

$= 0.7 \times 35 \times 3 \times \pi \times 0.5$

$= 115.45$ kN

2) Brownish grey sandy silt with clay binders

let $\varnothing = 30°$ let $\gamma = 20\ kN/mm^3$

$\delta = 3/4\ \varnothing = 22.5$ $k = 1$

$\sigma_{av} = 20 \times ½ = 10\ kN/m^2$

$f_s = \sigma_{av}\ k \tan\delta = 10 \times 1 \times \tan 22.5$

$= 3.86$

SFR = $f_s A_s$ $= 3.86 \times \pi \times 0.51 \times 1$

$= 6.50$ kN

3) Dark grey clay with decomposed wood

$C_u = 25\ kN/m^2$ $\gamma = 17$ kN/m3 $\alpha = 0.7$

SFR = $0.7 \times 25 \times \pi \times 0.5 \times 8$

$= 219.91$ kN

4) Bluish grey silty clay with kankar

$C_u = 65\ kN/m^2 \alpha = 0.4$

SFR = $0.4 \times 65 \times \pi \times 0.5 \times 3$

$= 122.52$ kN

5) Motled.brown silty clay

$C_u = 70\ kN/m^2$ $\alpha = 0.3$

SFR = 0.3 x 70 x π x 0.5 x 4

= 131.95 kN

Point load at tip = 119.7 kN

Total Q_{ult} = 716.9 kN

Q_{ult} per pile = 716.9/3 = 238.9 kN or 240 kN

Q_{uplift} per pile = 589.83/3 = 196.61 kN or 200 kN

Case 1 loading:

DL = 240 kN lateral serveice wind = 17.5 kN

LL = 520 kN moment = 17.6 x 6.5 = 113.75 kNm

Self weight of cap = 100 kN

Total weight = 860 kN

E_x = 113.75/860 = 0.13

$Q_m = Q_g/N \pm (Q_g.e_x)x/\Sigma x^2$

= (860/4) ± (860 x 0.13 x 2/ 16)

= 215 ± 13.9

$Q_{m,\ max}$ = 229 kN < 240 kN ok

$Q_{m,\ min}$ = 201 kN ≈ 200 kN ok

Case 2 loading:

DL = 240 kN lateral serveice wind = 140 kN

LL = 100 kN moment = 140 x 6.5 = 910 kNm

Total weight = 340 kN

E_x = 113.75/860 = 0.13

$Q_m = Q_g/N \pm (Q_g.e_x)x/\Sigma x^2$

$= (340/4) \pm (340 \times 2.67 \times 2/ 16)$

$= 85 \pm 113.49$

$Q_{m, max} = 198$ kN < 240 kN ok

$Q_{m, min} = -28$ kN < 200 kN ok

Hence the pile groups are safe for both the cases of loading

Total lateral load on pile = 140/4 = 35 kN

Stiffness factor $T = (EI/K)^{1/5}$

$EI = 61.2 \times 10^6$

$K= 10 \times 10^6$

$T = (61.2 \times 106/10 \times 10^6)^{1/5}$

$= 1.436$ m

$L/T = 12.53 > 5$ hence it is a long pile

$L_1 = 0$ $L_e = 18$ m

$L_1/ T = 0$

Hence $L_f/ T = 1.3$

$L_f = 187.6$ cm

Calculation for deflection

$Y= Q (L_1 + L_f)^3/12 EI$

Case 1:

Service wind load = 1750 kg

$Y = 1750 (0 +186.7)^3/(12 \times 61.2 \times 10^9)$

$= 0.155$ mm

Case 2:

Storm wind load = 14000 kg

$= 14000 \times (186.7)^3/ (12 \times 61.2 \times 10^9)$

$= 0.128$ cm or 1.28 mm

Pop#4

Solution:

$E = 20\ kN/mm^3 = 2000\ kN/cm^3$

$I = (\pi D^4/64) x\ (4.5)^4 = 20.14\ cm^4$

$K = 10 \times 10^6\ N/m3 = 10N/cm^3$

$EI = 4.048 \times 10^7$

$T = (4.048 \times 10^7 / 10 \times 10^6)^{1/5} = 1.32\ m$

Since $L > 5T$ (so it is a long flexible pile)

$L_1 + L_e = 15m$

$L_1 = 0$

$L_e = 15\ m$

$L_1/T = 0$ corresponding value of $L_f/T = 1.3$

Hence $L_f = 1.3 \times 1.32 = 1.716m$

$Y = Q\ (L_1 + L_f)^3 / 3EI$

$= 30 \times 10^2 (171.6)^3 / (3 \times 4.04 \times 10^{10})$

$= 0.125$ cm or 1.25 mm

APPENDIX B
REINFORCEMENT DETAILING AND DRAWINGS

Grade of concrete M-25
Grade of steel Fe-415

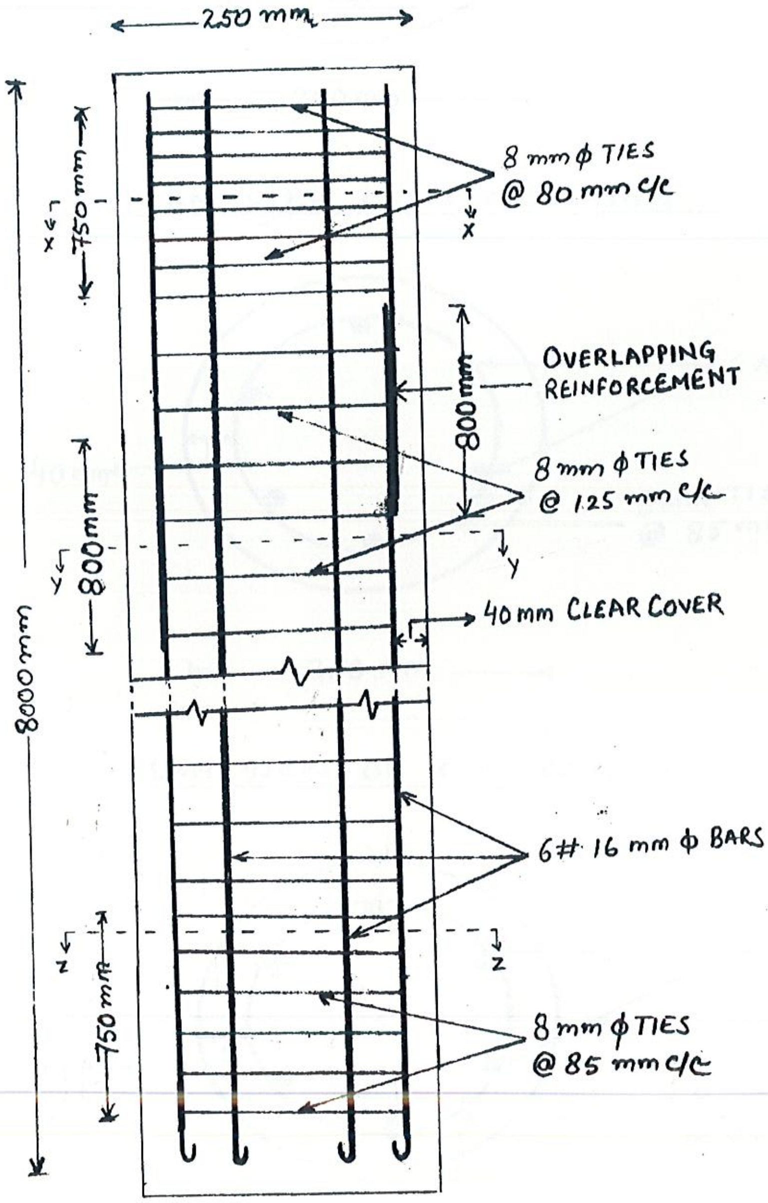
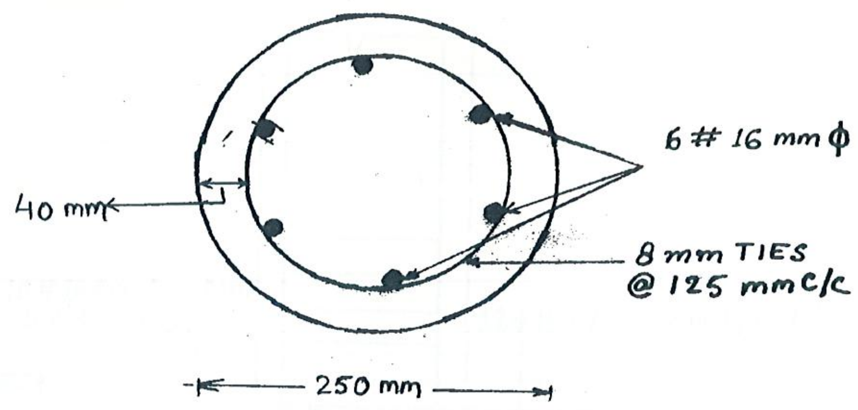


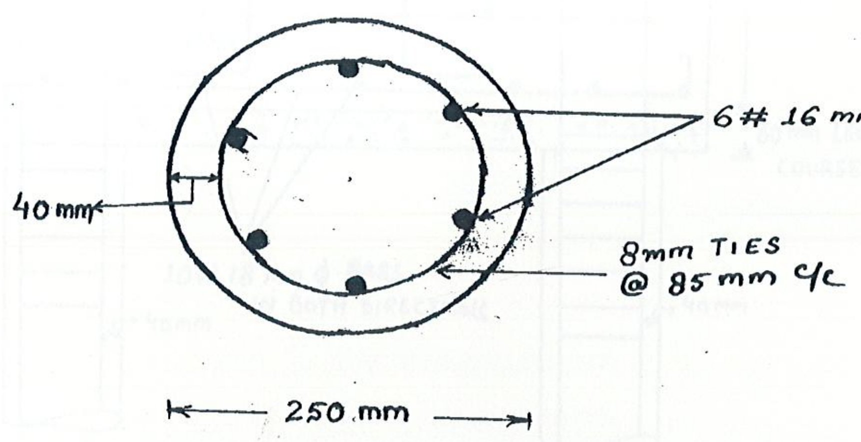
Fig. a Reinforcement detailing of piles

Grade of concrete M-25
Grade of steel Fe-415

PLAN VIEW OF Y-Y SECTION



PLAN VIEW OF Z-Z SECTION



PLAN VIEW OF X-X SECTION

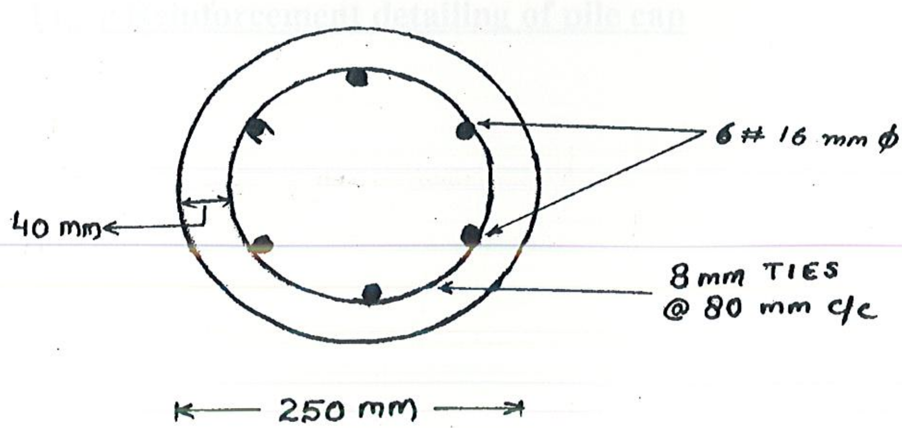


Fig.b Plan view of pile detailing

Grade of concrete M-25
Grade of steel Fe-415

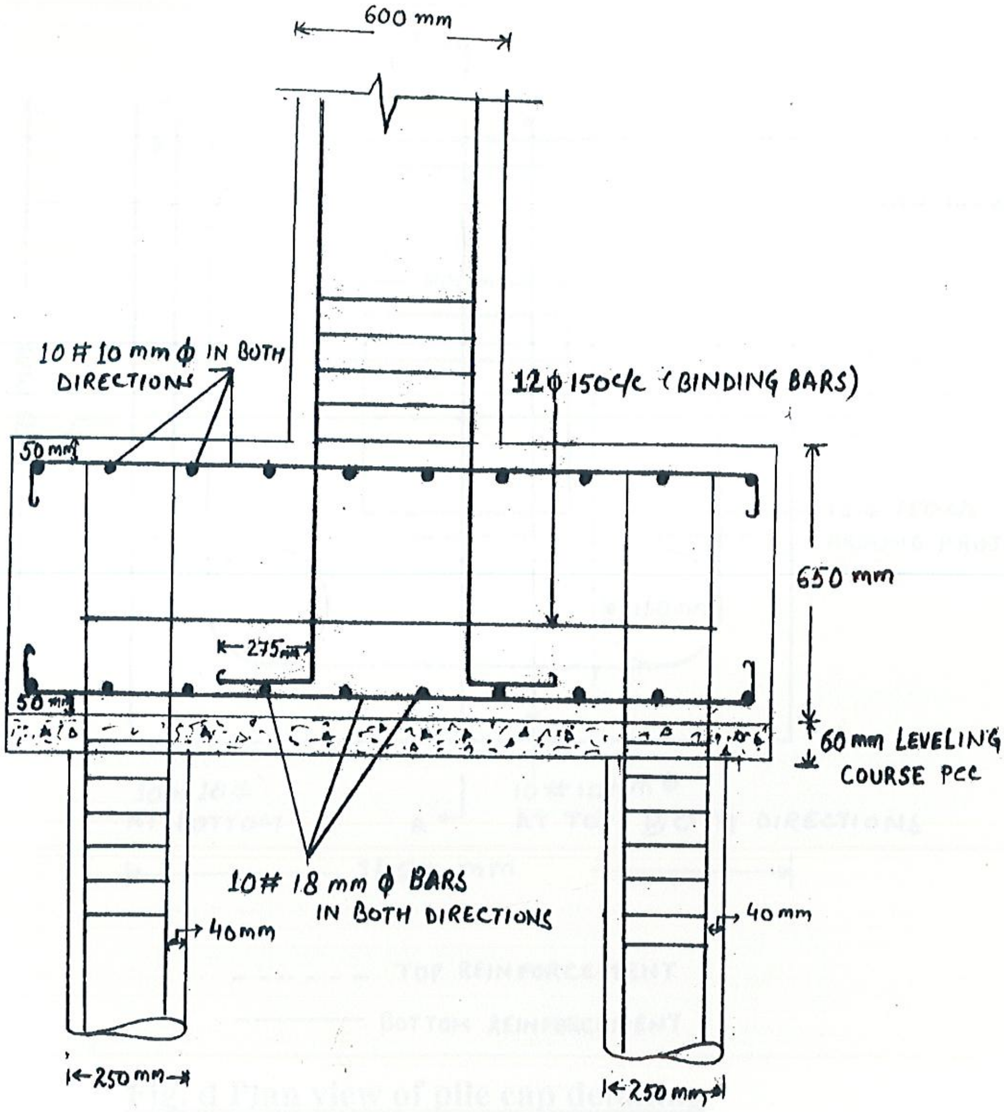


Fig. c Reinforcement detailing of pile cap

Grade of concrete M-25
Grade of steel Fe-415

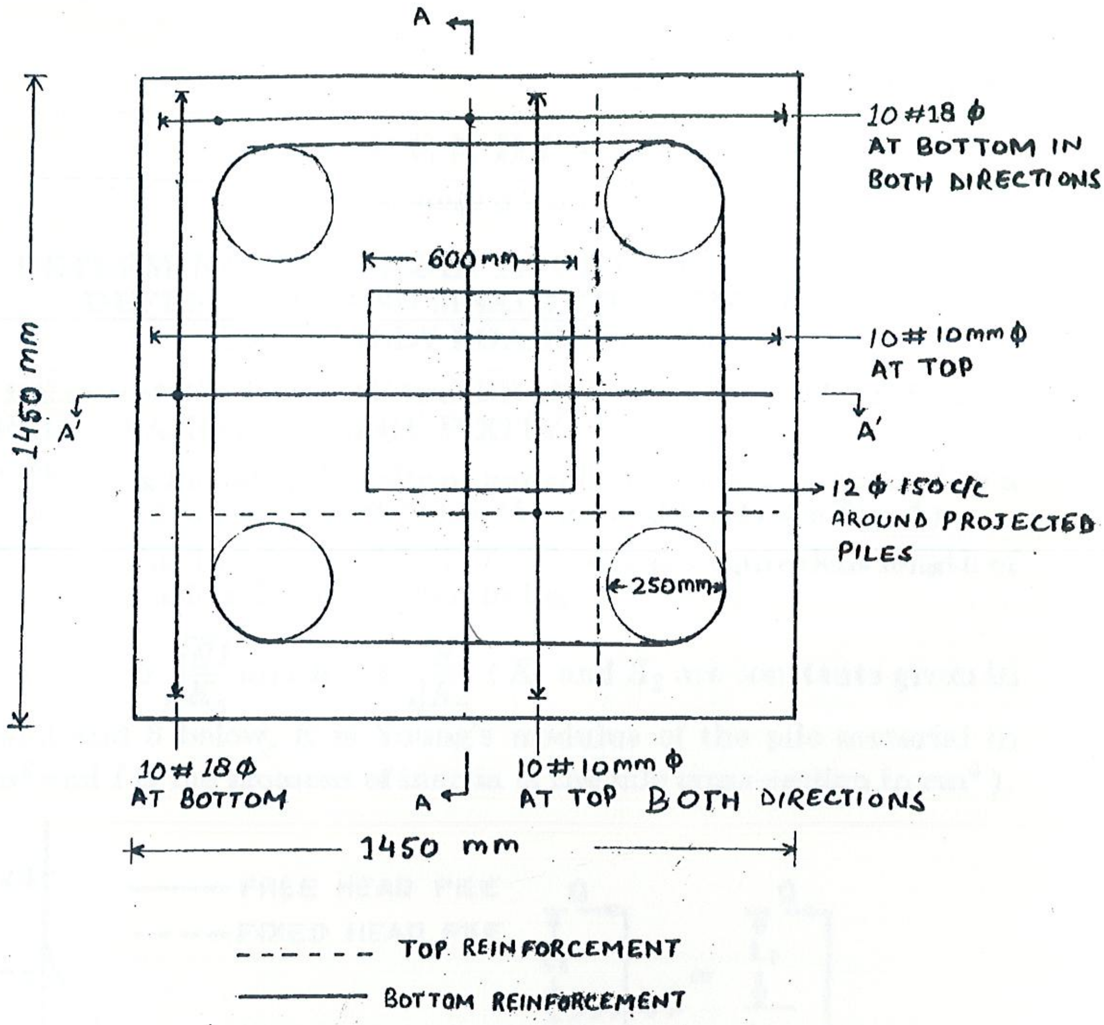


Fig. d Plan view of pile cap detailing

APPENDIX C

(*Clause* 5.2.5)

DETERMINATION OF DEPTH OF FIXITY, LATERAL DEFLECTION AND MAXIMUM MOMENT OF LATERALLY LOADED PILES

C-1. DETERMINATION OF LATERAL DEFLECTION AT THE PILE HEAD AND DEPTH OF FIXITY

C-1.1 The long flexible pile, fully or partially embedded, is treated as a cantilever fixed at some depth below the ground level (*see* Fig. 4).

C-1.2 Determine the depth of fixity and hence the equivalent length of the cantilever using the plots given in Fig. 4.

where $T = \sqrt[5]{\frac{EI}{K_1}}$ and $R = \sqrt[4]{\frac{EI}{K_2}}$ (K_1 and K_2 are constants given in Tables 2 and 3 below, E is Young's modulus of the pile material in kg/cm^2 and I is the moment of inertia of the pile cross-section in cm^4).

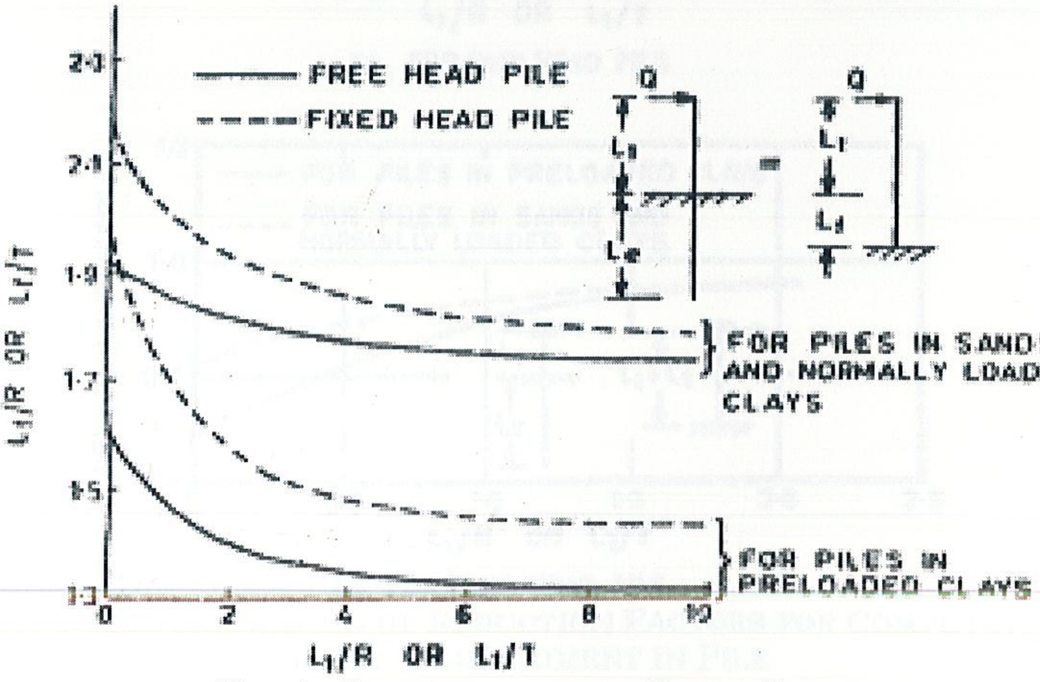
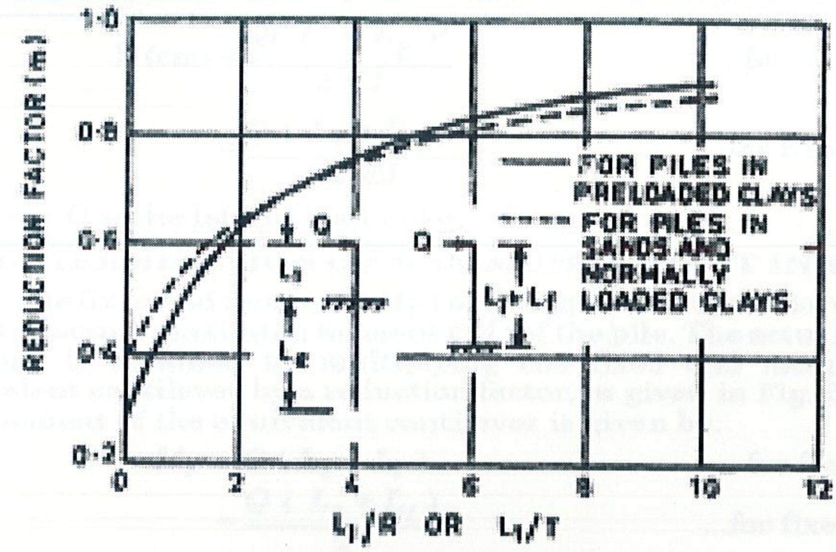


FIG. 4 DETERMINATION OF DEPTH FIXITY

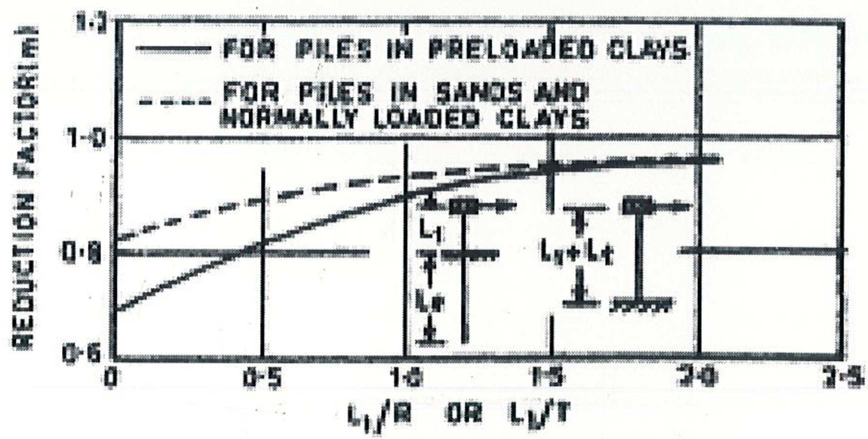
NOTE — Figure 5 is valid for long flexible piles where the embedded length, L_e is $\geq 4R$ or $4T$.

TABLE 2 VALUES OF CONSTANT K_1 (kg/cm^3)

TYPE OF SOIL	VALUE	
	Dry	Submerged
(1)	(2)	(3)
Loose sand	0.260	0.146
Medium sand	0.775	0.525
Dense sand	2.075	1.245
Very loose sand under repeated loading or normally loading clays	—	0.040



5A FOR FREE HEAD PILE



5B FOR FIXED HEAD PILE

FIG. 5 DETERMINATION OF REDUCTION FACTORS FOR COMPUTATION OF MAXIMUM MOMENT IN PILE

TABLE 3 VALUES OF CONSTANT K_2 (kg/cm^3)

UNCONFINED COMPRESSIVE STRENGTH kg/cm^2	VALUE
(1)	(2)
0.2 to 0.4	7.75
1 to 2	48.80
2 to 4	97.75
More than 4	195.50

C-1.3 Knowing the length of the equivalent cantilever the pile head deflection (Y) shall be computed using the following equations:

$$Y \text{ (cm)} = \frac{Q(L_1 + L_F)^3}{3\,EI} \qquad \text{...for free head pile}$$

$$= \frac{Q(L_1 + L_F)^3}{12\,EI} \qquad \text{...for fixed head pile}$$

where Q is the lateral load in kg.

C-2. DETERMINATION OF MAXIMUM MOMENT IN THE PILE

C-2.1 The fixed end moment (M_F) of the equivalent cantilever is higher than the actual maximum moment (M) of the pile. The actual maximum moment is obtained by multiplying the fixed end moment of the equivalent cantilever by a reduction factor, m given in Fig. 5. The fixed end moment of the equivalent cantilever is given by:

$$M_F = Q(L_1 + L_F) \qquad \text{...for free head pile}$$

$$= \frac{Q(L_1 + L_F)}{2} \qquad \text{...for fixed head pile}$$

The actual maximum moment (M) = m (M_F).

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