UNIT - IV

PILE FOUNDATIONS

INTRODUCTION

Deep foundations are employed when the soil strata immediately beneath the structure is not capable of supporting the load with tolerable settlement or adequate safety against shear failure.

Piles, piers, caissions and well foundations are examples of deep foundations.

Pile is a slender structural member made of steel, concrete or wood. A pile is either driven into the soil or firmed in-situ by excavating a hole and filling it with concrete.

A pier is vertical column of relatively larger cross section than a pile. A cast-in-situ pile greater than 0.6 m diameter is generally termed as a pier. A pier is installed in a dry area by excavating a cylindrical hole of large diameter to the desired depth and then backfilling it with concrete.

Caissions and wells are constructed by excavation and are sunk to the required depth. These usually permit the vertical excavation of the soil or rock on which they rest. They are normally used to carry very heavy loads such as those from bridge piers.

Necessity of Pile Foundations

Piles are used in the following conditions

1. When the strata at or just below ground is highly compressible and very weak to support the load transmitted by the structure.
2. When the plan of the structure is irregular relative to its outline and load distribution. This leads to non-uniform settlement if a shallow foundation is constructed. A pile foundation is required to reduce differential settlement.
3. Pile foundations are required for the transmission of structural loads through deep water to a firm stratum.
4. Pile foundations are used to resist horizontal forces in addition to support the vertical loads in earth retaining structures and tall structures that are subjected to wind and earthquake.
5. Pile foundations are required when the soil conditions are such that a washout, erosion or scour of soil may occur underneath shallow foundations.
6. Pile foundations are used for structures subjected to uplift (Towers, offshore platforms etc).
7. Piles are used to transfer the load below active zone in case of expansive soils.
8. Piles are used in collapsible soils to transfer the load beyond the zone of moisture changes. Soils such as loess lose their structure suddenly when there is an increase in water content.

Classification of Piles

1. Classification according to material used
   (a) **Steel piles**: These are generally either in the form of thick pipes or rolled steel H-sections and are driven with their ends open or closed. A driving point or shoe is provide at the lower end.

(b) **Concrete piles**: These are either precast or cast in-situ. In precast piles, reinforcement is provided to resist handling and driving stresses. These can also be prestressed using high strength steel pretensioned cables.

Cast-in-situ pile is constructed by making a hole in the ground and filling it with concrete. This pile may be cased or uncased.

A cased pile is constructed by driving the casing to the desired depth and gradually with drawing casing when fresh concrete is filled. Uncased pile may have a pedestal.

(c) **Timber piles**: Made from trimmed, straight , sound trunks free from defects. Steel shoes are provided to damage during driving. Splicing is done using a pipe sleeve or metal straps and bolts to increase the length of timber pile.

Timber piles below the water table usually have long life. They are effected by insects above water table.

(d) **Composite piles**: Made of two materials. May consist of steel in lower portion and cast-in-concrete timber below permanent water table and upper portion of concrete. Disadvantage is to provide joint between two dissimilar materials.

2. **Classification based on mode of Transfer of loads.**

(a) **End bearing piles**: Transmits the loads through bottom tips. These act as columns and transmit the load through a weak material to a firm stratum below. If required, these piles are extended to the bed rock.

The ultimate capacity of the soil depends upon the bearing capacity of the rock. If instead of bed rock, a firmly compact and hard stratum of soil exists at a reasonable depth, piles can be extended a few meters into the hard stratum.

End bearing piles are also known as Point bearing piles.

The ultimate load carried by the pile (Q_u) is equal to the load carried by the point or bottom of pile (Q_p).

(b) **Friction piles**: These piles do not reach the hard stratum. Load is transferred through the skin friction between embedded surface of the pile and surrounding soil. These are used when hard stratum does not exist at reasonable depth.
These piles are also known as floating piles as they do not reach the hard stratum.
The ultimate load (Q_u) carried by the pile is equal to transferred by skin friction (Q_s). In clayey soils, load is transferred by adhesion and not friction.

(c) **Combined end bearing and friction piles**.
These piles transfer loads by combination of end bearing at the pile bottom and friction along the surface of pile shaft.
The ultimate load carried by a pile (Q_u) = $Q_p + Q_s$

3. **Classification based on method of Installation**

(a) **Driven piles**: These are driven into the soil by applying the blows of a heavy hammer.

(b) **Driven and Cast-in-situ Piles**: These are formed by driving a casing with a closed bottom end into soil. The casing is later filled with concrete. The casing may or may not be withdrawn.
(c) **Bored and cast-in-situ piles**: These are formed by excavating a hole into the ground and then filling it with concrete.
(d) **Screw piles**: These are screwed into the ground.
(e) **Jacked piles**: These are jacked into the ground by applying a downward force with the help of a hydraulic jack.

4. Classification based on use

(a) **Load bearing piles**: Used to transfer the load of the structure to suitable stratum by the end bearing, friction or both.
(b) **Compaction piles**: these are driven into loose granular soils to increase relative density. Densification is caused by vibrations and hence bearing capacity is increased.
(c) **Tension piles**: These piles are in tension. They are used anchor down structures subjected to hydrostatic uplift forces or overturning forces.
(d) **Sheet piles**: These form a contains wall or bulk head which is used for retaining earth or water.
(e) **Fender piles**: These are sheet piles used to protect water front structures from impact of ships and vessels.
(f) **Anchor piles**: These are used to provide anchorage for anchored sheet piles. These provide resistance against horizontal pull for a sheet pile wall.

5. Classification based on displacement of soil

(a) **Displacement piles**: All driven piles are displacement piles as a huge volume of soil is displaced laterally during pile installation.
The soil gets densified. Installation may cause heaving of the surrounding ground.
Precast concrete pile and closed end pipe piles are high displacement piles.
Steel H-Piles are low displacement piles.
(b) **Non displacement piles**: Bored piles are non - displacement piles. The installation of these piles causes very little change in the stresses in the surrounding soil.

Selection of Pile Type

Selection of type of pile depends on

(i) Type of structure and load it carries
(ii) Location of the site
(iii) Soil conditions and position of water table
(iv) Required pile length and structural capacity of the pile
(v) Durability
(vi) Economy

Selection of pile material depends on the magnitude of structural load.
Light loads – Timber piles – 10 to 18 m long – 150 to 200 kN
Heavy loads – RCC or Steel piles.
In crowded areas where structures are already constructed – only small or non – displacement piles have to be used.

In loose to medium dense sand, displacement piles help compact the soil.
In clays, use of displacement piles may result in heaving of the soil. Hence non-displacement piles are preferred.
Driven piles are unaffected by ground water table (GWT) conditions while in bored piles, contracting has to be done carefully.

Pile Driving

1. **Hammer Driving:** A pile driving rig is used. It is consist of a horst mechanism, a guiding frame and a hammer device.

   Types of hammers used for pile driving

   (i) **Drop hammer:** A drop hammer is raised by winch and allowed to drop on the top of the pile under gravity from a certain height.
   During driving operation, a cap is fixed to the top of the pile and a cushion is provided between the pile and the cap. Another cushion, known as hammer cushion, is placed pm the pile cap on which the hammer causes the impact. This is old (rarely used).

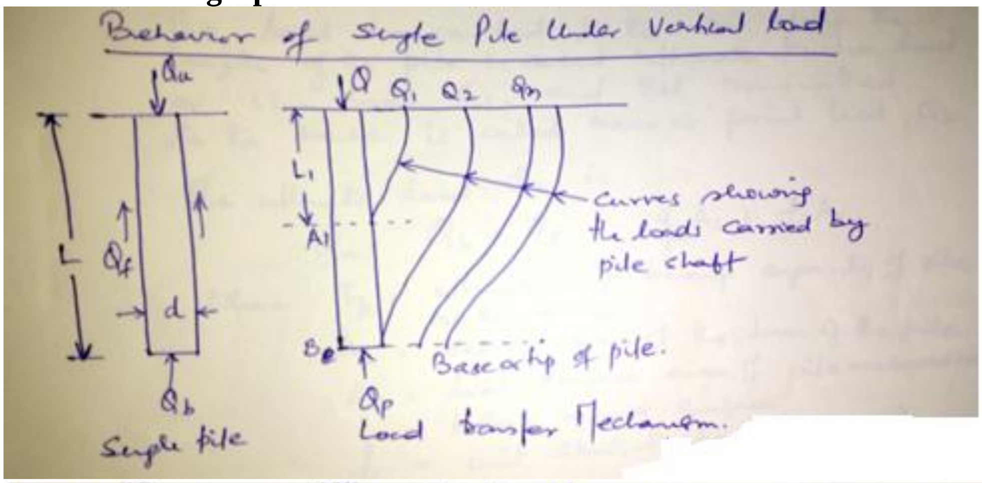
   (ii) **Single acting hammer:** In acting hammer, the ram is raised by air or steam pressure to the required height. It is then allowed to drop under gravity on the pile cap provided with a hammer cushion.

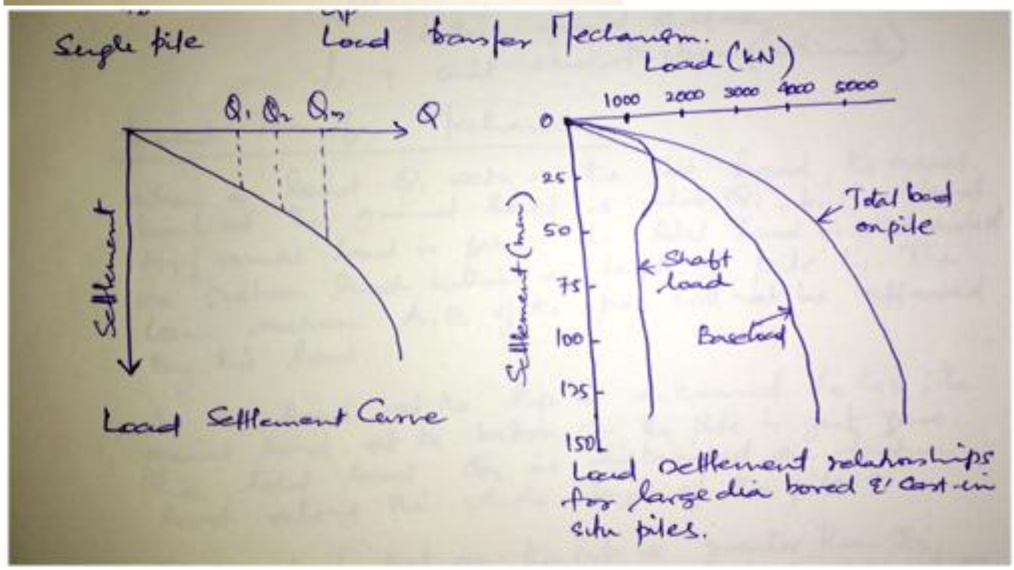
   (iii) **Double acting hammer:** The steam or air pressure is to raise the hammer to the required height and also the hammer is pushed under air or steam pressure. This increases the impact energy of the hammer.

   (iv) **Diesel hammer:** This hammer consists of a ram and a fuel injection system. It is also provided with an anvil block at its lower end. The ram is first raised manually and fuel is injected near the anvil. As soon is the hammer is released, it drops on the anvil and compresses the air fuel **mixture** and ignition takes place. The pressure so developed pushes the pile downward and raises the ram. The fuel is again injected and process repeated.
   These are not suitable for driving in soft soils.

2. **Vibratory pile driver:** This driver consists of weights, called “exciters”, which rotate in opposite directions. The horizontal components of centrifugal force so generated cancel each other and vertical components get added. A sinusoidal dynamic vertical force is applied to the pile. The frequency of vibration is kept equal to natural frequency of pile – soil system for better results.
3. **Jetting techniques:** Water under pressure is discharged at the pile bottom by means of a pipe to wash and loosen the hard layer.
4. **Partial Augering method:** Batter piles (inclined piles) are usually advanced by this method.
   Power augering is used to drill the hole for a part of the depth. The pile is then inserted in the hole and driven with hammers to the required depth.

Behavior of single pile under vertical load





Consider a single pile of uniform diameter ‘d’ and length ‘L’ to which a static vertical load is applied.

Determination of ultimate bearing capacity (Q_u) of the pile

When the ultimate load applied on the top is Q_u , a part of load is transmitted to the soil along the length of the pile and balance is transmitted to the pile base.

The load transmitted to the soil along the length of the pile is called ultimate friction load or skin load, Q_f , and that transmitted to the base is called base or point load, Q_b.

The ultimate load Q_u is

$$Q_u = Q_b + Q_f = q_u A_b + f_s A_s$$

Where q_{pu} = ultimate unit bearing capacity of pile at the base
A_b = bearing area of the base of the pile

A_s = total surface area of the pile embedded below ground surface
F_s = unit skin friction (ultimate)

Load Transfer Mechanism

When a load Q_1 acts on the pile head, the axial load at the ground level is also Q_1, but at level A_1, axial load is zero. The total load is distributed as friction load within a length of the pile L_1. The lower section A_1B of the pile will not be affected by this load.

As the load at the top is increased to Q_2, the axial load at the bottom of the pile is just zero. The total load Q_2 is distributed as friction load along the whole length of the pile L.

If the load put on the pile is greater than Q_2, a part of this load is transferred to the soil at the base point load and the rest is transferred to the surrounding soil around the pile. With the further increase in load Q on the top, both the friction and point loads go on increasing the friction load attains an ultimate value Q_f at a particular load level, say Q_m, at the top, and any further increment of load added to Q_m, will not increase the value of Q_f. However, the point load Q_p, still goes on increasing till the soil fails by punching shear failure. The point load Q_p increases linearly with the elastic compression of the soil at the base.

The relative proportions of load carried by skin load and point load depend on the shear strength and elasticity of the soil. Generally, the vertical movement of the pile which is required to mobilize full end resistance is much greater than that required to mobilise full skin friction.

Note:

(a) When the ultimate skin friction is mobilised, only a fraction of the ultimate load mobilised.

(b) When the ultimate point load is mobilised, the skin friction resistance decreases to a lower value than its peak.

If $Q_p >> Q_f$, the pile may be called “Point Bearing Pile”.
If $Q_f >> Q_p$,the pile may be called “Friction Pile”.

*

$$Q_{pu} = \left[C_{ub}N_c + \ 0.3\ \gamma B\ N_\gamma + \ q'N_q\right] A_b --- for\ circular\ pile$$
$$Q_{pu} = \left[C_{ub}N_c + \ 0.4\ \gamma B\ N_\gamma + \ q'N_q\right] A_b --- for\ square\ pile$$

For C = 0

$$Q_{pu} = \left[0.3\ \gamma B\ N_\gamma + \ q'N_q\right] A_b --- for\ circular\ pile$$
$$Q_{pu} = \left[0.4\ \gamma B\ N_\gamma + \ q'N_q\right] A_b --- for\ square\ pile$$

Pile load capacity in compression

General requirements for satisfactory behavior of pile foundations are the same as for other foundations i.e., adequate safety against shear failure and excessive settlement.

The load capacity of the pile can be estimated by

(a) Static formulae
(b) Dynamic formulae
(c) Pile load test
(d) Correlation with penetration test data

IS code says:
Static formulae may be used only as a guide for load capacity estimates. More reliance is to be place on load test on piles.

Static formulae
The general equation for unit point resistance q_{pu} for a C - Φ soil may be written in the form

$$q_{pu} = CN_c + q'N_q + 0.5\,\gamma B\,N_\gamma --- (1)$$

Where, B = width or dia of the pile
q′ = effective overburden pressure at the tip of the pile equal to γL
L = Length of embedment of the pile
N_c, N_q and N_γ = Bearing capacity factors
C = unit cohesion
γ = effective unit weight of the soil

In a deep foundation, the term $0.5\,\gamma B\,N_\gamma$ is quite small compared to $q'N_q$ and hence it is usually neglected.
∴The equation for C - Φ soil reduces to

$$q_{pu} = CN_c + q'N_q --- (2)$$

For granular soils, C = C′ = 0

$$\therefore q_{pu} = q'N_q --- (3)$$

For a clay soil C = C_u and Φ_u = 0

$$\therefore q_{pu} = C_{ub}N_c --- (4)$$

Where C_{ub} = undrained shear strength of clay at the base of the pile.
The ultimate point load Q_{pu} can be expressed as

$$Q_{pu} = q_{pu}A_b --- (5)$$

Where A_b = sectional area of pile at its base.

The general equation for the ultimate skin friction resistance Q_f may be written as

$$Q_f = f_sA_s --- (6)$$

f_s = unit skin friction resistance
A_s = surface are of the pile in contact with the soil

∴ Ultimate load capacity Q_u is

$$Q_{pu} = q_{pu}A_b + f_sA_s --- (7)$$

One of the first steps in designing a single pile is to relate q_{pu} and f_s to basic soil strength parameters.

For piles in granular soils, the design is based on effective stress analysis. In clays it is common to use total stress analysis in which load capacity is related to undrained shear strength C_u.

PILES IN GRANULAR SOILS (Sand & Gravel)
Driven piles:
C′ = 0 for granular soil. The ultimate load capacity of a single pile, driven into granular soil, is obtained by eq. (7) i.e.,

$$Q_u = q_{pu}A_b + f_sA_s$$

(a) Point bearing:
In granular soil,

$$q_{pu} = q\,'N_q + 0.5\,\gamma B\,N_\gamma$$

For circular piles, $q_{pu} = q'\,N_q + 0.3\,\gamma\,B\,N_\gamma$
For square piles, $q_{pu} = q'\,N_q + 0.4\,\gamma\,B\,N_\gamma$

The above equation suggests that the unit point resistance increases in direct proportion to the embedded length of the pile. However, q_{pu} increases only upto a limited depth, beyond which it becomes constant. This depth is called "Critical depth" of pile.

Critical depth (Z_c) depends on Φ′ and width or depth of pile.

In loose to medium sands, critical depth = 15D
In dense sands, critical depth = 20D
Where D = dia or width of pile.

Critical depth concept is not applicable to clay strata where arching effect is absent.
It is recommended that unit point resistance q_{pu} be limited to 11000 kN/m^2 in normal silica sand and 5000 kN/m^2 in calcareous sand.

IS:2911 Part I (1979) provides a plot between Φ and N_q, using which N_q is determined. N_q is plotted on y – axis in log – scale. Minimum value of Φ on this plot is 20°.

The code includes the term $0.5\,\gamma\,B\,N_\gamma$ in addition to $q'\,N_q$ for determining q_{pu}, even though it is small and usually neglected.

The code also recommends that N_γ values should be taken corresponding to general shear failure.

In general, $\left(N_\gamma\right)_{deep\ foundation} = 2\left(N_\gamma\right)_{shallow\ foundation}$

A factor of safety of 2.5 on the ultimate load capacity is recommended for computing the safe load.

When piles are driven to refusal into a very dense stratum or rock, the safe load on the pile will be governed by the strength of the piles as a structural member than the ultimate load capacity.

IS code recommends that in working out pile capacities using static formulae for piles longer than 15 to 20 pile diameters, maximum effective overburden pressure should correspond to pile lengths equal to 15 to 20 diameters.

$$q' = \gamma Z \quad if \;\; Z < Z_c \; (Critical\ depth)$$
$$q' = \gamma Z_c \quad if \;\; Z > Z_c$$

Critical depth is the depth beyond which the vertical stress does not increase linearly with depth.

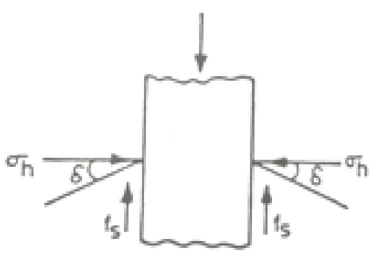
(b) Skin friction:
The unit skin friction acting at any depth is equal to the soil pressure acting normal to the pile surface at that depth multiplied by the coefficient of friction between the soil and the pile material($\tan \delta$).

The soil pressure acting normal to the pile surface, σ_h , is horizontal and is related to the effective vertical overburden pressure, q′, by the equation.

$$\sigma_h = Kq' \quad - - - (8)$$

$$\boxed{(q' = \sigma_h' = \gamma Z)}$$

Where K is lateral earth pressure coefficient.



Thus

$$f_s = \sigma_h \tan \delta$$
$$f_s = K\, q' \tan \delta \quad - - - (9)$$
$$Q_f = \left(K\, q'_{avg} \tan \delta\right) A_s \quad - - - (10)$$

q'_{avg}= avg. effective overburden pressure over the embedded length of the pile.

IS code recommends a value of $\delta = \Phi$.
For piles in loose to medium sands, the recommended value of K is between 1 and 3.

Though eqn.(9) suggests that $\mathbf{f_s}$ increases continuously with depth, which varies from 15 to 20 pile diameters. Below the critical depth, the value of **q′** and hence the value of $\mathbf{f_s}$ remains constant.

Maximum value of f_s should be limited to 100 kN/m^2 for straight sided piles in normal silica sands and to 20 kN/m^2 in calcareous sands.

Tapered piles have much higher value of f_s than straight piles.

Bored and Cast-in-situ piles (Sand & Gravel)

The load carrying capacity of a bored cast-in-situ pile will be much smaller than that of a driven pile in sand.

$$(q_{pu})_{bored\ pile} = \frac{1}{2}\ to\ \frac{1}{3}\ (q_{pu})_{driven\ pile} in\ gravel$$

The procedure used for a driven pile can be used for bored pile also, but the in-situ angle of shearing resistance of soil is reduced by 3°, to account for loosening of sand due to drilling of the hole.

K for a bored pile can be calculated approximately as $= 1 - \sin \Phi$. K varies from 0.3 to 0.75, with a medium value of 0.5.

$\delta = 0$ for bored piles excavated in dry soil.
$\delta < \Phi$ If slurry is used during excavation.

Driven cast-in-situ piles (Sand & Gravel)
Load carrying capacity is calculated in the same manner as for a driven pile, if steel tube is left in place.

If steel tube is refracted while the concrete is being poured,

(a) Loose soil condition may be assumed if concrete is not compacted.
(b) Medium dense soil condition may be assumed if concrete is compacted well.

LOAD CARRYING CAPACITY OF PILES IN CLAY

Piles in cohesive soils, barring underground piles of large diameter, generally carry most of the load by virtue of the skin friction resistance developed on the pile shaft. The bearing capacity is normally calculated using total stress approach.

The ultimate load capacity of the pile is estimated as

$$Q_u = q_{pu}A_b + f_sA_s$$

In clays, $q_{pu} = C_{ub}N_c$
$f_s = C_a = \alpha\ C_u$
$\therefore\ Q_u = C_{ub}\ N_c\ A_b + \alpha\ C_u\ A_s --- (11)$

Where, C_{ub} = undrained cohesion at the base of the pile

Nc = Bearing capacity factor for deep foundation. For square and circular piles, $N_c = 9$.The pile must go atleast 5D inside the bearing stratum.

α = adhesion factor ; for a single pile, contact is between pile and soil; hence the adhesion factor is used

C_u = Undrained cohesion in the embedded length of the pile.

Accurate determination of α is very important. The value of α depends on undrained strength of the soil. Smaller the undrained strength of the soil, softer is the consistency of the soil and greater the tendency for the soil to adhere to the pile. For this case, α is close to 1.

For stiff clays, α can be as low as 0.3.

Under – Reamed Piles

An under reamed piles is one with an enlarged base or bulb. The bulb is called under ream. There could be one or more under – reams in a pile. Under – reamed piles are cast-in-situ piles, which may be installed both in sandy and clayey soils.

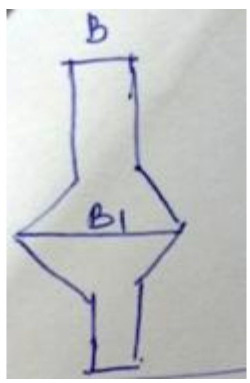
The ratio of bulb size to pile shaft size may be 2 to 3

$$\frac{Bulb\ size}{Pile\ shaft\ size} = 2\ to\ 3\ ; usually\ 2.5\ is\ used.$$

The bearing capacity of the pile increases because of increased bearing area; the more the number under-reams, the more the capacity.

Load carrying capacity of Under-reamed pile

Single under-reamed pile

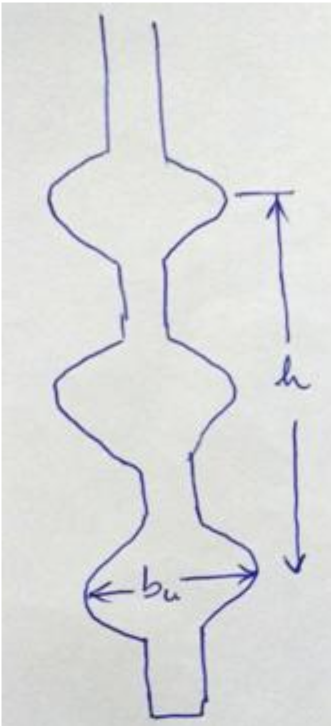


$$Q_{up} = Q_{eb} + Q_{fs} = q_b A_b + f_s A_s$$

q_b =	Unit point bearing capacity of the bulb
f_s =	Unit skin friction = $C_a = \alpha C$, $\alpha = 0.4$
A_b =	Area of section of the bulb
A_s =	Surface area of the embedded pile shaft

$$Q_u = \frac{\pi}{4} B^2 \,(9C) + \frac{\pi}{4}\left({B_1}^2 - B^2\right)(9C) + \alpha\, C' A_s'$$

For multi-under reamed pile



$$Q_{up} = q_b A_b + f_s A_s + \overline{f_s}\,\bar{A}_s$$

q_b =	Unit point resistance
f_s =	Unit skin friction
$\overline{f_s}$=	Unit frictional resistance between soil and soil
A_b =	Area of the section of the lowest bulb
A_s =	Surface area of the embedded portion of pile above the top bulb $= \pi {d_u}^2 h$
$\bar{A}_s$=	Surface area of a cylinder of diameter b_u and height equal to the distance between the center of the extreme bulbs

Allowable load

$$Q_{allowable} = \frac{Q_u}{FS}$$

FS remains between 2.5 to 4.0
Min FS = 2.5 (as per IS 2911)

DYNAMIC FORMULAE

The load carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving penetration.

Dynamic formulae zone fairly good results only in the case of free draining sands hard clays in which high pore water pressures do not develop during the driving of piles.

In saturated fine grained soils, high pore water pressure develops during the driving operation and the strength of the soil is considerably changed and the methods do not give reliable results.

The methods cannot be used for submerged, uniform fine sands which may be loose enough to become quick temporarily and show much less resistance.

The dynamic formulae are based on the assumption that the kinematic energy developed by the hammer during driving operation is equal to the work done on the pile. Thus,

$$Wh\eta_h = R\ x\ s$$

W = weight of hammer (kN)
h = height of ram drop (cm)
η_h= efficiency of pile hammer
R = pile resistance, taken equal to Q_u.
S = Pile penetration per blow (cm)

1. Engineering News Record Formula

$$Q_u = \frac{Wh\eta_h}{S + C}$$

S	=	Penetration of pile per hammer blow. It is generally based on the average penetration obtained from the last few blows (cm)
C	=	Constant
	=	2.54 cm for drop hammer
	=	0.254 cm for steam hammer (Single & double acting)

W x h can be replaced by the rated energy of hammer E_n (in kN – cm). Thus,

$$Q_u = \frac{E_n\eta_h}{s + C}$$

η_h	=	Efficiency of drop hammer = 0.7 to 0.9
η_h	=	0.75 to 0.85 for single acting and double acting hammer
η_h	=	0.80 to 0.9 for diesel hammer
FS	=	6

Modified ENR formula

$$Q_u = \frac{W h \eta_h}{S + C}\left(\frac{W + e^2 P}{W + P}\right)$$

P = weight of the pile
e = coefficient of Restitution

2. Hiley formula

$$Q_u = \frac{W h \eta_b \eta_h}{S + \frac{C}{2}}$$

η_h	=	Efficiency of hammer blow
h	=	Height of free fall of the rammer/hammer (cm)
S	=	Final set or penetration per blow (cm)
C	=	$C_1 + C_2 + C_3$
C_1	=	Temporary elastic compression of dolly and packing $=1.77 \frac{R}{A}$ for driving without dolly $=9.05 \frac{R}{A}$ for driving with short dolly
C_2	=	Temporary compression of pile $= 0.657 \frac{RL}{A}$ L = length of the pile
C3	=	Temporary compression of ground $= 3.55 \frac{R}{A}$ A = cross sectional area of pile in cm^2 R = pile resistance in tones L = Length of the pile in m

For W > e P, $\eta_b = \frac{W+e^2P}{W+P}$

For W < e P, $\eta_b = \frac{W+e^2P}{W+P} - \left(\frac{W-e\,P}{W+P}\right)^2$

*****ENR formula**

For drop hammers,

$$Q_a = \frac{WH}{6(S + 2.5)}$$

Single acting steam hammers

$$Q_a = \frac{WH}{6(S + 0.25)}$$

Q_a and W in kg,
H = fall of hammer or length of piston stroke in cm
S in cm / blow = avg. penetration for last 5 blows for drop hammer or 20 blows of steam hammer

$$Q_a = \frac{166.64E}{S + 2.54}$$

Q_a = allowable pile load in kN
E = energy per blow in kJ
S = avg. penetration in mm per blow for the final 150 mm of driving

3. Danish formula

$$Q_u = \frac{Wh\eta_h}{S + \frac{1}{2}S_0}$$

$$S_0 = \left[\frac{2\,\eta_h(W\ h\ L)}{A\ E}\right]^{\frac{1}{2}}$$

S_0 = elastic compression of pile
L = length of pile in cm
A = cross section area of pile (m^2)
E = Modulus of elasticity of pile material kN/m^2
Factor of safety: 3 to 4

Taking $Q_u = 3\ Q_u$
The final set (S) per blow is

$$S = \left(\frac{Wh\eta_h}{3Q_a}\right) - \frac{1}{2}S_0$$

Q_a = allowable load.

In-situ Penetration Tests for pile capacity

I. (SPT)

(i) For driven pile in sand, unit point or tip resistance (q_p) is related to Uncorrelated blow count N near the pile point as

$$q_p = 40\ N\ \left(\frac{D}{B}\right) \leq 400\ N$$

q_p = point resistance (kN/m^2)
D = Length of pile
B = Width (diameter) of pile
q_p is limited to 400 N.
f_s is related to the average value of the blow count ($\overline{N}$)

For high displacement piles, $f_s = 2.0\ \overline{N}\ kN/m^2$
For low displacement piles, $f_s = 1.0\ \overline{N}\ kN/m^2$

$\overline{N}$ = average of uncorrelated N values along the length of piles.

(ii) For piles in sand

$$q_p = 14\,N\left(\frac{D_b}{B}\right)\,kN/m^2$$

D_b= actual penetration into granular soil.

$f_s = 0.67\,\overline{N}$ kN/m^2

II. Dutch Cone test

$$\text{Point resistance, } q_p = \frac{q_c}{10}\left(\frac{D_b}{B}\right)$$

$$\text{Unit skin friction, } f_s = \frac{q_c}{200} \quad for\ dense\ sand$$

$$= \frac{q_c}{400} \quad for\ loose\ sand$$

$$= \frac{q_c}{150} \quad for\ silt$$

Group Action of piles

Piles are always used in a group. This is to ensure that the structural load from a member like a column or a wall lies within the zone of influence of the foundation.

Piles under a wall are arranged on either side of the centerline of the wall in a staggered formation.

The load is transferred to the piles in the group through a reinforced slab or beam, called as pile cap. The pile cap is structurally tied to the pile tops which help the group act as an integral unit.

The pile cap is standing clearly above the ground level, the pile group is called free standing pile group. In this case, the pile cap is kept away from direct contact with an expansive soil.

When the pile cap rests on the soil, the pile group is referred to as a piled foundation. In a piled foundation, the pile cap, under certain soil conditions, helps transmit a part of the load to the soil on which it rests.

When driving piles in sand, it is advisable to begin at the center of a group and then proceed outwards to avoid problems of ground tightening.

The requirement of driven piles that they be arranged in a group of atleast three piles is not necessarily applicable for bored piles.

Bored piles can be placed in vertical position fairly accurately and hence even one pile may be sufficient where the loads are light, eliminating the need for a pile cap also.

Ultimate load capacity of pile groups

The ultimate load capacity of a pile group is not necessary equal to the sum of the individual load capacities of the piles in the group.

The ratio of ultimate load capacity of the pile group, Q_{ng} , to the sum of individual load capacities of the piles in the group, is called group efficiency, η.

$$\eta = \frac{Q_{ug}}{nQ_u}$$

n = no. of piles in the group

Q_u= load capacity of one pile

Disturbance of soil during installation of piles and overlap of stress between the adjacent piles may cause the group capacity to be less than the sum of the individual capacities, making $\eta < 1$.

Generally, for smaller spacing's between piles, $\eta < 1$. For larger spacing's, the effect of pile interaction diminishes and η approaches unity.

In driven piles where the soil around the piles gets densified, as in loose to medium sand, η may even be more than 1.

The group efficiency (η) depends mainly on

(a) Spacing between the piles
(b) Type of soil in which the piles are installed
(c) Method of pile installation

Spacing between piles

When driven piles are spaced closely in dense soils or soft clays, the soil between the piles tends to move upwards and cause piles to be lifted up. On the other hard , large spacing's necessitate a bigger pile cap which is uneconomical. Bored cast-in-situ piles permit smaller spacing's, because then installation does not result in densification of soil around piles.

IS code recommendations for pile spacing (minimum)

2.5 D for end bearing piles
3.0 D for friction piles
2.0 D for piles in loose sands or fill deposits

D = shaft dia
D = dia of circumscribing circle for non – circular piles.

Piles groups in clay

A group of piles may fail in one of the following two ways.

(i) By Block failure
(ii) By individual pile failure

Block failure: A block failure occurs when piles are spaced less than 2 to 3 pile diameters.

Individual pile failure: This occurs for wider pile spacings. η approaches unity when the pile spacing is about diameters.

In block failure, the soil is bound by the perimeter of the pile block and the embedded length of pile acts as one unit or a block.

The ultimate load capacity of the pile group by block failure, Q_{ug} is given by

$$Q_{ug} = C_{ub} N_c A_b + P_b \, L \, C_u' \quad --- (1)$$

C_{ub} =	Undrained strength of clay at the base of the pile
C_u' =	Average undrained strength of clay along the length of the block
A_b =	Cross sectional area of the block
P_b =	Perimeter of the block
L =	Embedded length of the pile

Q_{ug} for individual pile failure is

$$Q_{ug} = n \, Q_u \quad --- (2)$$

The ultimate load capacity of the group is taken as smaller of the two values given by Eq.(1) and (2).

Pile group in sand

For piles groups in sand, η = 1 is commonly assumed in design.

$$Q_{ug} = n \, Q_u$$

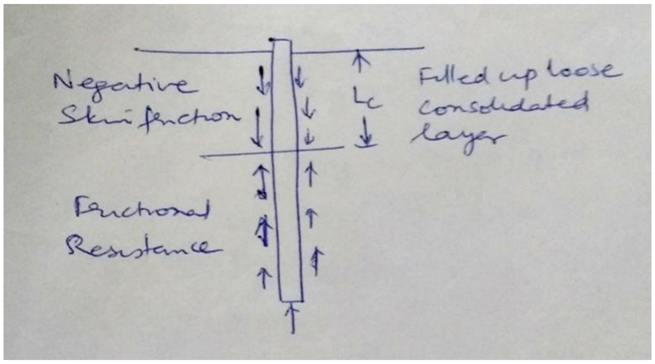
Negative skin friction

Piles installed in freshly placed fills of soft compressible deposits are subjected to a downward drag, as a consequence of the consolidation of the strata after the piles are installed.

This downward drag on the pile surface, when the soil moves down relative to the pile, adds to the structural loads and is called negative skin friction. Negative skin friction has the effect of reducing the allowable load on the pile.

Negative skin friction may also develop if the fill material is a loose sand deposit. It can also occur effective stress and thus causing consolidation of the soil resulting in downward drag on piles.

A small relative movement between the soil and the pile of about 10 mm is sufficient to mobilise full negative skin friction.



In bearing piles, when the settlement of the piles is negligible negative skin friction becomes a pile capacity problem.

For piles in compressible soils where pile capacity is contributed by both point resistance and shaft resistance, negative skin friction should be considered as a settlement problem.

Negative skin friction in Single piles
The magnitude of negative skin friction (F_n) for a single pile in a filled up soil is estimated as

Cohesive soil

$$F_n = PL_cC_a = P\,L_c\,\alpha\,C_u$$

P = Perimeter of the pile
L_c = Length of pile in compressible stratum
α = Adhesion factor
C_u = Undrained cohesion of the compressible layer

Cohesionless Soils

$$F_n = \frac{1}{2} P {L_c}^2 \gamma K \tan\delta$$

$$\delta = \frac{1}{2}\,\Phi \;\; to \;\; \frac{2}{3}\,\Phi$$

Negative skin friction in pile groups
When a pile group passes through a soft unconsolidated stratum, the magnitude of the negative skin friction, F_{ng} of the group may be estimated as

$$F_{ng} = n\,F_n$$
$$F_{ng} = C_u\,L_c\,P_g + \gamma\,L_c\,A_g$$

The higher of the values obtained from these equations should be used in the design

n = Number of piles in group
P_g = Perimeter of the group
γ = Unit weight of the soil with the pile group upto depth L_c
A_g = Area of the pile group within the perimeter, P_g

$$Factor\ of\ safety = \frac{ultimate\ load\ capacity\ of\ single\ pile\ or\ a\ group\ of\ piles}{working\ load + negative\ skinfriction\ load}$$

In the field, negative skin friction can be reduced in precast piles by painting the pile surface with bitumen.

LOAD TEST ON A PILE

VERTICAL LOAD TEST

Load test on a pile may be conducted on a driven pile or cast-in-situ pile, on a working pile or a test pile, and on a single pile or a group of piles.

A working pile is one which forms part of the foundation, while a test pile is one which is used primarily to check estimated capacities (as determined by the other methods). Test pile does not carry structural loads.

Both cohesive and cohesionless soils will have their properties altered by pile driving. In clays, the disturbance causes remoulding and consequent loss of strength with passage of time, much of the original strength will be regained.

The effect of pile driving in sand is to create a temporary condition wherein extra resistance is developed, which is lost later by stress relaxation.

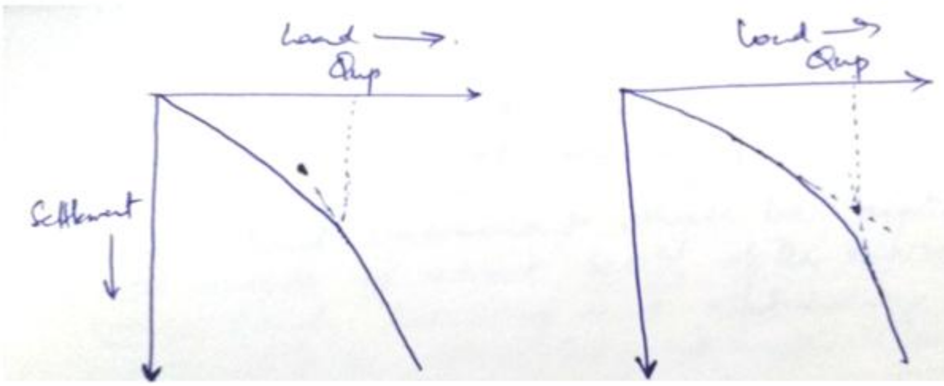
Hence, the pile load test should be conducted only after laps of few weeks in clay and atleast a few days in sand.

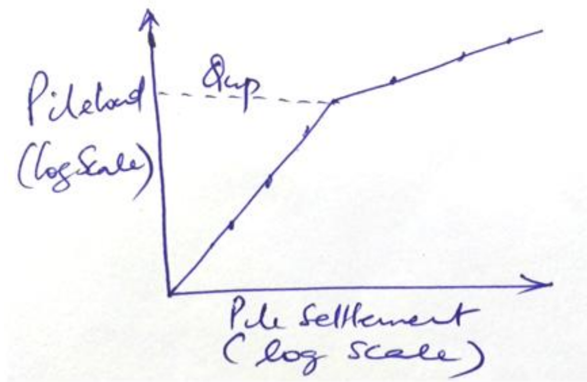
Loads may be applied by using a hydraulic jack against a supported platform, or against a reaction girder secured to anchor piles. Instead of reaction loading, gravity loading may also be used.

Measurement for pile settlement is related to a fixed reference mark. The reference mark has to be located outside the zone affected by pile movements.

The load is maintained slowly. About **5 to 8 equal increments are used until the load reaches about double the design value.** Time settlement data are recorded for each load increment. **Each increment** is maintained until the rate of **settlement becomes** a value **less than 0.25 mm / hour.** The final load is maintained for 24 hours.
The load settlement curve is obtained from the test data often; the definition of failure load is arbitrary.

The failure load may be taken when a predetermined amount of settlement has occurred or when the load – settlement plot is no longer a straight line.





Two categories of tests are usually carried out:

1. **Initial Test**: Carried out on test piles to estimate the allowable load or to predict settlement at a working load.
2. **Routine test**: Carried out as a check on working piles and to assess displacement corresponding to working load.

The allowable load on a single pile may be obtained as one of the following (IS 2911 - Part I)

1. 50% of ultimate load at which the total settlement is equal to $\frac{1}{10}$ of the pile dia.
2. Two thirds of load which causes a total settlement of 12 mm.
3. Two thirds of load which causes a net (plastic) settlement of 6mm (total settlement minus elastic settlement).

Pile group: Allowable load is lesser of

(i) Find load at which total settlement is 25 mm

(ii) 2/3 of final load at which total settlement attains a value of 40 mm.

VERTICAL CYCLIC PILE LOAD TEST

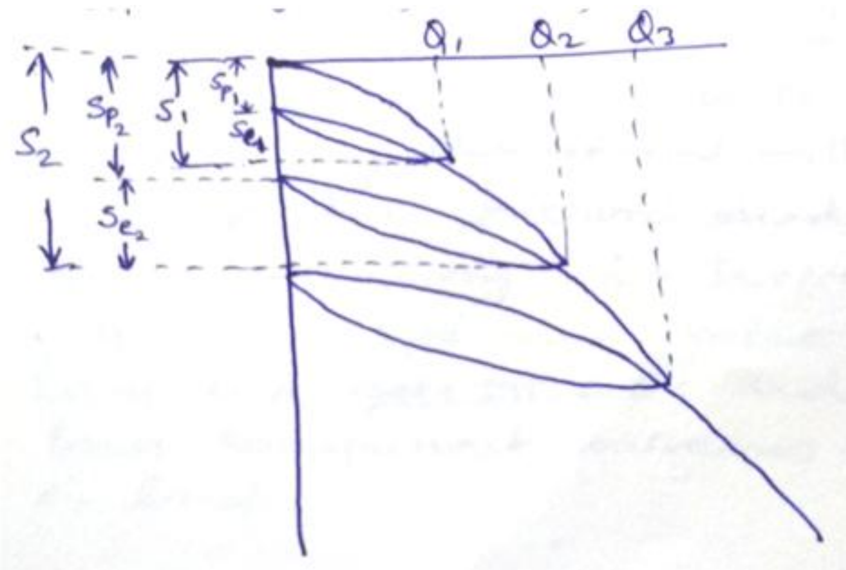
In the case of a cyclic load test, the load is raised up to a particular level, released to zero and again raised to a higher value and released to zero, i.e., loading and unloading is carried out alternatively.

Vertical cyclic pile load test is carried out when it is required to separate pile load into skin friction and point bearing on single piles of uniform diameter. It is generally limited to initial tests only.

The load increment shall be applied in increments of about 20% of the estimated safe load. Loading and unloading shall be carried out alternatively at each stage and the elastic rebound of the pile should be measured by atleast 3 dial gauges of 0.02 mm sensitivity.

The loading is continued upto $2\frac{1}{2}$ times the safe load or until total settlement of pile top equals to 10% of stem dia, whichever is earlier.

Settlements are recorded at each load increment or decrement. A typical plot of a cyclic load test data is shown below.



The total settlement 'S' of the pile head at any load may be written as

$$S = S_e + S_p$$

And $S = \Delta l + S_{es} + S_p$

S_e =	Elastic compression of pile and soil at base
S_p =	Plastic compression of soil at base
S_{es} =	Elastic compression of soil at base
ΔL =	Elastic compression of pile

*(plastic compression of pile is assumed negligible)

ΔL may be obtained from the equation

$$\Delta L = \frac{\left(Q - \frac{Q_f}{2}\right) L}{AE}$$

Q = load on pile
Q_f = frictional resistance component
L, A, E = length, c/s area and modulus of the elasticity of pile.

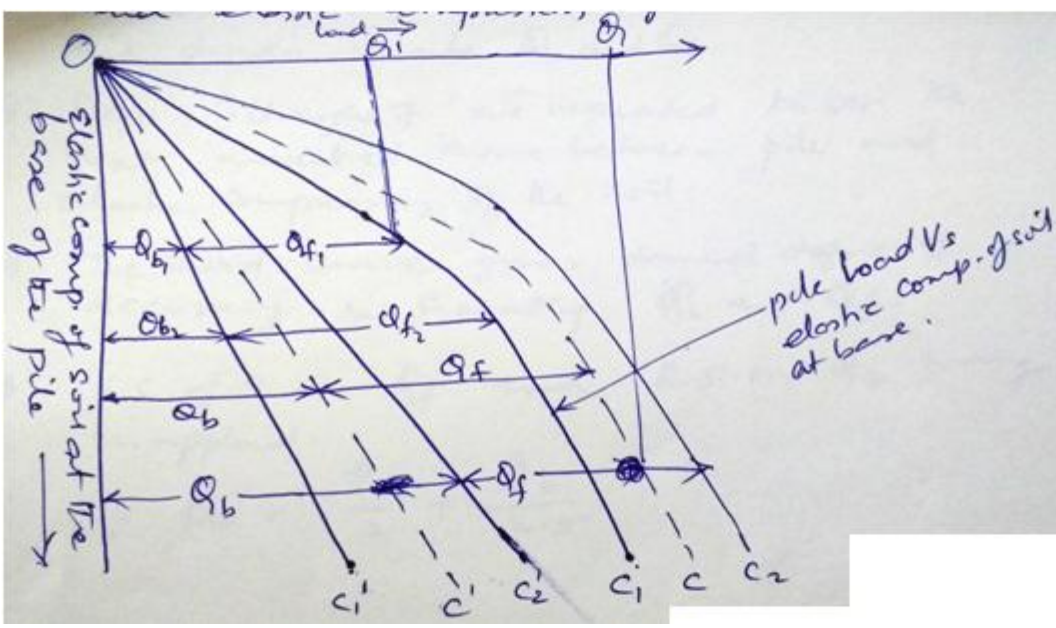
The total settlement may be separated into elastic and plastic components by removal of the load and observation of net settlement.

The elastic settlement of the soil at the base is obtained by subtracting the elastic settlement of pile from the total elastic component of settlement.

Separation of load into point bearing & frictional components (Van Wheeke, 1957)

Until the load reaches a certain value, the point bearing component will be zero. With increase in load, both friction and point bearing components go on increasing. The frictional component attains a maximum value at a particular load and thereafter point load component increases with further increase in load.

The point load component increases linearly with elastic compression of the soil at the base and the straight line showing this linear relationship is parallel to the straight line portion of the curve between the load on the curve and elastic compression of the soil.



1. Since Q_f is not known to start with, ΔL is assumed to be zero. Then elastic compressing of the soil at the base is equal to total elastic recovery of pile head.
   A curve OC, is drawn between pile load and the elastic compression of soil, calculated in this manner.

2. A line $OC_1^{'}$ is drawn from the origin O parallel to straight line portion of OC_1. This line is supposed to divide Q into Q_b and Q_f.
3. For different loads Q_1, Q_2, ….., Q_{f1}, Q_{f2} etc. are determined.
4. The value of ΔL corresponding to different values of Q_f are computed from

$$\Delta L = \frac{\left(Q - \frac{Q_f}{2}\right) L}{AE}$$

   ΔL = Elastic compression of pile
5. The elastic compression of soil are obtained by determining these values of ΔL from the corresponding elastic recovery of pile head.
6. A modified curve OC_2 is now drawn between pile load and elastic compression of soil.
7. Through origin, line OC_2' is drawn parallel to straight line portion of OC_2. This line divides Q into Q_b and Q_f.
8. Steps 3 through 7 are repeated to set the next modified curve between pile and elastic compression of the soil.
9. The third curve given designed degree of accuracy in separating Q_b and Q_f.
   FS of 2 on Q_f and 2.5 on Q_b may be applied.

$$\therefore Q_a = \frac{Q_f}{2} + \frac{Q_b}{2.5}$$

WELL FOUNDATIONS

Well foundations are deep foundations. Well foundations also called as Caissons, have been in use for foundations of bridges. Well foundation has its origin in India.

The term ‘Caisson’ is originally derived from the French word “Caisse” which means box or chest. Hence caisson means box like structure, rectangular or round, which is sunk from the surface of either land or water to the desired depth.

Well foundations provide a solid and massive foundation for heavy loads. These can be conveniently installed in a boulder stratum. Wells are used as foundations for bridge piers and abutments. They are useful as foundations where uplift loads are large as in the case of transmission line towers.

A caisson is advantageous compared to other type of when one or more of the following conditions exist:

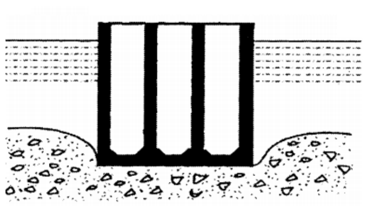
1. The soil contains large boulders which obstruct the penetration of piles or placement or placement of drilled piers (drilled piers are large diameter bored piles)
2. A massive substructure is required to extend below the river bed to resist the forces due to scour and/or floating objects.
3. Large magnitude of lateral forces are expected.

Types of caissons

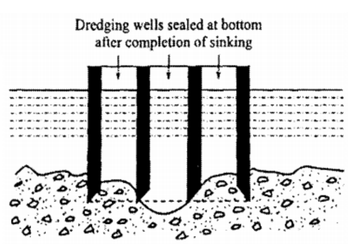
i. Floating or Box caisson: These are box shape, open both at the top and bottom during construction. This type of caisson is made of reinforced concrete, steel or timber. It is generally recommended when bearing stratum is available at shallow depth. The caisson is sunk in to position; a concrete seal is placed at its bottom in water. Finally, the inside is pumped dry and filled with concrete.

ii. Open caisson (wells): Open caisson is a box open both at top and bottom. It is made up of timber, concrete or steel. The open caisson is also called well.

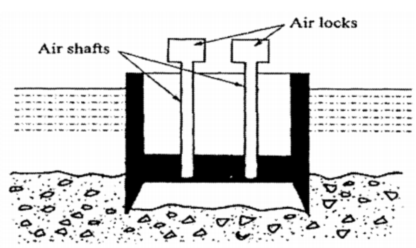
iii. Pneumatic caissons: These are of box shape, closed at the top, with a working chamber at the bottom from which water is kept off with the aid of compressed air. Thus, excavation is facilitated in the dry, and the caisson sinks as excavation proceeds. Finally, the working chamber is filled with concrete, upon reaching the final location at the desired depth.



Floating or Box Caisson



Open caisson

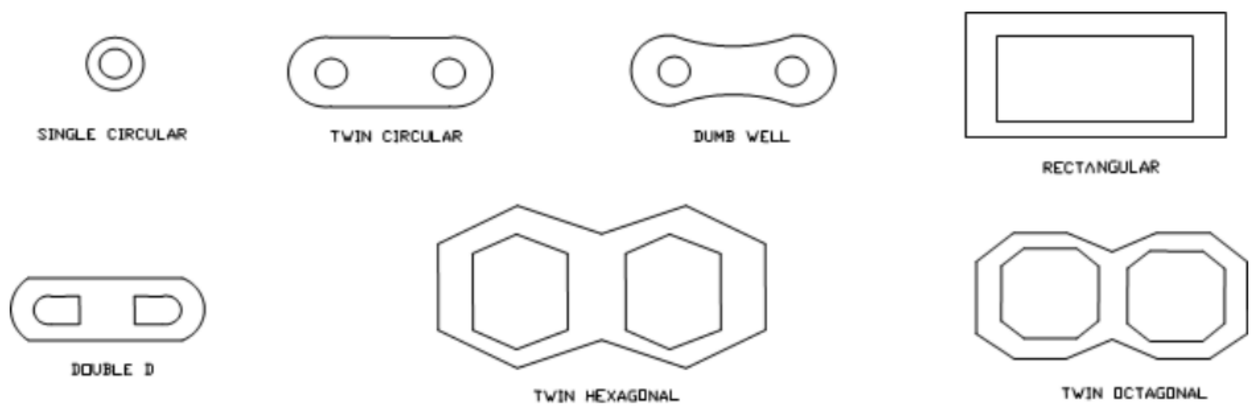


Pneumatic caisson

Different Shapes of Well

The common types of well shapes are

a) Single circular
b) Dumb well
c) Twin circular
d) Rectangular
e) Twin octagonal
f) Twin hexagonal
g) Double-D



Different shapes of well foundation

The choice of a particular shape of well depends upon the size of the pier, the considerations of tilt and the shift during sinking and the vertical and horizontal forces to which well is subjected. A circular type well has the minimum perimeter for a given dredge area. Since, perimeter will be equidistant at the points from the centre of dredge-hole; the sinking is more uniform as compared to the other shapes. In circular well a disadvantage is that in the direction parallel to the span of bridge, the diameter of the well is much more than required to accommodate minimum size of pier and hence circular well obstruct water way much in comparison to other shapes.

Oblong shape may be preferred for the superstructure either to avoid restriction of flow or for convenience of navigation. Circular or pointed shape may be preferred on the upstream side to minimize the possible impact from large and heavy floating objects.

Twin circular, twin –rectangular, twin-hexagonal, twin-octagonal and the double-D are used to support heavy loads from bridge piers

The size of a caisson is governed by the following factors:

1. Size of base: The size of caisson should be such that the caisson has a minimum projection of 0.3 m all-round the base of the superstructure; this would help take care of a reasonable amount of investable tilting and mislaignmnet.
2. Bearing pressure: The area required is obviously governed by the allowable bearing pressure of the soil.

3. Practical minimum size: A minimum size of 2.5 m is considered necessary from the point of view of convenience in sinking and economy in construction; smaller sizes of caisson frequently prove to be more expensive than other type of deep foundaions

Components of well foundation

The following are the components of well foundation

1. Cutting edge
2. Curb
3. Concrete Seal or bottom
4. Steining
5. Top plug
6. Well Cap

Cutting edge: The function of cutting edge is to facilitate easy penetration or sinking into the soil to the desired depth. As it has to cut through the soil , it should be as sharp as possible, and strong enough to resist the high stresses to which of is subjected during the sinking process. Hence it usually consists of an angle iron with or without additional plate of structural steel.

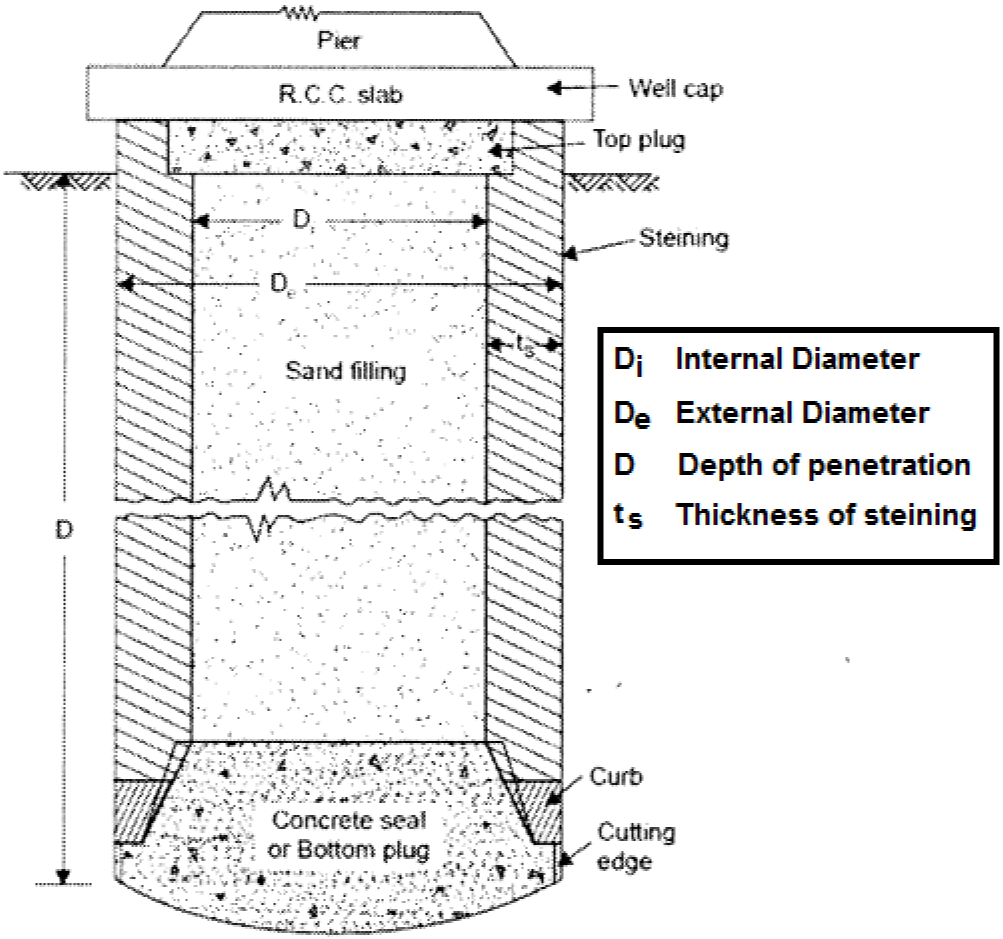
Curb: The well curb is a transition member between the sharp cutting edge and the thick steining. It is thus tapering in shape. It is usually made of reinforced concrete as it is subjected to severe stresses during the sinking process.

Concrete seal or bottom plug: After the well foundation is sunk to the required depth so as to rest on a firm stratum, a thick layer of concrete is provided at the bottom inside the well, generally under water. This layer is called the concrete seal or bottom plug, which serves a s the base for the well foundation. This is primarily meant to distribute the loads on to a large area of the foundation and hence may be omitted when the well is made to rest on hard rock.

Steining: The steining forms the bulk of the foundation and may be occasionally be constructed with brick or stone masonry, or with plain or reinforced concrete occasionally. The thickness of the steining is made uniform throughout the depth. It is considered desirable to provide vertical reinforcements to take care of the tensile stresses which might occur when the well is suspended from top during any stage of sinking.

Top plug: After the well foundation is sunk to the desired depth, the inside of the well is filled with sand either partly or fully, and a top layer of concrete is placed. This is known as the top plug. The sand filling serves to increase the stiffness of the well foundation. However, this adds to the weight and load transmitted to the foundation stratum. Therefore, the desirability or otherwise of providing the sand filling from the point of view of bearing power and settlement should be ascertained. The top plug of concrete serves to transmit the loads to the base in a uniform manner.

Well cap: The well cap serves as a bearing pad to the superstructure, which may be a pier or an abutment. It distributes the superstructure load onto the well steining uniformly.



Depth of well foundation

- In order to resist the lateral forces acting on the well foundation, it should be placed at sufficient depth below the maximum scour level
- Depth of well foundation is chosen by considering the grip length and bearing capacity.
- The depth of the bottom of the well below the maximum scour level is known as grip length.
- The maximum and minimum pressure developed at the base of the well should be within the permissible limit.

The depth of scour can be ascertained by sounding near the proposed site immediately after flood. The maximum scour would be greater than measured scour as design discharge is greater than flood discharge and there is increase in velocity due to obstruction to flow by construction of bridge.

The normal depth of scour may also be calculated by Lacey's formula:

$$d = 0.473(Q/f)^{1/3}$$

Where d = normal scour depth, measured below high flood level (m)

Q = design discharge (m^3/s)

f = Lacey's silt factor

The silt factor f may be calculated from the equation

$$f = 1.76\sqrt{d_m}$$

where d_m is the mean size of particle.

The grip length for wells of railway bridges is generally taken as 50% of maximum scour depth while for road bridges 30% of maximum scour depth is considered adequate. The base of the well is usually taken to a depth of 2.67d below the high flood level (HFL)

According to IS:3955-1967, the depth should not be less than 1.33 times the maximum scour depth. The depth of the base of the well below the scour level is kept not less than 2m for piers and abutments with arches and 1.2 m for piers and abutments supporting other types of structures.

List of forces to be considered while designing a well foundation are as follows:

1. **Dead loads:** it includes weight of superstructure (pier/abutment) + self weight of well.
2. **Live loads:** The design live loads incase of railway bridges are extracted from I dial Railway bridge rule and codes. For road bridges, the live loads may be specified via standard specifications and code of practice for road bridges.
3. **Impact loads:** the impact loads is the result of live load and shall be considered only during the design of a pier cap and the bridge seat on the abutment. However, for other components of the well this effect shall be neglected.
4. **Wind loads:** the wind loads shall be seen only on the exposed area in elevation and hence acts laterally on the bridge.
5. **Water pressure:** the water pressure due to water current is acted on the portions of substructure that lies between the water level and the maximum scour level. In case of piers lying parallel to the direction of water, the intensity of water shall be determined by, **P = kV2** Where, p = intensity of pressure (KN/m2), k = constant that depend upon the shape of well. Maximum — 0.788 for square ended piers, Minimum — 0.237 for piers having cut and ease/clam water, V- –Velocity of current/water flow (m/s), An assumption is made that V2 is maximum at free surface of water and zero at the deepest scour level. The velocity at surface is assumed to be ?2 times the average velocity.
6. **Longitudinal forces:** longitudinal forces results from tractive and braking forces. The longitudinal forces depend on the type of vehicles and bearing. These forces get transferred/transmitted into the substructure via fixed bearings and friction in movable bearings.
7. **Centrifugal force**
8. **Buoyant forces:** the buoyancy tends to decrease the effective weight of well. For masonry/concrete steining 15-20% buoyancy is assumed to account for the porousness.
9. **Earth pressure:** The Rankine' s theory and Coulombs theory is utilized to calculate the earth pressure.
10. **Temperature stresses:** the longitudinal forces are resulted because of the temperature changes. The movements caused by temperature changes are partly restrained in the girder bridges due to friction at the moveable end.

11. **Seismic forces:** seismic forces are vital when the wells are constructed in seismic zones. The seismic forces act on every members of the superstructure. These forces is principally determined as, ?W, where w= weight of component and ?=seismic coefficient which depends upon the type of seismic zone and its value shall extracted from code. Usually taken between 0.01-0.08. This seismic force is characterized to cat through the C.G. of the component. These may act in any direction but generally assumed to act in one direction at a time.

Construction and sinking of wells

Construction of well curb

If the river bed is dry the cutting edge over which the well curb is to built is placed in the correct position after excavating the bed for about 150 mm for seating. For small depth of water, sand island is created before placing the curb. In case, the depth is more, the curb is built on the bank and floated to the site.

Construction of well steining

The steining is constructed with a height of 1.5 m at a time and sinking done after allowing for at least 24 hours for setting. Once the well has acquired a grip length of 6 m into the ground, steining can be raised 3m at a time.

Sinking process

Well steining is commenced after the curb is cast and the first stage of steining is ready after curing. The soil is excavated from inside manually or mechanically. Manual dredging is feasible when the depth of water inside the well is not more than 1 m. When the depth of water is small, an automatic grab operated by diesel winches is used. Blasting with explosives is used when weak rock is encountered. Additional loading known as kentledge is used if necessary. Kentledge is generally in the form of sand bags placed on a suitable platform on top of the well. Water jetting on the exterior face is applied along with kentledge.

Shifts and tilts

The well should sunk straight and vertical at the correct position. Sometimes, the well tilts on to one side or its shifts away from the desired position. The following precautions may be taken to avoid tilts and shifts.

1. The outer surface of the well curb and steining should be smooth.
2. The curb diameter should be kept 40 to 80 mm larger than the outer diameter of the steining.
3. The cutting edge should be uniformly thick and sharp.
4. Dredging should be done uniformly on all sides and in all pockets.

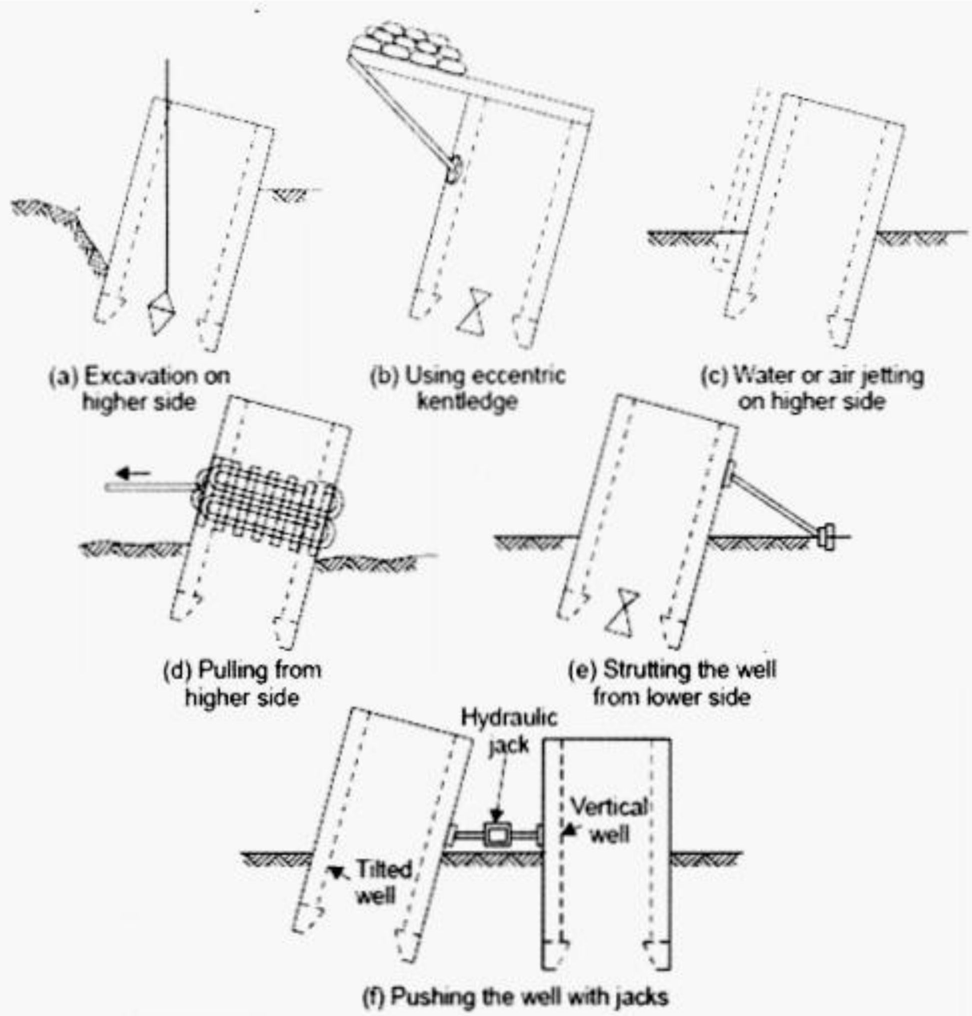
Tilts should be generally limited to 1 in 60. The shift should be restricted to 1 % of depth sunk.

Remedial measures for rectification of tilts and shifts

1. Regulation of excavation
   The higher side is grabbed more by regulating the dredging in the initial stages otherwise well may be de-watered if possible and open excavation may be carried out on the higher side.
2. Eccentric loading

Eccentric placing of the kentledge may be resorted to provide a greater sinking effort on the higher side as the depth of sinking increases. Heavier kentledge with greater eccentricity would be required to rectify the fault.

3. Water jetting
   If water jets are applied on the outer face of the well on the higher side, friction is reduced on that side and tilt may be reduced.
4. Excavation under the cutting edge
   If hard clay is encountered, open excavation is done under the cutting edge, if de-watering is possible; if not, divers may be employed to loosen the strata.
5. Insertion of wooden sleepers under the cutting edge
   Wooden sleepers or hooks may be inserted temporarily below the cutting edge on the lower side to avoid further tilt.
6. Pulling the well
   In the early stages of sinking, pulling the well to the higher side by placing one or more steel ropes round the well with vertical sleepers packed in between to distribute pressure over larger areas of well steining is effective.
7. Strutting the well
   The well is strutted on its tilted side with suitable logs of wood to prevent further tilt. The well steining is provided with sleeper to distribute the load from the strut. The other end of log rests against a form base having driven piles.
8. Pushing the well with jack
   Tilt can be rectified by pushing the wells by suitably arranging mechanical or hydraulic jacks.



Remedial measures for correction of tilt of wells