Soil Exploration

Definition of soil exploration :

An accurate assessment of the characteristics and engineering properties of the soils at a site is essential for proper design and successful construction of any structure at the site. The field and laboratory investigations required to obtain the necessary data for the soils for this purpose are collectively called soil exploration.

Need for soil exploration :

The choice of the foundation and its depth, the bearing capacity, settlement analysis and such other important aspects depend very much upon the various engineering properties of the foundation soils involved.

Soil exploration may be needed not only for the design and construction of new structures, but also for deciding upon remedial measures if a structure shows signs of distress after construction. The design and construction of highway and airport pavements will also depend upon the characteristics of the soil strata upon which they are to be aligned.

Objectives of soil exploration :

The primary objectives of soil exploration are :

(i) Determination of the nature of the deposits of soil,

(ii) Determination of the depth and thickness of the various soil strata and their extent in the horizontal direction,

(iii) The location of groundwater and fluctuations in GWT,

(iv) Obtaining soil and rock samples from the various strata,

(v) The determination of the engineering properties of the soil and rock strata that affect the performance of the structure, and

(vi) Determination of the in-situ properties by performing field tests.

Site Investigation :

'Site investigation refers to the procedure of determining surface and subsurface conditions in the area of proposed construction. Thus, this term has a broader connotation than 'soil exploration', and includes the latter.

Usually the cost of a thorough investigation and exploration programme will be less than 1% of the total cost of the entire project.

Site investigations may involve one or more of the following preliminary steps :

1. Reconnaissance

2. Study of maps

3. Aerial photography.

Methods of soil exploration :

The subsoil exploration should enable the engineer to draw the soil profile indicating the sequence of the strata and the properties of the soil involved.

In general, the methods available for soil exploration may be classified as follows :

1. Direct methods… Test pits, trial pits or trenches

2. Semi-direct methods … Borings

3. Indirect methods … Soundings or penetration tests and geophysical methods

In an exploratory programme, one or more of these methods may be used to yield (or to get) the desired information.

Test pits

Test pits or trenches are open type or accessible exploratory methods. Soils can be inspected in their natural condition. The necessary soils samples may be obtained by sampling techniques and used for ascertaining strength and other engineering properties by appropriate laboratory tests.

Test pits will also be useful for conducting field tests such as the plate – bearing test.

Test pits are considered suitable only for small depths – up to 3m; the cost of these tests increases rapidly with the depth. For greater depths, especially in pervious soils lateral supports or bracing of the excavations will be necessary. Ground water table may also be encountered and may have to be lowered.

Hence, test pits are usually made only for supplementing other methods or for minor structures.

Boring

Making or drilling bore holes into the ground with a view to obtaining soil or rock samples from specified or from known depths is called ‘boring’.

The common methods of advancing bore holes are:

1. Auger boring

2. Auger and shell boring

3. Wash boring

4. Percussion drilling

5. Rotary drilling

4 and 5 are More commonly employed for sampling in rock strata.

Auger boring

‘Soil auger’ is a device that is useful for advancing a bore hole into the ground. Augers may be i) hand – operated: these are used for relatively small depths (less than 3m to 5m)

ii) power- driven: these *are used for greater* depths.

The soil auger is advanced by rotating it while pressing it into the soil at the same time. It is used primarily in soils in which the bore hole can be kept dry and unsupported. As soon as the auger gets filled with soil, it is taken out and the soil sample collected.

Two common types of augers, the post hole auger and the helical auger are shown in figures.

Auger and Shell Boring

If the sides of the hole cannot remain unsupported, he soil is prevented from falling in by means of a pipe known as ‘shell’ or ‘casing’. The casing is to be driven first and then the auger; whenever the casing is to be extended, the auger has to be withdrawn, this being an impediment to quick progress of the work.

An equipment called a ‘boring rig’ is employed for power – driven augers, which may be used up to 50m depth (A hand rig may be sufficient for borings up to 25m in depth). Casings may be used for sands or stiff clays. Soft rock or gravel can be broken by chisel bits attached to drill rods. Sand pumps are used in the case of sandy soils.

Wash boring

Wash boring is commonly used for exploration below ground water table for which the auger method is unsuitable. This method may be used in all kinds of soil except those mixed with gravel and boulders. The set-up for wash boring is shown in Fig.

Initially, the hole is advanced for a short depth by using an auger. A casing pipe is pushed in and driven with a drop weight. The driving may be with the aid of power. A hollow drill bit is screwed to a hollow drill rod connected to a rope passing over a pulley and supported by a tripod. Water jet under pressure is forced through the rod and the bit into the hole. This loosens the soil at the lower end and forces the soil-water suspension upwards along the annular surface between the rod and the side of the hole. This suspension is led to a settling tank where the soil particles settle while the water overflows into a sump. The water collected in the sump is used for circulation again.

The soil particles collected represent a very disturbed sample and is not very useful for the evaluation of the engineering properties. Wash borings are primarily used for advancing bore holes; whenever a soil sample is required, the chopping bit is to be replaced by a sampler.

The change of the rate of progress and change of colour or wash water indicate changes in soil strata.

Percussion Drilling

A heavy drill bit called ‘churn bit’ is suspended from a drill rod or a cable and is driven by repeated blows. Water is added to facilitate the breaking of stiff soil or rock. The slurry of the pulverized material is bailed out at intervals. The method cannot be used in loose sand and is slow in plastic clay.

The formation gets badly disturbed by impact.

Rotary Drilling

This method is fast in rock formations. A drill bit, fixed to the lower end of a drill rod, is rotated by power while being kept in firm contact with the hole. Drilling fluid or bentonite slurry is forced under pressure through the drill rod and it comes up bringing the cuttings to the surface. Even rock cores may be obtained by using suitable diamond drill bits. This method is not used in porous deposits as the consumption of drilling fluid would be prohibitively high.

Planning of programme and preparation of soil investigation report :

A report is the final document of the whole exercise of soil exploration. A report should be comprehensive, clear and to the point. Many can write reports, but only very few can produce a good report. A report writer should be knowledgeable, practical and pragmatic. No theory, books or codes of practice can provide all information required to produce a good report. It is only the experience of a number of years of dedicated service in the field that helps a geotechnical consultant to make the report writing an art.

A good soil exploration report should normally comprise the following:

1. Introduction, which includes the scope of the investigation.
2. Description of the proposed structure, the location and the geological conditions at the site.
3. Details of the field exploration programme, indicating the number of borings, their location and depths.
4. Details of the methods of exploration
5. General description of the sub-soil conditions as obtained from in-situ tests, such as standard penetration test and cone penetration test.
6. Details of the laboratory tests conducted on the soil samples collected and the results obtained.
7. Depth of the ground water table and the changes in water levels.
8. Analysis and discussion of the test results.
9. Recommendations about the allowable bearing pressure, the type of foundation of structure.
10. Calculations for determining safe bearing pressure, pile loads, etc.
11. Tables containing bore logs, and other field and laboratory test results.
12. Drawings which include an index plan, a site-plan, test results plotted in the form of charts and graphs, soil profiles, etc.
13. Conclusions. The main findings of investigations should be clearly stated. It should be brief but should mention the salient points.

Field tests:

Penetration test:

Plate load test:

Pressure meter:

Shallow Foundations

Introductory Concepts on Foundations :

The ultimate support for any structure is provided by the underlying earth or soil material and, therefore, the stability of the structure depends on it. Since soil is usually much weaker than other common materials of construction, such as steel and concrete, a greater area or volume of soil is necessarily involved in order to satisfactorily carry a given loading. Thus, in order to impart the loads carried by structural members of steel or concrete to soil, a load transfer device is necessary. The structural foundation serves the purpose of such a device. A foundation is supposed to transmit the structural loading to the supporting soil in such a way that the soil is not overstressed and that serious settlements of the structure are not caused (Chapter 14). The type of foundation utilized is closely related to the properties of the supporting soil, since the performance of the foundation is based on that of the soil, in addition to its own. Thus, it is important to recognize that it is the soil-foundation system that provides support for the structure; the components of this system should not be viewed separately. The foundation is an element that is built and installed, while the soil is the natural earth material which exists at the site.

Since the stability of structure is dependent upon the soil-foundation system, all forces that may act on the structure during its lifetime should be considered. In fact, it is the worst combination of these that must be considered for design. Typically, foundation design always includes the effect of dead loads plus the live loads on the structures. Other miscellaneous forces that may have to be considered result from the action of wind, water, heat ice, frost, earthquake and explosive blasts.

General types of foundations:

The various types of structural foundations may be grouped into two broad categories – shallow foundations and deep foundations. The classification indicates the depth of the foundation relative to its size and the depth of the soil providing most of the support. According to Terzaghi, a foundation is shallow if its depth is equal to or less than its width and deep when it exceeds the width.

The 'floating foundation', a special category, is not actually a different type, but it represents a special application of a soil mechanics principle to a combination of raft-caisson foundation, explained later.

A short description of these with pictorial representation will now be given.

Spread footings

Spread footing foundation is basically a pad used to "spread out" loads from walls or columns over a sufficiently large area of foundation soil. These are constructed as close to the ground surface as possible consistent with the design requirements, and with factors such as frost penetration depth and possibility of soil erosion. Footings for permanent structures are rarely located directly on the ground surface. A spread footing needs not necessarily e at small depths; it may be located deep in the ground if the soil conditions or design criteria require.

Spread footing required to support a wall is known as a continuous, wall, or strip footing, while that required to support a column is known as an individual or an isolated footing.

An isolated footing may be square, circular, or rectangular in shape in plan, depending upon factors such as the plan shape of the column and constraints of space.

If the footing support more than one column or wall, it will be a strap footing, combined footing or a raft foundation.

The common types of spread footings referred to above are shown in Fig. 15.2. Two miscellaneous types the monolithic footing, used for watertight basement (also for resisting uplift), and the grillage foundation, used for heavy loads are also shown.

Strap footings

A ‘strap footing’ comprises two or more footings connected by a beam called ‘strap’. This is also called a ‘cantilever footings’ or ‘pump-handle foundation’. This may be required when the footing of an exterior column cannot extent into and adjoining private property. Common types of strap beam arrangements are shown in Fig. 15.3.

Combined footings

A combined footing support two or are columns in a row when the areas required for individual footings are such that they come very near each other. They are also preferred in situations of limited space on one side owing to the existence of the boundary line of private property.

The plan shape of the footing may be rectangular or trapezoidal; the footing will then be called ‘rectangular combined footing’ or ‘trapezoidal combined footing’, as the case may be. These are shown in Fig. 15.4.

Raft foundations (Mats)

A raft or mat foundation is a large footing, usually supporting walls as well as several columns in two or more rows. This is adopted when individual column footings would tend to be too close or tend to overlap; further, this is considered suitable when differential settlements arising out of footings on weak soils are to be minimized. A typical mat or raft shown Fig. 15.5.

Deep footings

According to Terzaghi, if the depth of a footing is less than or equal to the width, it may be considered a shallow foundation. Theories of bearing capacity have been considered for these in Chapter 14. However, if the depth is more, the footings are considered as deep footings (Fig. 15.6); Meyerhof (1951) developed the theory of bearing capacity for such footings.

Pile foundations

Pile foundations are intended to transmit structural loads through zones of poor soil to a depth where the soil has the desired capacity to transmit the loads. They are somewhat similar to columns in that loads developed at one level are transmitted to a lower level; but piles obtain lateral support from the soil in which they are embedded so that there is no concern with regard to buckling and, it is in this respect that they differ from columns. Piles are slender foundations units which are usually driven into place. They may also be cast-in-place (Fig. 15.7).

A pile foundation usually consists of a number of piles, which together support a structure. The piles may be driven or placed vertically or with a batter. More detailed treatment of this type of foundation is given in Chapter 16.

Pier foundations

Pier foundations are somewhat similar to pile foundations but are typically larger in area in area than piles. An opening is drilled to the desired depth and concrete is poured to make a pier foundation (Fig. 15.8). Much distinction is now being lost between the pile foundation and pier foundations, adjectives such as ‘driven’, ‘bored’, or ‘drilled’, and ‘precast’ and ‘cast-in-situ’, being used to indicate the method of installation and construction. Usually, pier foundations are used for bridges.

Caissons (Wells)

A caisson is a structural box or chamber that is sunk into place or built in place by systematic excavation below the bottom. Caissons are classified as ‘open’ caissons, ‘pneumatic’ caissons, and ‘box’ or ‘floating’ caissons. Open caissons may be box-type of pile-type.

The top and bottom are open during installation for open caissons. The bottom may be finally sealed with concrete or may be anchored into rock.

Pneumatic caisson is one in which compressed air is used to keep water from entering the working chamber, the top of the caisson is closed. Excavation and concreting is facilitated to be carried out in the dry. The caisson is sunk deeper as the excavation proceeds and on reaching the final position, the working chamber is filled with concrete.

Box or floating caisson is one in which the bottom is closed. It is cast on land and towed to the site and launched in water, after the concrete has not cured. It is sunk into position by filling the inside with sand, gravel, concrete or water. False bottoms or temporary bases of timber are sometimes used for floating the caisson to the site. The various types of caissons are shown in Fig. 15.9.

Floating foundation

The floating foundation is a special type of foundation construction useful in locations where deep deposits of compressible cohesive soils exist and the use of piles is impractical. The concept of a floating foundation requires that the substructure be assembled as a combination of a raft and caisson to create a rigid box as shown in Fig. 15.10.

The foundation is installed at such a depth that the total weight of the soil excavated for the rigid box equals the total weight of the planned structure. Theoretically speaking, therefore, the soil below the structure is not subjected to any increase in stress; consequently, no settlement is to be expected. However, some settlement does occur usually because the soil at the bottom of the excavation expands after excavation and gets recompressed during and after construction.

Choice of foundation type and preliminary selection

The type of foundation most appropriate for a given structure depends upon several factors : (i) view function of the structure and the loads it must carry, (ii) the subsurface conditions, (iii) the cost of the foundation in comparison with the cost of the superstructure.

These are the principal factors, although several other considerations may also enter into the picture

There is usually more than one acceptable solution to every foundation problem in view of the interplay of several factors. Judgment also plays an important part. Foundation design is enriched by scientific and engineering developments; however, a strictly scientific procedure may not be possible for practicing the art of foundation design and construction.

The following are the essential steps involved in the final choice of the type of foundation:

1. Information regarding the nature of the superstructure and the probable loading is required, at least in a general way.

2. The approximate subsurface conditions or soil profile is to be ascertained.

3. Each of the customary types of foundation is considered briefly to judge whether it is suitable under the existing conditions from the point of view of the criteria for stability – bearing capacity and settlement. The obviously un suitable types may be eliminated, thus narrowing down the choice.

4. More detailed studies, including tentative designs, of the more promising types are made in the next phase.

5. Final selection of the type of foundation is made based on the cost – the most acceptable compromise between cost and performance.

The design engineer may sometimes be guided by the successful foundations in the neighborhood. Besides the two well known criteria for stability of foundations – bearing capacity and settlement – the depth at which the foundation is to be placed, is another important aspect.

For small loading on good soils, spread footings could be selected. For columns, individual footings are chosen unless they come too close to one another, in which case, combined footings are used.

For a series of closely spaced columns or walls, continuous footings are the obvious choice. When the footings for rows of columns come too close to one another, a raft foundation will be the obvious choice. In fact, when the area of all the footings appears to be more than 50 per cent of the area of the structure in plan, a raft should be considered. The total load it can take will be substantially greater than footings for the same permissible differential settlement.

In case a shallow foundation does not answer the problem on hand, in spite of choosing a reasonable depth for the foundation, some type of deep foundation may be required. A pier foundation is justified in the case of very heavy loading as in bridges. Piles, in effect, are slender piers, which are used to bypass weak strata and transmit loading to hard strata below. As an alternative to raft foundation, the economics of bored piles is considered.

After the preliminary selection of the type of the foundation is made, the next step is to evaluate the distribution of pressure, settlement, and bearing capacity.

Certain guidelines are given in table 15.1 with regard to the selection of the type of foundation based on soil conditions at a site. For the design comments it is assumed that a multistory commercial structure, such as an office building, is to be constructed.

Spread Footings

Spread footings are the most widely used type among all foundations be because they are usually more economical than others. Least amount of equipment and skill are required for the construction of spread footings. Further, the conditions of the footings and the supporting soil can be readily examined.

Other types of foundations are more favorable when the soil has a very low bearing capacity or when excessive settlements are expected to result due to the presence of compressible strata within the active zone.

Common Types of Spread Footings

A spread footing is a type of shallow foundation used to support a wall or a column. In the former case, it is called a continuous or wall footing and in the latter, it is called an isolated or individual footing. The commonly used variations of individual footings are illustrated in Fig. 15.11.

The base area of the footing is governed by the bearing capacity of the soil. The plain footing is usually of reinforced concrete and is used to support a reinforced concrete column. The mass concrete footing is used to support a steel column. Usually the sloped footing will be of the same material as that for the column; alternatively, it can be of reinforced concrete. The stepped footing is used either for a column or for a wall. All the steps may be of concrete or the bottom most step alone may be of concrete, the others being of the same material as for the column.

Depth of Footings

The important criteria for deciding upon the depth at which footings have to be installed may be set out as follows:

1. Footings should be taken below the top (organic) soil, miscellaneous fill, debris or muck.

If the thickness of the top soil is large, two alternatives are available:

(a) Removing the top soil under the footing and replacing it with lean concrete; and (b) removing the top soil in an area larger than the footing and replacing it with compacted sand and gravel; the area of this compacted fill should be sufficiently large to distribute the loads from the footing on to a larger area.

The choice between these two alternatives, which are shown in Fig. 15.12 (a) and (b) will depend upon the time available and relative economy.

2. Footings should be taken below the depth of frost penetration. Interior footings in heated buildings in cold countries will not be affected by frost. The minimum depths of footings from this criterion are usually specified in the load building codes of large cities in countries in which frost is a significant factor in foundation design.

The damage due to frost action is caused by the volume change of water in the soil at freezing temperatures. Gravel and coarse sand above water level, containing less than 3%

fines, cannot hold water and consequently are not subjected to frost action. Other soils are subjected to frost-heave within the depth of frost penetration.

In tropical countries like India, frost is not a problem except in very few areas like the Himalayan region.

3. Footings should be taken below the possible depth of erosion due to natural causes like surface water runoff. The minimum depth of footings on this count is usually taken as 30 cm for single and two-storey constructions, while it is taken as 60 cm for heavier construction.
4. Footings on sloping ground be constructed with a sufficient edge distance (minimum 60cm to 90 cm) for protecting against erosion.
5. The difference in elevation between footings should not be so great as to introduce undesirable overlapping of stresses in soil the guideline used for this is that the maximum difference in elevation should be maintained equal to the clear distance between two footings in the case of rock and equal to half the clear distance between two footings in the case of soil. This is also necessary to prevent disturbance of soil under the higher footings= due to the excavation for the lower footing.

Bearing Capacity of Soils under Footings

Granular soils

Bearing capacity of granular soils depends upon the unit weight γ and the angle of internal friction Φ of the soil, both of which vary primarily with the density index of the soil. Dense soils have large values of γ and Φ consequently high hearing capacity. Loose soils, on the other hand, have small γ and Φ values and low bearing power.

The density index of granular soil *in–situ* is generally determined by standard penetration tests. The relationship between *N*-values and Φ – values established empirically by Peck, Hanson and Thorn burn may be used, and later the relevant Terzaghi equations may be applied to get the bearing capacity.

In conventional design, the allowable bearing capacity should be taken as the smaller of the following two values:

(i) Bearing capacity based on shear failure: This is the ultimate bearing capacity divided by a suitable factor of safety; usually a value of 3 is used for normal loading and 2 for maximum load. Empirical equations for bearing capacity in terms of N – value may be used.

(ii) Allowable bearing pressure based on tolerable settlement: Empirical equation given by Terzaghi and Peck may be used in terms of N – values – for the net allowable bearing pressure.

The value may be modified by using the linear relationship with permissible settlement, if it is desired for a different value of the permissible settlement.

If D_f / b > 1, the value is obtained multiplying by the factor (1+ D_f / b.

The allowable bearing pressure is taken as the smaller of (i) and (ii) finally.

Cohesive soils

The ultimate bearing capacity of cohesive soils depends primarily on their shear strength (or consistency). This may be determined by any one of the following:

(i) Standard penetration tests: For conservative design of small jobs, the correlation between standard penetration value, consistency and allowable bearing, capacity given by Terzaghi and Peck (Chapter 14) may be used.

(ii) Unconfined compression tests: For medium jobs, the shear strength obtained from unconfined compression tests should be used. Skempthon's equation for bearing capacity is used in which cohesion is taken as half the unconfined compression strength.

(iii) Triaxial tests: For large jobs, the shear strength may be determined from triaxial tests on undisturbed samples. The shear parameters are obtained by plotting the data from triaxial tests. Drainage conditions in the field are to be simulated in the laboratory and careful interpretation of the results is required

Silts

Silt is often a poor foundation soil and should be avoided for supporting footings. Apparent cohesion, exhibited by moist silt disappears on immersion. Plate load tests at about the ground water level are advocated in this case.

Compacted fills

Bearing capacity for compacted fills must be determined both before and after compaction.

Organic soils

Organic soils are not suitable for supporting footings. Highly organic soils settle unduly even under their own weight, both by consolidation and by decay or decomposition of the organic matter.

Rocks

Generally speaking, rocks can withstand pressures greater than concrete can do. Rocks with fissures. Folds, faults and bedding planes are exceptions to this. Shales may become clay or silt on soaking. Weathered rocks are treacherous and lose strength on wetting.

Settlement of Footings

If the allowable bearing pressure is determined based on the smaller value from the two criteria – shear strength and permissible settlement – footings on granular soils do not suffer detrimental settlement.

Footings on clay will experience settlement which consists of three components (Skempton and Bjerrum, 1957):

$S = S_i + S_c + S_s$

Where, S = Total settlement,

S_i = Immediate elastic settlement,

S_c = Consolidation settlement due to primary compression, and

S_s = Settlement due to secondary compression of the clay.

These and other details of settlement analysis have already been dealt with exhaustively (Chapter 11).

Proportioning Sizes of Footings and Choice of Column Loads

A structure is usually supported on a number or columns. These columns usually carry different loads depending upon their location with respect to the structure. Differential settlements are minimized b proportioning the footings for the various columns so as to equalize the average bearing pressure under all columns.

But each column load consists of dead load plus live load. The full live load does not act all the time; further live loads such as those due to heavy wind do not produce significant settlement since they act only for short durations; this is especially true in the case of cohesive soils. Hence, dead load plus full live load is not a realistic criterion for producing equal settlement.

What is known as the 'service load' is a better criterion. This is the actual load expected to act on the foundation during the normal service of the structure, i.e., for most of the time. In ordinary buildings, this is taken as the dead load plus one half the live load; a larger fraction of the live load should be used for warehouses and other industrial structures.

The following procedure is given by Teng (1976) based on the recommendations of Peck, Hanson and Thorn burn (1974):

(i) Dead load, inclusive of self-weight of column and estimated value for footing, is noted for each column footing.

(ii) The live load for each column is calculated (appropriate values are chosen from the relevant I.S. Codes of Practice).

(iii) The ratio of live load to dead load is calculated for each column footing; the maximum value of this ratio is noted.

(iv) The allowable bearing pressure of ht soil is determined by the procedures given in Chapter 14.

(v) For the footing with the largest live load to dead load ratio, the area of footing required is calculated by dividing the total load (dead load plus maximum live load) by the allowable bearing pressure of the soil.

(vi) The service load for the column with the maximum live load to dead load ratio is computed by adding the appropriate fraction of the live load to the dead load.

(vii) The allowable bearing pressure to be used for all the other column footings is obtained by dividing the service load for the column with maximum live load to dead load ratio by the area of the footing for this column. (This pressure will be obviously somewhat less than the computed allowable bearing pressure of step (iv).

(viii) Service loads for all other columns are computed.

(ix) The area of the footing for each of the other columns is obtained by dividing the corresponding service load by the reduced allowable bearing pressure of step (vii).

Corresponding service load by the reduced allowable bearing pressure of the soil is never exceeded under any circumstances and the reduced or service loads, which are effective during most of the time, are expected to result in equal settlements.

The procedure, as standardized by the ISI, is set out in "IS : 1080-1985 Code of Practice for Design and Construction of simple spread foundations (Second Revision)".

Footings Subjected to Moments – Eccentric loading

Footings supporting axially loaded columns and which are symmetrically placed with respect to the columns will be subjected to uniform soils pressures. However, footings may often have to resist not only axial loads but also moment about one or both axes. The moment may exist at the bottom of an axially loaded column, whence it is transmitted to the footing;

alternatively, it may be produced by an axial vertical load located eccentrically from the centroid to the base of the footing, positioned un symmetrically with respect to the column. If the moment in the first case is equal to the product o the axial load and eccentricity in the second, the soil pressure distribution will be just identical. Thus, the substitution of an equivalent eccentric load for a real moment is considered a convenient method which simplifies computations in some cases.

Foundations for retaining walls may have to resist moments due to the active earth pressure and those for bridge piers may have to resist moments produced primarily by wind and traction on the superstructure. These foundations also have to be treated in a somewhat similar manner as footings subjected to moments.

Once the soil reactions are determined, the design data such as critical moments and shear may be obtained as a prerequisite for the structural design. Fundamental to all these computations are the laws of statics. The distribution of vertical soil pressure at the base must satisfy the requirements of statics that (i) the total upward soil reaction must be equal to the sum of the downward loads on the base, and (ii) the moment of the resultant vertical load about any point must equal to the moment of the total soil reaction about the same point. In addition, an adequate horizontal soil reaction must be available, by virtue of frictional resistance at the base, to oppose the resultant horizontal load.

Ordinary footings are commonly assumed to act as rigid structures. This assumption leads to the conclusion that the vertical settlement of the soil beneath the base must have a planar distribution since a rigid foundation remains plane when it settles. Another assumption is that the ratio of pressure to settlement is constant, which also leads to the conclusion regarding the planar distribution of soil pressure. Although, neither of these assumptions is strictly valid, each is considered to be sufficiently accurate for ordinary purposes of design.

Two distinct cases arise:

(1) Resultant force within the middle third of the base; and,

(2) Resultant force outside the middle third of the base.

The forces acting on the including self-weight are resolved into V and H. The moment M may be expressed as $M = H \cdot h = V \cdot e$.

We have, e = M/V

This equation enables one to determine the eccentricity of the resultant of all forces acting on the base regardless of how complicated the conditions of loading may be.

If the vertical load V acts alone, it produces uniform soil pressure due to the direct stress as shown in Fig. 15.14 (c). if the horizontal load H acts alone, it produces a shear that must be resisted by the soil at the base and also a moment which produces soil pressure distribution shown in Fig. 15.14 (d), due to bending.

The resultant soil pressure will be the combined effect of V and M is shown in Fig. 5.14 (e).

The maximum and minimum soil pressure is obtained as:

Equation 15.3 is merely a special form of the basic formula for the resultant stress on a section subjected to a direct load p and a moment M, expressed in strength of materials, in the form:

The maximum eccentricity for no tension to occur in the base is obtained by equating q_{min} to zero, and solving for e:

Since the eccentricity can occur to either side of the middle depending upon the direction of H, the resultant force should fall within the middle – third of the base in order that no tensile stresses occur anywhere in the base.

If the eccentricity occurs with respect to the axis which bisects the other dimension L of the footing.

This leads to the concept of 'kern' or 'core' of a section, which is the zone within which the resultant should fall for the entire base to be subjected to compression.

For a rectangular section, the kern is a centrally located rhombus with the diagonals equal to one-third of the breadth and length; for a circular section it is a concentric circle with diameter one-fourth of that of the circle.

Most footings are designed so that the resultant of the loads falls within the kern and the soil reaction everywhere is compressive. However, in certain cases such as the design of the base slab of a cantilever retaining wall, the resultant may fall outside the kern, and the distribution of pressure shown in Fig. 15.15. Must be used for the structural design of the footing.

Resultant force outside the middle-third of the base

If the horizontal component of the total load increases beyond a certain limit in relation to the vertical component, the resultant force falls outside the middle-third of the base, the eccentricity being more than the limiting value of one – sixth the size of the base. It must be remembered that soil cannot provide tensile reaction; it just loses contact with the footing in the zone of tension. This situation is shown in Fig. 15.16.

From the laws of statics, the total upward force must be equal to V and also collinear with V. That is to say:

Where A = bL.

Equation 15.9 reveals that the maximum soil pressure is merely twice the average pressure produced by V acting on the area XL.

Moment about both axes

When moments act simultaneously about both axes, for example when a vertical load acts at an eccentricity with respect to both the axes, as shown in fig. 15.17, the soil pressure is given by the following equation:

This is under the assumption that the entire base is under compression.

The location of the maximum and minimum soil pressures may be determined readily by observing the direction on the moments. Likewise, the proper signs in Eq. 15.12 may be determined by inspection for any other point on the base of the footing.

If the minimum soil pressure computed appears to be negative, there exists a zone like CZZ in which the footing loses contact with the soil and hence, there will be no pressure in the zone. Equation 15.12 will not be applicable to this case. For the determinations of soil pressures for this situation, the reader is referred to Peck, Hanson and Thorn burn (1974), who give an excellent trial and error procedure.

Useful width concept

For the determination of the bearing capacity of an eccentrically loaded footing, the concept of 'useful width' has been introduced. By this concept, the portion of the footing which is symmetrical about the load is considered useful and the other portion is simply assumed superfluous for the convenience of computation (Teng, 1976). This is illustrated in Fig. 15.18.

If the eccentricities are e_b and e_L, as shown, the useful widths b′ and L′ are:

$b' = b - 2e_b \qquad L' = L - 2e_L$

The equivalent area A′ is considered to be subjected to a central load for the determination of bearing capacity:

$A' = b'L' = (b-2e_b)(L-2e_L)$

The procedure may be used even if the eccentricity is with respect to one of the axes only.

This concept simply means that the bearing capacity of a footing decreases linearly with the eccentricity of load. This is almost true in the case of cohesive soils: however, the relationship is parabolic rather than linear in the case of granular soils (Meyerhof, 1953).

Therefore, it is considered better to use a reduction factor R_e for getting the reduced

$q'_{ult} = q_{ult} . R_e$

where q'_{ult} = bearing capacity of an eccentrically load footing size b x L,

q_{ult} = bearing capacity of a centrally loaded footing of size b x L, and

R_e = reduction factor for eccentricity.

If there is eccentricity about both axes, the product of the two factors must be used.

Footings with unsymmetrical shapes

The assumption till now has been that at least one axis of symmetry exists for the footing in plan. If an unsymmetrical section is involved under eccentric loading, computation of soil pressures becomes a problem, since Eq. 15.12 is not applicable even though the entire base may be in compression. However, the errors involved in using Eq. 15.12 may not be intolerable for design, unless the footing is greatly unsymmetrical.

Inclined Loading

The conventional procedure of analyzing the stability of footings subjected to inclined loading consists in resolving the load into a vertical component V and a horizontal component H, and dealing with the effect of each separately. The soil pressure due to the vertical load is considered to be uniform and the stability against ultimate failure is analyzed in the usual way.

The stability against the horizontal load is analyzed by ensuring a minimum factor of safety against sliding at the base, which is defined as the ratio between the total resistance to sliding and the applied horizontal force. The total horizontal resistance usually consists of passive resistance of the soil and a frictional resistance F at the base, which is dependent upon the coefficient of friction between the base of the footing and the soil beneath it. This is illustrated in Fig. 15.20.

Janbu (1957) proposed an analysis which is a direct extension of the Terzaghi theory with an additional factor N_h, in addition to Terzaghi's factors N_c, N_γ and N_q :

Meyerhof (1953) proposed an analysis of footings subjected to inclined loads and constructed convenient charts, shown in fig. 15.22 the load is considered to act vertically and the bearing capacity is obtained by the normal procedure. It is then corrected by multiplying by the factor R_i.

Footings on Slopes

Meyerhof (1957) again proposed an equation for the bearing capacity of footings on sloping ground as follows:

The values of the bearing capacity factors N_{cq} and $N_{\gamma q}$ for continuous footings are given in fig. 15.23. These factors vary with the slope of the ground, the relative position of the footing and the angle of internal friction of the soil.

Footings must be constructed only on slopes which are stable. The stability of the slope itself may be endangered by the construction of footings.

Construction of Spread Footings

Footings are relatively simple to construct. The inspection of subsoil conditions, the relative depth of footing and dewatering of excavation when necessary require special attention. Depending on the nature of soil, bracings may be required for this sides.

The average soil condition based on the soil boring results must be ascertained. As the foundation is constructed, the actual soil conditions encountered must be checked with respect to the boring analysis.

Adjacent footings should be constructed such that their difference of levels, if any, does not introduce undue additional stress at the lower footings and also that the lower footing does not affect the stability of the upper one. This difficulty is generally avoided by keeping the difference in the elevations of footings not greater than one-half the clear distance between the footings. It is always a good practice to construct the lower footings first, so that the elevation of the upper footing may be adjusted if necessary.

The excavation should be kept dry during the construction period because free water gives rise to many difficulties. The soil conditions under water cannot be easily inspected. In clay soils, free water tends to soften the upper portion of the soil and cause settlements. Placing concrete under water also poses problems. For these reasons, it is considered necessary to dewater the excavations where necessary.

For certain recommendation in this regard, the reader is referred to "IS" 1080 – 1980 Code of Practice for design and construction of simple spread foundations (First Revision)"

STRAP FOOTINGS

A relatively common type of combined construction is the 'strap footing' or 'cantilever footing' as has already been seen in Sec. 15.2. This is usually employed when the footing of an exterior column cannot be allowed to extend into adjoining private property. Straps may be arranged in a variety of ways (Fig. 15.2), and the choice depends upon the specific conditions of each case.

The Cantilever Principle

The cantilever principle is largely concealed in actual footings of this type. This principle is illustrated in Fig. 15.24.

It may be inferred from this figure that the two individual footings is a problem of statics if the allowable soil pressure is known and if the dimension b of the exterior footing is either fixed or assumed. Also, the centroid of the two areas must lie on the line of the action of the resultant load. This requirement may not be obvious because the two areas are usually found rather independently from ractions determined from the principles of statics.

Basis for Design of Strap Footings

Strap footings are designed based on the following assumptions:

(i) The strap footing is considered to be infinitely stiff. It serves to transfer the column loads onto the soil with equal and uniform soil pressure under both the footings.

(ii) The strap is a pure flexure member and does not directly take soil reaction. The soil below the strap will be loosened up in order that the strap does not rest on the soil and exert pressure.

With these assumptions, the procedure of design is simple. With reference to Fig. 15.25, it may be given as follows:

Assume a trail value of e and compute the reactions R_i and Re from statics. The tentative areas of the footing are equal to the reactions R_i and R_e divided by the allowable bearing pressure q. the value of e is computed with tentative sizes. These steps are repeated until the trail value of e is identical with the final value.

The shearing force and bending moment in the strap are determined, the strap being designed to withstand the maximum values of these.

Each of these footings is assumed to be subjected to uniform soil pressure and designed as simple spread footings. Under the assumptions given above, the resultant of the column loads P_e and P_i would coincide with the centre of gravity of the areas of the two footings.

Rectangular Combined Footing

A combined footing is usually given a rectangular shape if the rectangle can extend beyond each column the necessary distance to make the centroid of the rectangle coincide with the point at which the resultant of the column loads intersects the base.

If the footing is to support an exterior column at the property line where the projection has to be limited, provided the interior column carries the greater load, the length of the combined footing is established by adjusting the projection of the footing beyond the interior column. The width is then obtained by dividing the sum of the vertical loads by the product of the length and the allowable soil pressure. A rectangular combined footing is shown in Fig. 15.26.

The B.M. and S.F. diagrams may be sketched, assuming that the column loads are concentrated loads. The maximum values are used for design.

Trapezoidal Combined Footing

When the two column loads are unequal, the exterior column carrying higher load and when the property line is quite close to the exterior column, a trapezoidal combined footing is used. It may be used even when the interior column carries higher load; but the width of trapezoid will be higher in the inner side. The location of the resultant of the column loads establishes the position of the centroid of the trapezoid. The length is usually limited by the property line at one end and adjacent construction, if any, at the other.

The width at either end of the trapezoid can be determined from the solution of two simultaneous equations – one expressing the location of the centroid of the trapezoid and the other equating the sum of the column loads to the product of the allowable soil pressure and the area of the footing.

The resulting pressure distributions are linear or uniformly varying (and not uniform) as shown in Fig. 15.27).

RAFT FOUNDATIONS

A ‘raft’ or a ‘mat’ foundation is a combined footing which covers the entire area beneath of a structure and supports all the walls and columns. This type of foundation is most appropriate and suitable when the allowable soil pressure is low, or the loading heavy, and spread footings would cover more than one half the plan area. Also, when the soil contains lenses of compressible strata which are likely to cause considerable differential settlement, a raft foundation is well-suited, since it would tend to bridge over the erratic spots, by virtue of its rigidity. On occasions, the principle of floating foundations may be applied best in the case of raft foundations, in order to minimize settlements.

Common Types of Raft Foundations

Common types of raft foundations in use are illustrated in Fig. 15.28.

Fig. 15.28. (a) represents a true raft which is a flat concrete slab of uniform thickness throughout the entire area; this is suitable for closely spaced columns, carrying small loads. (b) represents a raft with a portion of the slab under the thickened column; this provides sufficient strength for relatively large column loads. (c) is a raft with thickened bands provided along column lines in both directions; this provides sufficient strength, when the column spacing is large and column loads unequal. (d) Represents a raft in which pedestals are provided under each column; this alternative serves the same purpose as (b). (e) Represents a two-way grid structure made of cellular construction and of intersecting structural steel construction (Tang, 1949). (f) Represents a raft where in basement walls have been used as ribs or deep beams.

A raft foundation usually rests directly on soil or rock: however, it may rest on piles as well, if hard stratum is not available at a reasonably small depth.

Bearing Capacity of Rafts on Sands

Since the bearing capacity of sand increases with the size of the foundation and since rafts are usually of large dimensions, a bearing capacity failure of raft on sand is practically ruled out. As a raft bridges over loose pockets and eliminates their influence, the differential settlements are much smaller than those of a footing under the same pressure. Hence, higher allowable soi pressures may be used for design of rafts on sands.

Terzaghi and Peck (1948), as also Peck, Hanson and Thorn born (1974), recommend an increase of 100% over the value allowed for spread footings. The design charts developed for the bearing capacity from N- values for footings on sands may be used for this purpose. The effect of the location of water table is treated as in the case of footings.

Bearing Capacity of Rafts on Clays

The net ultimate bearing capacity is divided by the factor of safety to obtain the net allowable soil pressure for a footing. The same principle is applicable to rafts on clay. Accordingly, the factor of safety, η, in terms of net soil pressure, is given by

Where, c = unit cohesion,

N_c = bearing capacity factor for cohesion,

q = gross soil pressure or contact pressure,

γ = unit weight of soil,

and D_f = depth of raft below ground surface.

It is obvious that the factor of safety is very large for rafts established at such depths that γD_f is nearly equal to q. In fact, the theoretical value of η in infinite, when γD_f equals q; in such a case, the raft is said to be a 'fully compensated foundation' (Peck, Hanson and Thornborn, 1974).

Coefficient of Sub grade Reaction

The 'coefficient of sub grade reaction' or 'sub grade modulus' is defined as the ratio between the pressure and the settlement at a given point :

Where, = coefficient of sub grade reaction in N/mm3,

q = pressure against the footing or raft at a given point in N/mm2,

and S′ = settlement at the particular point in mm.

In other words, the coefficient of sub grade reaction is the pressure required to produce unit settlement. This is difficult to determine for clayey soils in view of the long time required for the consolidation settlement to occur. Equation 15.24 is based on the following assumptions :

(i) is independent of pressure

(ii) is the same at every point of footing or mat.

Actually, a number of factors affect the value of coefficient of sub grade reaction (Terzaghi, 1955) :

Effect of size

The value of decreases with increasing width of footing.

Where = coefficient of sub grade reaction for a very long footing of width b m, and

= coefficient of sub grade reaction for a very long footing of width 1 m.

Equation 15.25 is established from experiments and Eq. 15.26 from the pressure bulb concept.

General Considerations in the Design of Rafts

The conditions under which a raft foundation is suitable have already been discussed. In its simplest form a raft consists of a reinforced-concrete slab that supports the columns and walls of a structure and that distributes the load there form to the underlying soil. Such a slab is usually designed as continuous flat-slab floor supported without upward deflection at the columns and walls. The soil pressure acting against the slab is commonly assumed to be uniformly distributed and equal to the total of all column loads, divided by the area of the raft. The moments and shears in the slab are determined by the use of appropriate coefficients listed in codes for the design of flat-slab floors.

On account of erratic variations in compressibility of almost every soil deposit, there are likely to be correspondingly erratic deviations of the soil pressure from the average value. since the moments and shears are determined on the basis of the average pressure, it is considered good practice to provide the slab more reinforcement than the theoretical requirement and to use the same percentage of steel at top and bottom (Peck, Hanson and Thorn born, 1974).

The flat slab analogy is valid only if the deferential settlement between columns is small and furthermore, if the pattern of the differential settlement is erratic rather than systematic. Also, even if deep-seated or systematic settlements are negligible, the flat-slab analogy is likely to lead to uneconomical design unless the columns are more or less equally spaced and equally loaded. Otherwise, differential settlements may lead to substantial redistribution of moments in the slab.

Under such circumstances, rafts are sometimes designed on the basis of the concept of the modulus of sub grade reaction, which implies that soil is considered to be analogous to a bed of closely and equally spaced elastic springs of equal stiffness in its stress-strain behavior. Evaluation of the modulus of sub grade reaction, , for design is not a simple problem since is known to vary in a complex manner on the shape and size of the loaded area, as well as on the magnitude and position of near-by loaded areas. [For IS procedure, refer "IS:2950(Part-I-1974 Code of Practice for Design and Construction of Raft Foundation – Part-I Design"].

If a raft covers a fairly large area and significantly increases the stresses in an underlying deposit of compressible clay, it is likely to experience large systematic differential settlements. For these to be avoided, strength of the slab alone is not sufficient, but stiffness is also required. However, a stiff a raft is likely to be subjected to bending moments far in excess of those corresponding to the flat-slab or sub grade modulus analyses (Peck, Hanson and Thorn burn, 1974). These moments may require deep beams or trusses. Thus, the raft in such instances may be considered to consists of two almost independent elements : the base slab, which may still be designed by the flat-slab analogy; and the stiffening members, which have the function of preventing must of the differential settlement of the points of support for the base slab.

It has been known that contact pressure distributions in sand and clay are different from the uniform distribution commonly assumed in conventional raft design. In the case of sand, maximum pressure occurs at the middle and minimum, if any, occurs at the edges; in the case of clay, minimum pressure occurs in the middle and maximum (in fact, sometimes, very high) pressure occurs at the edges. It is also interesting to note that the pressure under a raft on clay may vary with time (Teng, 1949), and the worst conditions expected are to be considered for design.

It is unlikely that the edge pressure will exceed twice the average pressures.

As an alternative to the relatively high cost of a stiff raft of large-size above a compressible deposit, substantial economy can be realized by designing a flexible raft and superstructure that can deform without damage into the shape corresponding to the compression of the subsoil. It may often prove preferable to accept the deformations if the cost of a stiff foundation can be avoided. The design of a flexible raft foundation cannot be readily base on the calculation of stresses in the slab. Instead, it is necessary to estimate the maximum curvature to which the raft may be flexed, and to select the thickness of the slab and the quantum of reinforcement such that the slab will not develop cracks large enough to permit a serious leakage of ground water. as an approximate guideline, 1% of steel may be provided in each of two directions at right-angle to each other, equally divided between the top and bottom of the slab. The thickness of the slab should not be generally greater than 1% of the radius of curvature, though local increases of thickness near columns and walls may be required to prevent shear failures.

Construction of Raft Foundations

Raft foundations are invariably constructed of reinforced concrete. They are poured in small areas such as 10m x 10m to avoid excessive shrinkage cracks. Construction joints are carefully located at places of low shear stress – such as the centre lines between columns. Reinforcements should be continuous across points. If a bar is spliced, adequate lap is provided. Shear keys may be provided along joints so that the shear stress across the joint is safely transmitted. If necessary, the raft may be thickened to provided sufficient strength at the joints.

FOUNDATIONS ON NON – UNIFORM SOILS

It is generally assumed that the subsoil is relatively uniform either to a very great depth or else to a limited depth where a firm base is encountered. In reality, such situations are so uncommon as to be considered rare exceptions. The procedures of foundation design are not often directly applicable to practical problems; but these may be modified to give reliable indications of the probable behavior of foundations on non-uniform deposits.

Most sub soils consist either of definite strata or more or less lenticular elements. On the basis of preliminary information, such as that from exploratory borings together with standard penetration tests and simple laboratory tests, it is possible to identify deposits which are sufficiently strong and in compressible. This would enable one to concentrate on the weaker or more compressible strata, so as to ascertain their influence on the behavior of the proposed foundation. The load-carrying capacity of the doubtful materials is ascertained and based on failure or permissible settlement. Usually this information is adequate for a selection of the proper type of foundation. Sometimes, more elaborate exploratory procedures and soil tests may be required to provide the basis for a sound decision.

Stresses may be computed using Newmark's chart or by some simplified procedure. Although the chart is based on the assumption that the material is homogeneous, the errors due to stratification or other irregularities are not likely to be significant enough to invalidate the predictions of the probable behavior of the soil.

In the following sub sections, the more important kinds of non-uniform soil deposits will be discussed.

Pile Foundations

Introduction

When the soil at or near the ground surface is not capable of supporting a structure, deep foundations are required to transfer the loads to deeper strata. Deep foundations are, therefore, used when surface soil is unsuitable for shallow foundation, and a firm stratum is so deep that it cannot e reached economically by shallow foundations. The most common types of deep foundations are piles, piers and caissons. The mechanism of transfer of the load to the soil is essentially the same in all types of deep foundations.

A deep foundation is generally much more expensive than a shallow foundation. It should be adopted only when a shallow foundation is not feasible. In certain situations, a fully compensated floating raft may be more economical than a deep foundation. In some cases, the soil is improved by various methods to make it suitable for a shallow foundation.

A pile is a slender structural member made of steel, concrete or wood. A pile is either driven into the soil or formed in-situ by excavating a hole and filling it with concrete. A pier is a vertical column of relatively larger cross-section than a pile. 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. The distinction between a cast in-situ pile and a pier is rather arbitrary. A cast in-situ pile greater than 0.6m diameter is generally termed as a pier. A caisson is a hollow, watertight box or chamber, which is sunk through the ground for laying foundation under water. The caisson subsequently becomes an integral part of the foundation. A pier and a caisson differ basically only in the method of construction.

Pile foundations are discussed in this chapter. Piers and caissons are dealt with in chapter 26. Well foundations, which are special type of caissons, are discussed in Chapter 27.

NECESSITY OF PILE FOUNDATIONS

Pile foundations are used in the following conditions:

(1) When the strata at or just below the ground surface 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. It would cause 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 an tall structures that are subjected to horizontal forces due to wind and earthquake.

(5) Piles are required when the soil conditions are such that a wash out, erosion or scour of soil may occur from underneath a shallow foundation.

(6) Piles are used for the foundations of some structures, such as transmission towers, off-shore plate forms, which are subjected to uplift.

(7) In case of expansive soils, such as black cotton soil, which swell or shrink as the water content changes, piles are used to transfer the load below the active zone.

(8) Collapsible soils, such as loess, have a breakdown of structure accompanied by a sudden decrease in void ratio when there is an increase in water content. Piles are used to transfer the load beyond the zone of possible moisture changes in such soils.

CLASSIFICATION OF PILES

Piles can be classified according to (1) the material used (2) the mode of transfer of load, (3) the method of construction, (4) the use, or (5) the displacement of soil, as described below.

(1) Classification according to material used

There are four types of piles according to materials used.

(i) Steel Piles. Steel Piles are generally either in the form of thick pipes or rolled steel H – sections. Pipe steel Piles are driven into the ground with their ends open or closed. Piles are provided with a driving point or shoe at the lower end.

Epoxy coatings are applied in the factory during manufacture of pipes to reduce corrosion of the steel piles. Sometimes, concrete encasement at site is done as a protection against corrosion. To take into account the corrosion, an additional thickness of the steel section is usually recommended.

(ii) Concrete Piles. Cement concrete is used in the construction of concrete piles. Concrete piles are either precast or cast-in situ. Precast concrete piles are prepared in a factory or a casting yard. The reinforcement is provided to resist handling and driving stresses. Precast piles can also be pre-stressed using high strength steel pre-tensioned cables.

A cast-in situ pile is constructed by making a hole in the ground and the filling it with concrete. A cast- in situ pile may be cased or uncased. A cased pile is constructed by driving a steel casing into the ground and filling it with concrete. An uncased pile is constructed by driving the casing to the desired depth and gradually withdrawing casing when fresh concrete is filled. And uncased pile may have a pedestal.

(iii) Timber Piles. Timber piles are made from tree trunks after proper trimming. The timber used should be straight, sound and free from defects.

Steel shoes are provided to prevent damage during driving. To avoid damage to the top of the pile, a metal band or a cap is provided. Splicing of timber piles is done using a pipe sleeve or metal straps and bolts. The length of the pipe sleeve should be at least five times the diameter of the pile.

Timber piles below the water table have generally long life. However, above the water table these are attacked by insects. The life of the timber piles can be increased by preservatives such as creosote oils. Timber piles should not be used in marine environment where these are attacked by various organisms.

(iv) Composite piles. A composite pile is made of two materials. A composite pile may consist of the lower portion of steel and the upper portion of cast-in situ concrete. A composite pile may also have the lower portion of timber below the permanent water table and the upper portion of concrete. As it is difficult to provide a proper joint between two dissimilar materials, composite piles are rarely used in practice.

(2) Classification Based on Mode of Transfer of Loads

Based on the mode of transfer of loads, the piles can classified into 3 categories :

(i) End – bearing piles. End - bearing piles transmit the loads through their bottom tips. Such piles act as columns and transmit the load through a weak material to a firm stratum below. If bed rock is located within a reasonable depth, piles can extend to the rock. The ultimate capacity of the pile depends upon the bearing capacity of the rock. If instead of bed rock a fairly 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 end (Q_p).

(ii) Friction piles. Friction piles do not reach the hard stratum. These piles transfer the load through skin friction between the embedded surface of the pile and the surrounding soil. Friction piles are used when a hard stratum does not exist at a reasonable depth. The ultimate load (Q_u) carried by the pile is equal to the load transferred by skin friction (Q_s).

[**Note :** The term friction pile is actually a misnomer, as in the clayey soils, the load is transferred by adhesion and not friction between the pile surface and the soil].

The friction piles are also known as floating piles, as these do not reach the hard stratum.

(iii) Combined end bearing and friction piles. These piles transfer loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft. The ultimate load carried by the pile is equal to the sum of the load carried by the pile point (Q_p), and the load carried by the skin friction (Q_s).

(3) Classification based on method of installation

Based on the method of construction, the piles may be classified into the following 5 categories :

(i) Driven piles. These piles are driven into the soil by applying blows of a heavy hammer on their tops.

(ii) Driven and Cast-in-situ piles. These piles are formed by driving a casing with a closed bottom end into the soil. The casing is later filled with concrete. The casing may or may not be withdrawn.

(iii) Bored and Cast-in-situ piles. these piles are formed by excavating a hole into the ground and then filling it with concrete.

(iv) Screw piles. These piles are screwed into the soil.

(v) Jacked piles. These piles are jacked into the soil by applying a downward force with the help of a hydraulic jack.

(4) Classification based on use

The piles can be classified into the following 6 categories, depending upon their use.

(i) Load bearing piles. These piles are used to transfer the load of the structure to a suitable stratum by end bearing, by friction or by both. These are the piles mainly discussed in this chapter.

(ii) Compaction piles. These piles are driven into loose granular soils to increase the relative density. The bearing capacity of the soil is increased due to densification caused by vibrations.

(iii) Tension piles. These piles are in tension. These piles are used to anchor down structures subjected to hydrostatic uplift forces or overturning forces.

(iv) Sheet piles. Sheet piles form a continuous wall or bulkhead which is used for retaining earth or water.

(v) Fender piles. Fender piles are sheet piles which are used to protect water-front structures from impact of ships and vessels.

(vi) Anchor piles. These piles are used to provide anchorage for anchored sheet piles. These piles provide resistance against horizontal pull for a sheet pile wall.

(5) Classification based on displacement of soil

Based on the volume of the soil displaced during installation, the piles can be classified into 2 categories:

(i) Displacement piles. All driven piles are displacement piles as the soil is displaced laterally when the pile is installed. The soil gets densified. The 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.

(ii) Non-displacement piles. Bored piles are non-displacement piles. As the soil is removed when the hole is bored, there is no displacement of the soil during installation. The installation of these piles causes very little change in the stresses in the surrounding soil.

PILE DRIVING

Piles are driven into the ground by means of hammers or by using a vibratory driver. Such piles are called driven piles. in some special cases, piles are installed by jetting or partial auguring.

The following methods are commonly used.

(1) Hammer Driving. Fig. 25.1 shows a pile driving rig. It consists of a hoist mechanism, a guiding frame and a hammer device. The hammers used for pile driving are of the following types:

(i) Drop hammer. A drop hammer is raised by a winch and allowed to drop on the top of pile under gravity from a certain height. During the driving operation, a cap is fixed to the top of the pile and a cushion, is generally provided between the pile and the cap. Another cushion, known as hammer cushion, is placed on the pile cap on which the hammer causes the impact. The drop hammer is the oldest type of hammer used for pile driving. It is rarely used these days because of very slow rate of hammer blows.

(ii) Single-acting hammer. In a single-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. In a double-acting hammer, air (or steam) pressure is used to raise the hammer. When the hammer has been raised to the required height, air, (or steam) pressure is applied to the other side of the piston and the hammer is pushed downward under pressure. This increases the impact energy of the hammer.

(iv) Diesel hammer. A diesel 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 the fuel is injected near the anvil. As soon as 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 the process is repeated.

The ram lifts automatically. It has to be manually raised only once at the beginning.

Diesel hammers are not suitable for driving piles in soft soils. In such soils, the downward movement of the pile is excessive and the upward movement of the ram after impact is small. The height achieved after the upward movement of the hammer may not be sufficient to ignite the air-fuel mixture.

Diesel hammers are self-contained and self-active.

(2) Vibratory Pile Driver. A vibratory pile driver consists of two weights, called exciters, which rotate in opposite directions. The horizontal components of the centrifugal force generated by exciters cancel each other but the vertical components add. Thus a sinusoidal dynamic vertical force is applied to the pile, which forces the pile downward. The frequency of vibration is kept equal to the natural frequency of pile-soil system for better results.

A vibratory pile driver is useful only for sandy and gravelly soils. The speed of penetration is good. The method is used where vibrations and noise of conventional driving methods cannot be permitted.

(3) Jetting Techniques. When the pile is to penetrate a thin hard layer of sand or gravel overlying a softer soil layer, the pile can be driven through the hard layer by jetting techniques. Water under pressure is discharged at the pile bottom point by means of a pipe to wash and loosen the hard layer.

(4) Partial Auguring Method. Batter piles (inclined piles) are usually advanced by partial auguring. In this method, a power auger 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.

Construction of bored piles

(a) Drilling of holes.

Bored piles are constructed after making a hole in the ground and filling it with concrete.

The following methods are used for drilling of the hole.

(1) Hand auger. A hand auger can be used for boring without casing in soils which are self-supporting, such as firm to stiff clays and silts and clayey sands and gravels above the water table. The depth of the hole is generally limited to about 4.5 m. the diameter of the hole is usually not more than 350mm.

(2) Mechanical auger. For piles of diameter more than 350mm or depth greater than 4.5m, a hand auger becomes uneconomical. In such a case, a mechanical auger is used. A mechanical auger can be of rotary type or bucket type. It is power driven. The soil in this case must be self-supporting, with or without bentonite slurry. The soil should be free from tree roots, cobbles and boulders.

A continuous flight auger is also used to drill the bore hole.

(3) Boring rig. A boring rig is used to sink the hole in ground where hand or mechanical auguring is not possible, such as water-bearing sand or gravels, very soft clays and slits and the soils having cobbles and boulders.

A specially designed boring rig, known as grab-type bored pilling rig, is sometimes used. In this type of rig, the casing is given a continuous semi-rotary motion which causes its sinking as the bore hole is advanced by percussion drilling.

(4) Belling Bucket. Under reamed piles are large diameter bored piles with enlarged bases. Excavation for the under reamed piles is done by a special type belling bucket.

(b) Concreting

Before concrete is placed, the bored hole is bailed dry of water. Any loose or softened soil is cleaned out and the bottom of the hole is rammed. A layer of dry concrete is placed and rammed if the bottom of the hole is wet. Then the concrete with a readily workable mix (7.5 to 10 cm slump), not leaner than 300 kg cement/m^3 of concrete, is poured into a hopper placed at the mouth of the hole.

If the hole cannot be bailed or pumped dry before placing the concrete, the hole is lined with a casing throughout its depth. A mass of concrete is then deposited at the base of the hole by a Tromie pipe. As soon as the concrete has hardened and formed a plug, the hole is pumped free of water. The casing is then gently turned and lifted slightly to break the joint with the plug. The hole is pumped dry. The remainder of the concreting is done by placing it dry up to the ground surface. The casing is then lifted entirely from the bore hole.

If the ground water is under a high pressure, there will be inflow of water between the concrete plug and the inside of the casing. The inflow should be stemmed by caulking. The casing is cut by oxy-acetylene just above the plug. The shaft is then concrete and the casing raised. The cut portion of the casing around the plug is left permanently in place.

Instead of plugging the base of the pile and concreting, an alternative method is to concrete the entire shaft under using a tremie pipe. Concrete should be easily workable (slump 12.5 to 17.5 cm) and the cement content should be at least 400kg/m^3. A retarder is added to the concrete if there is a risk of the concrete setting before the casing is lifted out. However, the quality of concreting done under water is not good. This method should be avoided as far as possible.

DRIVEN CAST-IN-SITU CONCRETE PILES

A driven cast-in-situ concrete pile is formed in the ground by driving a casing with a plug or shoe at its bottom. If the casing is removed after concrete has been placed, it is known as uncased or shell – less pile. On the other hand, if the casing is left in the ground after concreting, it is called a cased pile. In uncased piles, the concrete comes in direct contact with the soil. The concrete may be rammed or vibrated after its deposition. A pedestal may be formed at the lower end of the shell-less pile if required.

Cast-in-situ driven concrete piles can be broadly classified into three types: (i) cased pile, (ii) uncased pile, and (iii) pedestal type. Different types of piles with patent rights are available. The main difference between different patents is in the method of construction, as described below.

(i) The Franki pile is a type of driven and cast-in-situ displacement pile. A heavy steel pile is first pitched in a shallow foundation. A plug of lean concrete is then placed in the bottom of the pipe and compacted with a heavy steel rammer. The plug is then rammed and with it the pipe also goes down. This driving operation is continued until the bearing stratum is reached. The concrete is hammered to from a pedestal. A reinforcement cage is then placed in the pipe and the pile shaft is concreted. The pipe is withdrawn as the concrete is rammed.

(2) In uncased – Western pile, a heavy steel drive pipe of 35 cm diameter with a steel core is driven. The concrete is deposited in the pipe after removing the core. The concrete is rammed as the pipe is withdrawn. The pedestal is formed after the drive pipe has been lifted to some height.

(3) In cased-Western pile, the hole is made using a heavy steel drive pipe as for the uncased- Western type. A shell of 30 cm diameter is lowered inside the drive pipe. After the shell has been filled with concrete, the drive pipe is withdrawn. A pedestal can be formed by placing some concrete before lowering the shell and ramming.

(4) A western button-bottom pile is formed by driving a steel pipe with a 43 cm diameter precast concrete point at its bottom. After reaching the required depth, a shell is lowered into the pipe and locked into the point at its bottom. The shell is then filled with concrete and the drive pipe is withdrawn.

(5) The Raymond Taper or Step-Taper piles are steel shell piles driven with a tapered steel mandrel. The mandrel and shells are driven to the required depth. The mandrel is then contracted and withdrawn, and the shell is concreted with or without a reinforcing cage.

(6) A simplex pile is formed by driving a steel tube with a detachable cast iron shoe. After the required depth has been reached, reinforcing cage is lowered. The tube is extracted by wire ropes connected to a winch. At the same time, the concrete is placed and rammed by a falling rammer working inside the cage.

(7) Alpha piles are formed by driving a steel tube closed with a detachable cast iron shoe. A concrete –filled mandrel is driven inside the tube. The mandrel is gradually raised and some concrete is allowed to slump down in the tube. The concrete is refilled in the mandrel and it is driven down as the tube is raised. Thus a pedestal is formed. After the formation of the pedestal, the mandrel is raised and refilled with concrete in stages. In each stage, the concrete in the pile shaft is pressed against the soil by the dead weight of the hammer on the mandrel.

LOAD – CARRYING CAPACITY OF PILES

Like a shallow foundation, a pile foundation should be safe against shear failure and also the settlement should be within the permissible limits. The methods for estimating the load-carrying capacity of a pile foundation can be grouped into the following 4 categories.

(1) Static Methods. The static methods give the ultimate capacity of an individual pile, depending upon the characteristics of the soil. The ultimate load capacity is given by

$$Q_u = Q_p + Q_s$$

Where Q_u = ultimate failure load, Q_p = point (or base or tip) resistance of the pile (Fig. 25.2), Q_s = shaft resistance developed by friction (or adhesion) between the soil and the pile shaft.

The methods for the determination of Q_p and Q_s are discussed in sects. 25.8 and 25.9, respectively, for sand and clay.

The static formulas give a reasonable estimate of the pile capacity if judiciously applied.

(2) Dynamic Formulas. The ultimate capacity of piles driven in certain types of soils is related to the resistance against penetration developed during driving operation. The ultimate load capacity formulas are based on the principle that the resistance of a pile to further penetration by driving depends upon the energy imparted to the pile by the hammer. It is tacitly assumed that the load-carrying capacity of the pile is equal to the dynamic resistance during driving.

The dynamic formulas are not much reliable.

(3) In-situ Penetration Tests. The pile capacity can be determined from the results of in-situ standard penetration test. Empirical formulas are used to determine the point resistance and the shaft resistance from the standard penetration number (N). Alternatively, the static formulas can be used after determining the N-value, as this value is related to the angle of shearing resistance (Φ).

Cone penetration tests are also used to estimate the pile capacity.

(4) Pile load Tests. The most reliable method of estimating the pile capacity is to conduct the pile load test. The test pile is driven and loaded to failure. The pile capacity is related to the ultimate load or the load at which the settlements do not exceed the permissible limits.

All the above methods are discussed in detail in the following sections.

Static methods for driven piles in sand

The ultimate capacity of a single pile driven into sand is obtained using Eq. 25.1, $Q_u = Q_p + Q_s$

Where $\quad Q_p = q_p \; A_p \quad$ ....(25.2)

and $\quad Q_s = f_s \; A_s \quad$ ....(25.3)

In above equations, q_p is the ultimate bearing capacity of the soil at the pile tip and A_p is the area of the pile tip; f_s is the average unit skin friction between the sand and the pile surface, and A_s is the effective surface area of the pile in contact with the soil.

(a) Methods for determination of Q_p. The ultimate bearing capacity (q_p) of the soil at the pile tip can be computed from the bearing capacity equation similar to that for a shallow foundation, as discussed in chapter 23. For sandy soils,

Where = effective vertical pressure at the pile tip, B = pile tip width (or diameter),

γ = unit weight of the soil in the zone of the pile tip.

N_q and N_γ = bearing capacity factors for deep foundations.

In driven piles, the second term of Eq. 25.4 is generally small and is, therefore, neglected. Thus

In case of driven piles, it has been established that the effective vertical pressure at the pile tip increases with depth only until a certain depth of penetration, known as the critical depth (D_c). Below the critical depth, the effective vertical pressure remains essentially constant [Fig. 25.3(a)]. The critical depth depends upon the angle of shearing resistance (Φ′) of the soil and the width (or diameter) of the pile [Fig. 25.3(b)]. Its value can be roughly taken as 10 B for loose sands and 20 B for dense sands.

The bearing capacity factor N_q depends upon the angle of shearing resistance (Φ′). Various investigators gave the expressions for N_q based on theoretical analysis. These values vary over a wide range because of different assumptions made in defining the shear zone near the pile tip. Fig. 25.4 shows the values of N_q given by various investigators and that given by IS = 2911. The values given by Berezontzev are quite dependable, and are generally used.

In the derivation of the value of Nq, it has been assumed that the soil above the pile tip is similar to the soil below the pile tip. If the pile penetrates a compact stratum only slightly and the soil above the tip is loose, it would be more appropriate to use the value of Nq for a shallow foundation given in chapter 23.

If the pile is of relatively large diameter, the second term in Eq. 25.4 becomes significant. The value of N_γ can be conservatively taken as the N_γ value used for shallow foundations, given in Chapter 23.

Meyerhof's method for q_p. The point bearing capacity (q_p) of a pile generally increases with the depth of embedment (D_b) in the bearing stratum. It reaches a maximum value at an embedment ratio of $(D_b/B)_{cr}$. For a homogeneous soil, D_b is equal to the actual depth D of the pile, but for a pile which has penetrated into a bearing stratum for a small length, D_b is less than D. Beyond the critical value of $(D_b/B)_{cr}$, the value of q_p remains constant, equal to the limiting q_l. The critical ratio $(D_b/B)_{cr}$, depends upon the soil friction angle (Φ) (Fig. 25.5).

Once the value of $(D_b/B)_{cr}$, has been determined, the following procedure is used to estimate q_p.

(1) Determine actual (D_b/B) ratio for the pile,

(2) Determine N_q for (D_b/B) ratio from Fig. 25.5.

The value of N_q increases linearly with (D_b/B) ratio and reaches a maximum value at $D_b/B = 1/2\ (D_b/B)_{cr}$.

(3). Determine the point resistance Q_p as

Where q_l = 50 N_q tan Φ, = vertical pressure at the pile tip (kN/m^2), A_p = area of the pile tip.

If the pile initially penetrates a loose sand layer and then a dense layer for a depth less than 10 B, the point resistance is given by

Where $q_{1(1)}$ = limiting unit point resistance of loose sand $(=50\ N_{q1}\ \tan \Phi_1)$

$q_{1(2)}$ = limiting unit point resistance of dense sand $(=50\ N_{q2}\ \tan \Phi_2)$

D_b = depth of penetration in dense sand.

It may be mentioned that the ultimate tip resistance given by Eq. 25.2 is the gross ultimate point resistance. The net tip load is given by

However, in practice, the deduction of is not usually made and Q_p (net) is taken equal to Q_p.

In case of H-piles and open-ended pipe piles, the enclosed soil plug should be considered as the part of the pile for computing the area of the point (A_p).

(b) Methods of determination of Q_s. The frictional resistance Q_s is obtained from Eq. 25.3 after estimating the unit skin friction (f_s). The unit skin friction for a straight – sided pile depends upon the soil pressure acting normal to the pile surface and the coefficient of friction between the soil and the pile material (Fig. 25.6).

The soil pressure normal to the vertical pile surface is horizontal pressure (σ_h) and is related to the effective vertical soil pressure as

Where K = earth pressure coefficient, = effective vertical pressure at that depth.

Thus unit skin friction (f_s) acting at any depth can be written as

Where tan δ coefficient of friction between sand and the pile material.

Selection of suitable values of δ and K requires good engineering judgment. Tomilson (1975) gave the values of δ and K, as given in Table 25.1, based on the studies carried by Broms (1966).

Table 25.1 values of δ and K.

Pile Material	δ	K (loose sand)	K (dense sand)
Steel	20^0	0.50	1.0
Concrete	0.75 Φ	1.0	2.0
Timber	0.67 Φ	1.5	4.0

In general, the value of δ generally varies between 0.5 Φ and 0.8 Φ. In most cases, the value of *K* varies between 0.6 and 1.25. Meyerhof (1956) recommends that the value of *K* can be taken as 0.5 for loose sand (Φ = 30^0) and as 1.0 for dense sand (Φ = 45^0). According to IS : 2911 – 1979, the value of δ may be taken equal to Φ. For driven piles in loose to medium sands, the recommended value of *K* is between 1 and 3.

Whether the sand should be considered as loose or dense depends upon not only on the initial relative density, but also on the method of installation. The larger the volume of soil displacement, the higher the value of the resulting friction. For high displacement driven piles, the soil is considered dense. For driven and cast-in place piles, the soil is considered as medium dense if the casing is left in place or if the concrete is compacted as the casing is withdrawn. The sand is considered to be loose, if the concrete is not compacted. Tapered piles develop greater unit friction than the straight piles. Further, the value of K is greater if the pile is driven into undisturbed soil than the one for installed in a predrilled hole.

As stated earlier, the effective vertical pressure increases with depth only up to the critical depth. Below the critical depth, the value of remains constant.

The frictional resistance (Q_s) can be expressed as

Where n = number of layers in which the pile is installed.

= effective normal stress in ith layer,

$(A_s)i$ = surface area of the pile in ith layer, Assuming linear variation of σ_v,

$$Q_s = K \tan \delta \, Ap \times (q_0 D)$$

Where A_p = perimeter of pile; q_0 = average effective pressure =

And D = depth of pile.

Eq. 25.9 (a) can be written as

STATIC METHOD FOR DRIVEN PILES IN SATURATED CLAY

Eq. 25.1 can be used for the determination of the load-carrying capacity of driven piles in saturated clay. The point resistance *(Q_p)* can be expressed as (Eq. 25.2),

$$Q_p = q_p A_p$$

Where q_p is the unit point resistance, equal to the ultimate bearing capacity (q_u) of the soil.

For cohesive soils (Φ = 0), the ultimate bearing capacity is found from the following equation, which is similar to that for a shallow foundation.

$$q_u = cN_c + q N_q$$

As Nq = 1.0 for Φ = 0, the above equation becomes

$$q_u = cN_c + q$$

Therefore, $Q_p\ (gross) = (cN_c + q)\ A_p$

Or $Q_p\ (net) = cN_c\ A_p$

In above equations c is the cohesion of the clay in the zone surrounding the pile tip, and N_c is the bearing capacity factor for the deep foundation.

The value of N_c depends upon the D/B ratio and it varies from 6 to 9.0 A value of N_c = 9.0 is generally used for the piles. in the case of short piles (D/B ≤ 5.0), the value of N_c is reduced to the values proposed by Skempton (see chapter 23).

The skin resistance (Q_s) of the pile can be expressed as (Eq. 25.3),

$$Q_s = c_a A_s$$

Where c_a = unit adhesion (or skin friction) developed between clay and pile shaft.

The unit adhesion *(c_a)* is related to the unit cohesion by the relation

C_a = α ………..

Where α is the adhesion factor and is the average cohesion along the shaft length.

The value of α depends upon the consistency of the clay. For normally consolidated clays. The value of α is taken as unity. According to IS: 2911 – 1979, the value of α can be taken as unity for soils having soft to very soft consistency. Fig. 25.7 shows the variation of α with the un-drained cohesion c. it may be noted that for normally consolidated clays, with c less than about 50 kN/m^2, the value of α is equal to unity.

As c increases, the value of α decreases. For over-consolidated stiff to hard clays, its value is usually taken as 0.3. For tapered piles, the value of α is generally 20% greater than that for a straight pile.

For very long piles (D ≥ 25 m), the above method for estimating the skin friction is very conservative. For such soils, the unit skin friction also depends upon the effective over burden pressure. According to Vijayvergiya and Focht (1972), the average unit skin friction can be expressed as

$$f_s = \lambda(\quad + 2c)$$

where λ = friction capacity factor, = mean effective vertical stress for the embedment length,

c = undrained cohesion.

The value of λ can be obtained from Fig. 25.8 (McClelland, 1974).

Once the unit skin friction has been estimated, the shaft resistance is determined from Eq. 25.3.

For cohesive soils, the ultimate load can be determined by adding the point resistance and the shaft resistance (Eq. 25.1).

Thus $$Q_u = cN_c A_p + \alpha \quad A_s \qquad \ldots(25.15)$$

As the clay gets remoulded when the pile is driven, this factor must be taken in to account when estimating the load carrying capacity. The remoulded strength is always less than the undisturbed strength, but because of thixotropy, the strength improves with time. The rate of gain of strength depends upon the consolidation characteristics of the soil and the rate of dissipation of excess pore water pressure. When using Eq. 25.15, the value of c and should be judiciously evaluated.

STATIC METHOD FOR BORED PILES

Bored piles are constructed by drilling a hole into the ground and filing it with concrete. The pile can be straight – sided for its full depth or may be constructed with a bell (or pedestal) at its base. The piles with a pedestal are also known as under-reamed piles.

The load-carrying capacity of the bored piles can be determined using the procedure similar to that adopted for the driven piles. However, the values of the soil parameters are different, as described below.

(a) Bored Piles in Sand. Eq. 25.10 can be used to determine the ultimate load. The equation can be written as

Where = effective vertical pressure, limited to a maximum value given by the critical depth.

K = lateral earth pressure coefficient for bored foundation.

tan δ = coefficient of friction between sand and concrete.

The sand in bored piles is loosened as a result of the boring operation. Even though it may initially be in a dense or medium dense state. The value of Φ to be used to obtain N_q should be for the loose condition.

An approximate value of K can be obtained from the following equation.

$$K = 1 - \sin \Phi$$

The value of *K* generally varies between 0.3 and 0.75. An average value of 0.5 is usually adopted.

The value of tan δ can be taken equal to tan Φ for bore piles excavated in dry soil. If a slurry has been used during excavation, the value of tan δ should be reduced.

In general, for a given initial value of Φ, bored piles have a unit resistance of ½ to 2/3 of that of corresponding driven piles. In driven piles, there is densification. Cast-in-place piles with a pedestal show about 50 to 100% greater unit point resistance compared with those without a pedestal. The impact energy of the hammer compacts the soil during the formation of the pedestal.

(b) Bored Piles in Clay. Eq. 25.15 can be used to estimate the ultimate load. The equation can be written as

$$Q_u = cN_c A_p + \ldots\ldots$$

Where As = area of shaft that is effective in developing skin friction.

The value of α depends upon the pile type and the method of drilling. For straight shafts excavated dry, α is taken equal to 0.5 and that when drilled with slurry is 0.3. For belled shafts, the corresponding values are 0.3 and 0.15.

For calculating the area of shaft that is effective in developing skin friction, the lower 1.5m (or 2 B) of the straight shaft and the bell section (if provided) are neglected, because of disturbance caused, for the same reason, the top 1.5 m is also neglected.

If a bored pile is installed if stiff, fissured clay, the value of cohesion (c) should be reduced to 75% of the value obtained from the triaxial test.

(c) Under reamed Piles in Clay. The base area of an under reamed pile is increased by under reaming and providing a bulb [Fig. 25.9 (a)]. The ultimate load is given by

$Qu = cNc\ Ab +$ A'_s …. [25.17 (a)]

Where A_b is the area of the enlarged base. The value of N_c is taken as 9.0. The adhesion factor α is taken as 0.40. When the bulb is slightly above the tip, A_b is taken equal to the area of the diameter of the bulb and the projected stem below the bulb is ignored. The average value of c at the bulb is taken. However, if the bulb is quite long, and there is considerable difference in the value of c at the bulb level and the level of the bottom tip of the pile, the ultimate load is given by

Where B = diameter of the pile shaft, B_1 is the diameter of the bulb, c is the unit cohesion at the tip, and c′ is the unit cohesion at bulb level. is the average cohesion on the shaft.

While calculating the surface area A_s, the length of the shaft equal to 2 B above the bulb is usually neglected. As the pile settles, there is a possibility of formation of a small gap between the top of the bulb and the overlying soil over a length of 2 B, and therefore, this length of the shaft is neglected. The little portion of the shaft projecting below the shaft is also neglected while computing A'_s.

When two or more bulbs are provided, the ultimate load is given by

Where A_s = surface area of shaft above the top bulb (ignoring 2 B length), A_{sb} = surface of the cylinder circumscribing the bulbs between top and bottom bulbs, c_a = average cohesion on A_s and c'_a = average cohesions on A_{sb}. For more details, see chapter 34.

ALLOWABLE LOAD

The allowable load (Q_{all}) is obtained from the ultimate load (Q_u) from the relation

$Q_{all} = Q_u/FS$

where FS is the factor of safety. FS generally varies between 2.5 and 4.0, depending upon the uncertainties involved in the computation of the ultimate load. According to IS : 2911 – 1979, the minimum factor of safety on static formula shall be 2.5. The final selection of the value of the factor of safety should take into account the load settlement characteristics of the structure as a whole.

NEGATIVE SKIN FRICTION

When the soil layer surrounding a portion of the pile shaft settles more than the pile, a downward drag occurs on the pile. The drag is known as negative skin friction.

Negative skin friction develops when a soft or loose soil surrounding the pile settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile. The negative friction imposes an extra downward load on the pile. The magnitude of the negative skin friction is computed using the same method as discussed in the preceding sections for the (positive) frictional resistance. However, the direction is downwards.

The net ultimate load-carrying capacity of the pile is given by the equation (Fig. 25.10).

$Q'_u = Q_u - Q_{nsf}$ ....(25.19)

Where Q_{nsf} = negative skin friction,

Q'_u = net ultimate load.

Where it is anticipated that negative skin friction would impose undesirable, large downward drag on a pile, it can be eliminated by providing a protective sleeve or a coating for the section which is surrounded by the settling soils.

DYNAMIC FORMULAE

The load-carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving operation. The methods give fairly good results only in the case of free draining sands and hard clays in which draining sands and hard clays in which high pour water pressures does not develop during the driving of piles. In saturated fine-grained soils, high pour 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 a much less resistance.

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

$W_h \eta_h = R X S$

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

In Eq. 25.20, no allowance has been made for the loss of energy during driving operation, loss caused by elastic contraction of the pile, soil, pile cap, cushion and due to the inertia of the pile. Some energy is also lost due to generation of heat. Various formulae have been proposed , which basically differ only in the methods for accounting of the energy losses, as described below.

(1) Engineering News Record Formula. According to Engineering News Record (ENR) formula (1888), the ultimate load is given by

………. (25.21)

Where S = penetration of pile per hammer blow. It is generally based on the average penetration obtained from the last few blows (cm), C = constant (For drop hammer, C = 2.54 cm and for steam hammer, C = 0.254 cm)

In Eq. 25.21, the product W X h can be replaced by the rated energy of hammer (En) in kN-cm. Thus

………. (25.22)

The efficiency η_h of the drop hammer is generally between 0.7 and 0.9, and that for a single – acting and a double – acting hammer is between 0.75 and 0.85. For diesel hammer, it usually lies between 0.80 and 0.90.

A factor of safety of 6 is usually recommended. However, the pile load tests reveal that the actual factor of safety varies between 2/3 and 30. The formula is, therefore, not dependable.

Modified Formula. The Engineering News Record formula has been modified recently. In the modified formula, the energy losses in the hammer system and that due to impact are considered. According to this formula.

………. [25.21(a)]

Where P = weight of pile; e = coefficient of restitution, and η_h = hammer efficiency.

The hammer efficiency (η_h) depends upon various factors, such as pile driving equipment, driving procedure, type of pile and the ground conditions. For drop hammers, it is usually taken between 0.75 and 1.0; for single acting hammers between 0.75 and 0.85; for double – acting or differential hammer, η_h =0.85 and for diesel hammer, η_h =0.85 to 1.00.

The representative values of the coefficient of restitution (e) are as under.

Brimmed timber pile	= 0.0
Good timber pile	= 0.25
Driving cap with timber dolly on steel pile	= 0.3

Driving cap with plastic dolly on steel pile = 0.5

Helmet with composite plastic dolly and packing on R.C.C. pile = 0.4

(2) Hiley Formula. Hiley (1925, 1930) gave a formula which takes into account various losses.

……..(25.23)

Where η_h =efficiency of hammer blow, h = height of free fall of the ram or hammer (cm), S = final set or penetration per blow (cm), C = sum of temporary elastic compression of the pile, dolly, packings and ground *(= C_1 + C_2 + C_3),* C_1 temporary compression of dolly and packing (= 1.77 R/A, when the driving is without dolly, = 9.05 R/A, when the driving is with short dolly), C_2 = temporary compression of pile (= 0.657 RD/A), C_3 = temporary compression of ground (= 3.55 R/A), D = length of the pile, A = cross-sectional area of pile, R = pile resistance (tones).

The efficiency of hammer blow (η_b) depends upon the weight of hammer (W), weight of pile, anvil and helmet follower (P) and the coefficient of restitution (e).

(a)

….(25.24)

(b)

….(25.25)

The coefficient of restitution (e) varies from zero for a deteriorated condition of the head of pile to 0.5 for steel ram of double – acting hammer striking on steel anvil and driving a reinforced concrete pile. For a C.I. ram of a single-acting or drop hammer striking on the head of R.C.C. pile, e = 0.4 an that striking on a well-conditioned driving cap and helmet with hard wood on R.C.C. pile, e = 0.25 (IS : 2911-1979).

(3) Danish Formula. According to Danish formula (1929),

….(25.26)

Where

….(25.27)

In which S_o = elastic compression of pile, D = length of pile, A = cross-sectional area, E = modulus of elasticity of pile material.

The allowable load is found by taking a factor of safety of 3 to 4.

Eq. 25.27 can also be used to determine the final set (S) per blow.

Taking $Q_u = 3\ Q_a,$

….(25.28)

Where Q_a = allowable load.

WAVE EQUATION ANALYSIS

As the hammer strikes the top of a pile, a stress wave is transmitted through the length of the pile. The wave transmission theory can be used to determine the load carrying capacity of the pile and the maximum stresses that can occur within the pile during driving operation.

In the wave equation analysis (Smith, 1962), the pile is represented by a series of individual spring-connected weights and spring damping resistance (Fig. 25.11). The weight W_1 represents the weight of the ram, and W_2 represents the weight of the pile cap. Weights W_3 to W_{10} correspond to the weights of incremental sections of the pile. The spring constant K_1 represents the elasticity of the cap block; the constants K_2 to K_{11} are for the elasticity of the pile sections. The damping springs R_3 to R_{11} represent the frictional resistance of the soil surrounding the shaft; R_{12} represents the soil resistance at the pile tip.

The propagation of the elastic wave through the pile is analogous to that caused by an impact on a long road. A partial differential equation is written to describe the pile model shown in Fig. 25.11 (a). The equation is solved with the aid of a digital computer, and the pile capacity is determined. The pile capacity is expressed as a function of penetration per blow or blows per cm [Fig. 25.11 (b)].

The major drawback of the wave equation analysis for determination of the dynamic resistance is its dependence on a computer. Moreover, the field tests ar required to estimate the equivalent spring constant and soil-damping values for the pile under study. Further, the result obtained is valid only for a particular pile driven by a specified pile hammer.

Despite the above shortcomings, the wave equation analysis is a useful tool for determining the pile capacity. The results can also be used for the selection of appropriate pile-driving equipment.

IN-SITU PENETRATION TESTS FOR PILE CAPACITY

(a) Standard penetration test. The load-carrying of a pile can be estimated from the standard penetration test value (N).

(i) For driven piles in sand, the unit tip resistance (q_p) is related to the uncorrected blow count (N) near the pile point (Meyerhof 1976).

$$Q_p = 40\ N\ (D/B) \leq 400\ N \qquad ….(25.29)$$

Where q_p = point resistance (kN/m^2), D = length of pile, B = width (diameter) of pile.

The value of q_p is usually limited to 400 N.

The average unit frictional resistance (fs) is related to the average value of the blow count ( ).

For high displacement piles, $f_s = 2.0 \quad kN/m^2$ …. [25.30 (a)]

For low displacement piles, $f_s = 1.0 \quad kN/m^2$ ….[25.30 (b)]

Where is average of uncorrected N-vales along the length of the pile.

(ii) For bored piles in sand, q_p = 14 N(Db/B) kN/m2 (25.31)

Where D_b = actual penetration into the granular soil.

For bored piles in sand, the unit frictional resistance (fs) is given by

….(25.32)

(b) Dutch cone test. Meyerhof (1965) relates the unit point resistance (q_p) and the unit skin traction (f_s) of driven piles to the cone point resistance *(q_c)*.

Point resistance, ….(25.33)

Unit skin friction (a) f_s (dense sand) $= q_c/200$ ….(25.34)

(b) f_s (loose sand) $= q_c/400$ ….(25.35)

(c) f_s (slit) $= q_c/150$ ….(25.36)

PILE LOAD TEST

The most reliable method for determining the load carrying capacity of a pile is the pile load test. The set-up generally consists of two anchor piles provided with an anchor girder or a reaction girder at their top (Fig. 25.12). the test pile is installed between the anchor piles in the manner in which the foundation piles are to be installed. The test pile should be at least 3 B or 2.5m clear from the anchor piles.

The load is applied through a hydraulic jack resting on the reaction girder. The measurements of pile movement are taken with respect to a fixed reference mark. The test is conducted after a rest period of 3 days after the installation in sandy soils and a period of one month in silts and soft clays. The load is applied in equal increment of about 20% of the allowable load. Settlements should be recorded with three dial gauges. Each stage of the loading is maintained till the rate of movement of the pile top is not more than 0.1 mm per hour in sandy soils and 0.02 mm per hour in case of clayey soils or a maximum of two hours (IS : 2911-1979). Under each load increment, settlement is observed at 0.5, 1, 2, 4,8,12,16,20,60 minutes. The loading should be continued up to twice the safe load or the load at which the total settlement reaches a specified value. The load is removed in the same decrements at 1 hour interval and the final rebound is recorded 24 hours after the entire load has been removed.

Fig. 25.13 shows a typical load-settlement curve (firm line) for loading as well as unloading obtained from a pile load test. For any given load, the net pile settlement (s_n) is given by

$$s_n = s_t - s_e$$

Where s_t = total settlement (gross settlement), s_e = elastic settlement (rebound).

Fig. 25.13 also shows the net settlement (chain dotted line).

Fig. 25.14 shows two load-net settlement curves obtained from a pile load tests on two different soils. At the ultimate load (Q_u), the load-net settlement curve becomes either linear as curve (2) or there is a sharp break as in the curve (1), as shown in the figure. The safe load is usually taken as one-half of the ultimate load.

According to IS :2911, the safe load is taken as one-half of the load at which the total settlement is equal to 10 per cent of the pile diameter (7.5 per cent in case of under-reamed piles) or two-thirds of the final load at which the total settlement is 12 mm, whichever is less. According to another criterion, the safe load is taken as one-half to two-thirds of the load which gives a net settlement of 6mm.

The limiting settlement criteria are also sometimes specified. Under the load twice the safe load, the net settlement should not be more than 20 mm or the gross settlement should not be more than 25mm.

The test described above is known as initial test. It is carried out on a test pile to determine the ultimate load capacity and hence the safe load. The pile load test described in this section is a type of load-controlled test, in which the load is applied in steps. The test is also known as slow maintained test.

OTHER TYPES OF PILE LOAD TESTS

(1) Constant rate of penetration test. In a constant-rate of penetration test, the load on the pile is continuously increased to maintain a constant rate of penetration (from 0.25 to 5 mm per minute). The force required to achieve that rate of penetration is recorded, and a load-settlement curve is drawn. The ultimate load can be determined from the curve.

The test is considerably faster than a load-controlled test.

(2) Routine Load test. This test is carried out on a working pile with a view to determine the settlement corresponding to the allowable load. As the working pile would ultimately form a part of the foundation, the maximum load is limited to one and a half times the safe load which gives a total settlement of 12 mm.

(3) Cyclic Load test. The test is carried out for separation of skin friction and point resistance of a pile. In the test, an incremental load is repeatedly applied and removed.

(4) Lateral Load test. The test is conducted to determine the safe lateral load on a pile. A hydraulic jack is generally introduced between two piles to apply a lateral load. The reaction may also be suitably obtained from some other support. The test may also be carried out by applying a lateral pull by a suitable set-up.

(5) Pull out test. The test is carried out to determine the safe tension for a pile. In the set-up, the hydraulic jack rests against a frame attached to the top of the test pile such that the pile gets pulled up.

GROUP ACTION OF PILES

A pile is not used singularly beneath a column or a wall, because it is extremely difficult to drive the pile absolutely vertical and to place the foundation exactly over its centre line. If eccentric loading results, the connection between the pile and the column may break or the pile may fail structurally because of bending stresses. In actual practice, structural loads are supported by several piles acting as a group. For columns, a minimum of three piles in a triangular pattern are used. For walls, piles are installed in a staggered arrangement on both sides of its centre line. The loads are usually transferred to the pile group through a reinforced concrete slab, structurally tied to the pile tops such that the piles act as one unit. The slab is known as a pile cap. The load acts on the pile cap which distributes the load to the piles.

The load carrying capacity of a pile group is not necessarily equal to the sum of the capacity of the individual piles. Estimation of the load-carrying capacity of a pile group is a complicated problem. When the piles are spaced a sufficient distance apart, the group capacity may approach the sum of the individual capacities. On the other hand, if the piles are closely spaced, the stresses transmitted by the piles to the soil may overlap, and this may reduce the load-carrying capacity of the piles. For such a case, the capacity is limited by the group action.

The efficiency (η_g) of a group of piles is defined as the ratio of the ultimate load of the group to the sum of individual ultimate loads.

Where $Q_{g\,(u)}$ = ultimate load of the group, Q_u = ultimate load of the individual pile,

N = Number of piles in the group.

Thus the groups efficiency is equal to the ratio of the average load per pile in the group at which the failure occurs to the ultimate load of a comparable single pile,

The group efficiency depends upon the spacing of the piles. Ideally, the spacing should be such that the efficiency is 100%. Generally, the centre to center spacing is kept between 2.5 B and 3.5 B, where B is the diameter of the pile.

The methods for the determination of the ultimate load of the individual piles have been discussed earlier. The methods for the estimation of the ultimate load of the group are explained in the following sections.

PILE GROUPS IN SAND AND GRAVEL

For piles driven in loose and medium dense cohesionless soils, the group efficiency is high. The soil around and between the piles is compacted due to vibration caused during the driving operation. For better results, it is essential to start driving the piles at the center and then work outward.

The piles and the soil between them move together as a unit when subjected to loads. The group acts as a pier foundation having a base equal to the gross plan area contained between the piles.

(a) End – bearing piles. For driven piles bearing on dense, compact sand with a spacing equal to or greater than 3 B, the group capacity is generally taken equal to the sum of individual capacity. Thus

$Q_g = NQ_u$

In this case, the load taken by the group is much greater (η_g > 100%) than the sum of the individual capacities, and the piles fail as individual piles.

For spacing less than 3 B, the group capacity is found for the block of piles group.

(b) Friction piles. The group efficiency of friction piles in sand is obtained from the following expression:

….(25.40)

Where P_g = perimeter of the block, P = perimeter of the individual pile, D = length of pile,

f_s = unit friction resistance.

If the centre-to-centre spacing is large, the group efficiency (ηg) may be more than 100%. The piles will behave as individual piles, and the group capacity is obtained from Eq. 25.39.

If η_g is less than 100%,

….(25.41)

The group efficiency can also be obtained from the converse- Lebarre equation given below.

….(25.42)

Where m = number of rows of piles, n = number of piles in a row, θ = tan-1 (B/s), B = diameter of pile, s = spacing of pile, centre-to-centre, η_g = group efficiency (expressed as a ratio).

Bored piles. For bored piles in sand at conventional spacing of 3 B, the group capacity is taken as 2/3 to ¾ times the sum of individual capacities for both the end-bearing and the friction piles. Thus

$Q_g\ (u) = (\ 2/3\ \text{ to } ¾\ (NQ_u)$ ….(25.43).

In bored piles, there is limited densification of the sand surrounding the pile group. Consequently, the efficiency is lower.

PILE GROUPS IN CLAY

As the pile group acts as a block, its ultimate capacity is determined by adding the base resistance and the shaft resistance of the block. The capacity of the block having closely spaced piles (s ≤ 3 B) is often limited by the behavior of the group acting as a block. The group capacity of the block is given by

Or $$Q_g\,(u) = q_p(A_g) + \alpha\, c\,(P_g\, D)$$

Where q_p = point resistance (N_c = 9.0), Ag = base area of the block, P_g = perimeter of the block, D = depth of the block, α adhesion factor (= 1.0 for soft clays), c = un drained cohesion.

As discussed earlier, the individual pile capacity is given Eq. 25.15,

$$Q_u = q_p\, A_p + \alpha\, c\,(pxD) \qquad \text{….(25.45)}$$

The group capacity considering the piles as individual piles is given by

$$Q_g\,(u) = N\, Q_u \qquad \text{….(25.46)}$$

The lower of the two values, given by Eqs. 25.44 and Eq. 25.46, is the actual capacity.

SETTLEMENT OF PILE GROUPS

The settlement of a pile group is due to elastic shortening of piles and due to the settlement of the soil supporting the piles. it is assumed that the pile group acts as a single large deep foundation, such as a pier or a mat. The total load is assumed to act at a depth equal to two-thirds the pile length in the case of frictional piles [Fig. 25.17 (a)]. In the case of end-bearing piles, the total load is assumed to act at the pile tips [Fig. 25.17 (b)]. In the case of combined action, the frictional component is assumed to act at 2/3 D and the bearing component at the tip.

For determination of the settlements, the compression characteristics of the soil are required. For clayey soils, the characteristics are determined from laboratory tests on undisturbed samples. For cohesionless soils, the characteristics are obtained from empirical correlations developed from in-situ penetration tests.

(a) Cohesionless soils

(i) Skempton method. The settlement of the pile group is estimated from the settlement of single pile, as determined in a pile-load test. The settlement of the group is generally very large because the pressure bulb for the group is much deeper than that of a single pile.

Skempton et al (193) published curves (Fig. 25.18) relating the settlement of the pile group (s_g) of a given total foundation width to that of a single pile (s_o). The curves can be used for both driven and bored piles.

(ii) Meyerhof method. Meyerhof (1976) suggests the following empirical relation for the elastic settlement of a pile group in sands and gravels.

….(25.47)

Where s_g = settlement of group (mm), q = load intensity (= Qg/Ag), B_g = width of the group, I – influence factor [= 1 – D/(8 B_g) ≥ 0.5], D = length of pile, N = corrected standard penetration number within the seat of settlement (approximately equal to B_g below the tip).

If static cone results are available, the settlement of the group can be obtained from the relation

….(25.48)

Where q_c = average cone penetration resistance within the seat of settlement.

(b) Clayey soils

The consolidation settlement of a pile group in clay can be determined using the procedure discussed in chapter 12. Generally, a 2 : 1 load distribution is assumed from the level at which the load acts. Sometimes, the load is assumed to spread outwards from the edge of the block at an angle of 30^0 to the vertical. For 2 : 1 distribution, the stress increase at the middle of each layer is calculated as (see chapter 11),

….(25.49)

Where Z_i is the distance from the level of the application of the load to the middle of clay layer i. The settlement of each layer caused by the increased stress is given by (see chapter 12).

….(25.50)

Where Δe (i) = change of void ratio caused by the stress increase, e_o (i) = initial void ratio of layer I, H_i = thickness of layer i.

Alternatively,

….(25.51)

The total consolidation settlement is equal to the sum of the settlement of all layers.

$$S_g = \Sigma \Delta s\ (i)$$

SHARING OF LOADS IN A PILE GROUP

All piles in a group share equal load if the load is central.

….(25.53)

However, if the load is eccentric or if the central load is accompanied by a moment, the sharing of load is computed assuming the pile cap as rigid. As the pressure distribution is planar, the pile reactions also vary linearly with the distance from the centroid of the cap (Fig. 25.19). The axial load in any pile m at a distance x from the centroid is given by

….(25.54)

Where e_x = eccentricity of load about Y-Y-axis, if the load is eccentric about both the axes.

….(25.55)

Where e_y = eccentricity of load about X-X axis,

In the above equations, the positive sign is taken for the piles on the same side as the eccentricity.

If the load on any pile is negative, it indicates that the pile is in tension. If the pile is not designed for tension, the load in that pile is taken as zero, and the load between other piles is redistributed. This would cause extra compression in other piles.

TENSION PILES

Piles supporting high structures, such as tall chimneys, transmission towers, water towers, are required to resist uplift forces due to wind. Some of the piles in these structures are required to resist tensile forces and are known as tension piles.

Resistance to uplift forces is provided by the friction between the pile and the surrounding soil. The uplift resistance of a straight – shaft pile can be computed I the same manner as the frictional resistance in frictional piles. However, the unit skin friction (f_s) and adhesion (c_a) for the uplift resistance are considerably less than those for the compressive loads. It is usual practice to reduce these values to one-half of the normal values if the piles are short. For large structures, it is essential to carry out pull out tests on piles to determine the safe value of the unit skin friction or adhesion fro uplift forces.

The uplift resistance of piles can be considerably increased in the case of bored piles by under – reaming or belling out the bottom. A bulb can also be formed in the case of driven and cast – in place piles to increase the uplift resistance.

Meyerhof and Adams (1968) gave the following equations for the pull – out resistance (P_u).

(a) Shallow piles Fig. [25.20 (a)]. Pull –out restance,

P_u = cohesive resistance + frictional resistance

Or P_u =

….(25.56)

Where B_1 = diameter of enlarged base, c_u = un drained cohesion, D = length of pile, Φ = angle of shearing resistance, s_f = shape factor (see Table 25.3), K_u = coefficient of lateral earth pressure (= K_p tan 2/3 Φ), K_p = γ = bulk unit weight, and W = weight of soil and pile in a cylinder of diameter B_1 and height D.

(b) Deep piles Fig. [25.20 (b)], Pull-out resistance,

P_u = cohesive resistance + frictional resistance

Or …(25.57)

Where H = maximum height of rupture surface (see Table 25.3) (For deep piles H ≤ D)

Table 25.3 Values of H/B_1 m and sf

Φ	20^0	25^0	30^0	35^0	40^0	45^0	50^0
H/B_1	2.5	3.0	4.0	5.0	7.0	9.0	11.0
m	0.05	0.10	0.15	0.25	0.35	0.50	0.60
sf	1.12	1.30	1.60	2.25	3.45	5.50	7.60

For purely cohesive soils, as Φ =0, the second term in Eqs. 25.56 and 25.57 is zero. For cohesionless soils, as c_u = 0, the first term is zero. The shape factor (sf) is equal to 1 + m D/B1 for short piles, and equal to 1 + mH/B_1 for deep piles, where m is a coefficient depending on Φ.

LATERALLY LOADED PILES

Piles are sometimes subjected to later loads due to wind pressure, water pressure, earth pressure, earthquakes, etc. when the horizontal component of the load is small I comparison with the vertical load (say, less than 20%), it is generally assumed to be carried by vertical piles and no special provision for lateral load is made.

If the horizontal load is large, inclined piles, known as raking piles or batter piles, are provided to take the horizontal load. These piles have a high resistance to lateral loads, as a large portion of the horizontal component of the load is carried axially by the pile. Batter piles, along with vertical piles, are provided in situations where the horizontal loads are significant, such as wharves, jetties, bridge piers, trestles, retaining wall and tall chimneys.

Batter piles are driven at a batter ranging from 1 : 12 to 1 : 25. However, driving of batter piles is more expansive than that of vertical piles. The resistance to failure of vertical piles subjected to horizontal loads is provided by the passive resistance of a wedge of soil in front of the piles. In case of batter piles, additional resistance is provided by the skin friction and the end bearing. Therefore, batter piles are more effective than vertical piles in resisting horizontal loads.

It is generally assumed that batter piles can take the axial load equal to that in the corresponding vertical pile. As the axis of the batter pile is inclined, it can resist t he horizontal load equal to Q cos θ, where Q is the axial load capacity and θ is the angle which the pile makes with the horizontal. When piles are oriented in two or three directions, Culmann's method, as described below, is used.

Steps :

(1) Group the piles according to their slopes. [In Fig. 25.21 (a), the piles are grouped in 3 directions].

(2) Draw the geometry of the pile group to some scale, and mark the directions of the inclined load Q_g and the centre line of each pile group (R_1, R_2 and R_3).

(3) Determine the location of point A which is at the intersection of R_1 and Q_g.

(4) Join A to the point B which is at the intersection of R_2 and R_3.

(5) Draw the force triangle [Fig. 25.21 (b)]. Select the line *ab* parallel to *AB*. From b draw a line b_c parallel to Q_g to some scale. Draw a vertical at *c* to determine ca which is equal to R_1.

From b draw a line parallel to R_3, and from a, line parallel to R_2, to complete the triangle *abd*.

(6) Determine forces in piles as follows. The magnitudes of R_2 and R_3 are, respectively, given by *ad* and *bd*. However, R_2 is compressive and R_3 is tensile.

The magnitude of R_1 is given by *ca* which is compressive.

Well Foundations

Well Foundations:

Well foundations have been used in India for centuries for providing deep foundations below water for monuments, bridges, and aqueducts. For example, the famous Taj Mahal at Agra stands on well foundations.

The construction of a well foundation is principle, similar to the conventional wells sunk for obtaining underground water; in fact, it derives its name owing to this construction technique. It is a monolithic and massive foundation and is relatively rigid in its engineering behavior.

Well foundations are similar to open Caissons referred to in Section 19.3. These are very popularly used to support bridge piers and abutments in India as they afford number advantages over other types of deep foundations for such large jobs.

Advantages of Well Foundations:

The following are the advantages of well foundations over other types of deep foundations such as pile foundations:

(i) The effect of scour can be better with stood by a well foundation because of its large cross – sectional area and rigidity.

(ii) The depth can be decided as the sinking progresses, since the nature of the strata can be inspected and tested, if necessary, at any desired stage. Thus, it is possible to ensure that it rests upon a suitable bearing stratum of uniform nature and bearing power.

(iii) A well foundation can withstand large lateral loads and moments that occur in the case of bridge piers, abutments, tall chimneys, and towers; hence it is preferred to support such structures.

(iv) There is no danger of damage to adjacent structures since sinking of a well does not cause any vibrations.

These advantages are not obtainable in the case of pile foundations, especially for large structures.

Well foundations have been found to be economical for large structures when a suitable bearing stratum is available only at large depths.

Elements of a well Foundation:

The elements of a well foundation are : (i) Cutting edge (ii) Curb (iii) Concrete seal or Bottom plug (iv) Steining (v) Top Plug, and (vi) Well Cap.

These shown in the sectional elevation of a typical well foundation of circular cross section.

(i) Cutting Edge: The foundation of the 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 it is subjected during the sinking process. Hence it usually consists of an angle iron with or without an additional plate of structural steel. It is similar to the sharp – edged cutting edge of a caisson shown in Fig.

(ii) Steining: The steining forms the bulk of the well foundation and may be constructed with brick or stone masonry, or with plain or reinforced concrete occasionally. The thickness of the steining is made uniform throughout its 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.

(iii) 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.

(iv) Concrete Seal or Bottom Plug: After the well foundation is sunk to the desired 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 as 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.

(v) 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 'top plug'.

The sand filling serves to distribute the load more uniformly to the base of the well, to reduce the stresses in the steining, and to increase the stiffness of the well foundation. However, as this adds to the weight and load transmitted to the foundation stratum, the engineer has to consider the desirability or otherwise of providing the sand filling from the point of view of bearing power and settlement.

The top plug of concrete serves to transmit the loads to the base in a uniform manner.

(iv) 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.

Tills and shifts

The well should be sunk straight and vertical at the correct position. It is not an easy task to achieve this objective in the field. Sometimes the well tilts onto one side or it shifts away from the desired position.

The following precautions may be taken to avoid tilts and shifts:

(i) The outer surface of the well curb and steining should be smooth.

(ii) The curb diameter should be kept 40 to 80 mm larger than the outer diameter of the steining, and the well should be symmetrically placed.

(iii) The cutting edge should be uniformly thick and sharp.

(iv) Dredging should be done uniformly on all sides and in all the pockets.

Tilts and shifts must be carefully noted and recorded. Correct measurement of tilt is an important observation in well sinking. It is difficult to specify permissible values for tilts and shifts. IS: 3955-1967 recommends that tilt should be generally limited to 1 in 60. The shift should be restricted to one percent of the depth sunk. In case these limits are exceeded, suitable remedial measures are to be taken for rectification.

Remedial Measures for Rectification of Tilts and Shifts

The following remedial measures may be taken to rectify tilts and shifts:

1. Regulation of Excavation : The higher side is grabbed more be regulating the dredging. In the initial stages this may be all right. Otherwise, the well may be dewatered if possible, and open excavation may be carried out on the higher side [Fig. 19.23 (a)]
2. Eccentric Loading : Eccentric placing of the kentledge may be resorted to provide greater sinking effort on the higher side. If necessary a plat form with greater projection on the higher side may be constructed and used for this purpose. As the depth of sinking increases, heavier kentledge with greater eccentricity would be required to rectify tilt [Fig. 19.23 (b)]
3. Water Jetting : If water jets are applied on the outer face of the well on the higher side, the friction is reduced on that side, and the tilt may get rectified [Fig. 19.23(c)].
4. Excavation under the Cutting Edge: If hard clay is encountered, open excavation is done under the cutting edge, if dewatering is possible; if not, divers may be employed to loosen the strata.
5. Insertion of Wood Sleeper under the Cutting Edge : Wood sleepers 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 [Fig. 19.23(d)].
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 sleepers to distribute the load from the strut. The other end of the logs rest against a firm base having driven piles [Fig. 19.23 (e)].
8. Pushing the Well with Jacks : Tilt can be rectified by pushing the well by suitably arranging mechanical or hydraulic jacks.

In actual practice, a combination of two or more of these approaches may be applied successfully [Fig. 19.23 (f)].

Shapes of Wells

The common types of well shapes are

1. Single Circular
2. Twin Circular
3. Dumb-Well
4. Double-D
5. Twin- hexagonal
6. Twin- octagonal
7. Rectangular

Depth of well foundations and bearing capacity :

The selection of the depth of a well is based on the following two criteria :

1. There should be adequate embedded length of well, called the grip length below the lowest scour level. In addition to minimum Rankine depth consideration, this is required for developing sufficient passive resistance to counteract the overturning moment due to horizontal forces acting on the bridge deck, as well as those due to wind and water.

2. The well should be taken deep enough to rest on strata of adequate bearing capacity in relation to the loads transmitted.

For alluvial soils, mostly met with in North Indian rivers, the normal scour depth can be calculated by Lacey's Formula :

$R_L = 1.35$

Where q = discharge in cumecs per linear metre of water way

f = Lacey's silt factor = 1.76

md = mean weighted diameter in mm.

The maximum depth of scour, at the nose of pier, is found to be twice the Lacey's value of normal scour depth :

$$R = 2R_L$$

Where R is measured below the high flood level (HFL).

$\therefore$ Scour level = H.F.L. $-R$ $= H.F.L. - 2R_L$

The grip length is taken as 1/3 R below the scour level according to the code of practice of the Indian Roads Congress and as ½ R in Railway practice. This means that the depth of foundation should be at least 1 1/3 R below HFL according to IRC code, and 1 ½ R below HFL according to Railway practice. It is further recommended that the minimum depth of embedment below the scour level should not be less than 2.0 m for piers and abutment with arches and 1.2 m for piers and abutments supporting other types of superstructure.

According to Terzaghi and Peck, the ultimate bearing capacity can be determined from the following expression :

$$Qf = Qp + 2\pi Rfs\ Df$$

$$Qp = \pi R2\ (1.2\ cNc + yDf\ Nq + 0.6y\ RNr)$$

Where Nc, Nq, Ny = Terzaghi's bearing capacity factors

R = radius of well

Df = depth of well (depth of foundation)

fs = average skin friction.

Forces acting on a well foundation :

In addition to the self –weight and buoyancy, a well carries the dead load of the super-structure, bearing pier and is liable to the following horizontal forces:

(i) Bracking and tractive effort of the moving vehicles.

(ii) Force on account of resistance of the bearings against movement due to variation of temperature,

(iii) Force on account of water current,

(iv) Wind forces,

(v) Seismic forces,

(vi) Earth pressure,

(vii) Centrifugal forces.

The magnitude, direction and point of application of all the above forces can be found under the worst possible combinations and they can be replaced by two horizontal forces, P and Q and a single vertical force W as shown in Fig. 27.3.

P = Resultant of all horizontal forces in the direction across the pier.

Q = Resultant of all horizontal forces in the direction along the pier.

W = Resultant of all vertical forces.

The analysis is done on the following assumptions (Banerjee and Gangopadyyay, 1960):

1. The well is acted upon by an uni-directonal horizontal force P in a direction across the pier.
2. The well is founded in sandy stratum.
3. The resultant unit pressure on soil at any depth is in simple proportion to horizontal displacement.
4. The ratio between contact pressure and corresponding displacement is independent of the pressure.
5. The co-efficient of vertical sub grade reaction has the same value for every point of surface acted upon by contact pressure.

The analysis that follows is that suggested by Banerjee and Gangopadhyay (1960).

Analysis of Well Foundation :

1. **Horizontal soil reactions :** When a rigid well, embedded in sand, starts moving parallel to its original position, under the action of a horizontal force P, it transforms the soil on one side to passive state of plastic equilibrium and the other side into active state. Assuming that the well movement ρ_1 is sufficient to mobilize fully the active and passive earth pressure, the resultant unit pressure at a depth z below the surface is given by $p_1 = y \, . \, z \, (K_p - K_a)$.

Where y = unit weight fo soil

Kp , Ka = co-efficient of passive and active earth pressure, and depend upon the angle of internal friction Φ, and angle of wall friction δ.

Let p be the load per unit area of vertical surface of sand and ρ be the corresponding displacement. Assuming that ρ_1 is the displacement required to increase the value of resultant unit pressure from zero to p_1, we have.

Factor m is called the coefficient of horizontal soil reaction which depends not only on the nature of soil but also on the size and shape of the area which carries the load.

2. Stability of well, assuming no plastic flow : Fig. 27.5 shows a well of length L and width B, acted upon by a horizontal force P per unit length of the well and a vertical load W. Let P act at a height H above the scour line, and let the depth of the well be D below the scour line. Let the well rotate at a point A situated at a depth D_1 below the scour line. The induced reactions are shown in fig 27.5 (a). Let ρ_1 = horizontal displacement of the centre line of the well at the scour line.

ρ_2 = horizontal displacement of the centre line of the well at the base level

ρ_3 = vertical downward displacement of one edge of the well at its base

ρ_3' = vertical upward displacement of the other edge of the well at its base (Assume $\rho_3 \approx \rho_3'$)

P_1 = resultant passive reaction of the well on the left face [Fig. 27.5 (b)]

P_2 = resultant vertical reaction of the well on the lower part of the right face

R = resultant vertical at the base of the well

μP_1 = skin friction on the left face of the well

μP_2 = skin friction on the right face of the well

μR = frictional resistance of the soil at its base.

From statics, the conditions of equilibrium are as follows :

$P = P_1 - P_2 - \mu R$

$PH = M_3 + M_2 - M_1 + \mu RD + \mu (P_1 - P_2) B/2$

and $W = \mu (P_1 + P_2) + R$

where M_1 = Moment at the scour line produced by P_1

M_2 = Moment at the scour line produced by P_2

M3 = Moment of the vertical soil reaction at the base

Let $\rho 4$ = Uniform vertical displacement of the well due to the resultant vertical force.

From Fig. 27.5 (a), we have following relation between the various displacements.

Design of Retaining Walls and Bulkheads

Introduction

(a) Design of Retaining Walls: Retaining walls are relatively rigid walls used for supporting the soil mass laterally so that the soil can be retained at different levels on the two sides. The lateral earth pressures acting on the retaining walls have been discussed in the preceding chapter. The types of retaining walls and their design features are explained in this chapter. However, the design is limited to the determination of the shear forces and bending moments. Actual structural design is outside the scope of this text.

(b) Bulkheads: Sheet pile walls, or bulkheads, are special type of earth retaining structures in which a continuous wall is constructed by joining sheet piles. Sheet piles are made of timber, steel or reinforced concrete and consist of special shapes which have interlocking arrangements. Sheet pile walls are used for water front structures, canal locks, coffer dams, river protection, etc. Sheet pile walls are embedded in the ground to develop passive resistance in the front to keep the wall in equilibrium. Various types of sheet pile walls and their analysis and design are discussed in this chapter.

Types of retaining walls

The most common types of retaining walls are classified as under:

(1) Gravity Retaining Walls. These walls depend upon their weight for stability. The walls are usually constructed of plain concrete or masonry. Such walls are not economical for large heights.

(2) Semi – Gravity Retaining Walls. The size of the section of a gravity retaining wall may be reduced if a small amount of reinforcement is provided near the back face. Such walls are known as semi-gravity walls.

(3) Cantilever Retaining Walls. Cantilever retaining walls are made of reinforced cement concrete. The wall consists of a thin stem and a base slab cast monolithically. Such wall is found to be economical up to a height of 6 to 8 m.

(4) Counter fort Retaining Walls. Counter fort retaining walls have thin vertical slabs. Known as counter forts, spaced across the vertical stem at regular intervals. The counter forts tie the vertical stem with the base slab. Thus the vertical stem and the base slab span between the counter forts. The purpose of providing the counter forts is to reduce the shear force and bending moments in the vertical stem and the base slab. The counter fort retaining walls are economical for a height more than 6 to 8 m.

(Note: Counter forts are on the side of the back fill).

Principles of the design of retaining walls

Before the actual design, the soil parameters that influence the earth pressure and the bearing capacity of soil must be evaluated. These include the unit weight of the soil, the angle of shearing resistance; the cohesion intercept and the angle of earth pressure have been discussed in chapter19. The bearing capacity theories are explained in chapter 23. With the earth pressure known, the retaining wall as a whole is checked for stability.

Fig. 20.2 shows a retaining wall with a smooth back face and no surcharge. The active pressure P_a acts horizontally, as shown. The front face of the wall is subjected to a passive pressure (P_p)

below the soil surface. However, it is doubtful whether the full passive resistance would develop. Moreover, often P_p is small and therefore it may be neglected. This gives more conservative design.

The weight W of the wall and the active pressure P_a have their resultant R which strikes the base at point D. There is an equal and opposite reaction R′ at the base between the wall and the foundation. For convenience, R′ is resolved into the vertical and horizontal components (R_v' and R_H'). From the equilibrium of the system,

$R_v' = W$, and $R_H' = P_a$

The third equation of equilibrium, namely the moment equation, is used to determine the eccentricity e of the force R_v' relative to the centre C of the base of the wall. Obviously, by taking moments about the toe,

Thus, eccentricity,

Where b = width of the base.

For a safe design, the following requirements must be satisfied.

(1) No Sliding

The wall must be safe against sliding. In other words,

$$\mu R_v > R_H$$

Where R_v and R_H are vertical and horizontal components of R, respectively. The factor of safety against sliding is given by

Where μ = coefficient of friction between the base of the wall and the soil (= tan)

A minimum factor of safety of 1.5 against sliding is generally recommended.

(2) No overturning

The wall must be safe against overturning about toe. The factor of safety against overturning is given by

Where ΣM_R = sum of resisting moment about toe,

and ΣM_o = sum of overturning moment about toe.

The factor of safety against overturning is usually kept between 1.5 to 2.0.

(3) No bearing capacity failure

The pressure caused by RV at the toe of the wall must not exceed the allowable bearing capacity of the soil.

The pressure distribution at the base is assumed to be linear. The maximum pressure is given by

The factor of safety against bearing failure is given by

Where q_{na} = allowable bearing pressure.

A factor of safety of 3 is usually specified, provided the settlement is also within the allowable limit.

(4) No tension

There should be no tension at the base of the wall. When the eccentricity (e) is greater than b/6, tension develops at the heel. Tension is not desirable. The tensile strength of the soil is very small and the tensile crack would develop. The effective base area is reduced. In such a case, the maximum stress is given by

Gravity retaining walls

As in design of all other structures, a trail section is first chosen and analyzed. If the stability checks yield unsatisfactory results, the section is changed, and rechecked. Fig. 20.3 shows the general proportion of a gravity retaining wall of overall height H. the top width of the stem should be at least 0.3 m for proper placement of concrete in the stem. The depth (D) of the foundation below the soil surface should be at least 0.6m. The base width of the wall is generally between 0.5 H to 0.7 H; with an average of 2H/3.

The earth pressure can be computed using either Rankine's theory or Coulomb's theory. For using Rankine's theory, a vertical line *AB* is drawn through the heel point *A*. It is assumed that the Rankine active conditions exist along the vertical line *AB*. However, the assumption for the development of Rankine's conditions along AB is theoretically justified only if the shear zone bounded by the line *AC* is not obstructed by the stem of the wall, where *AC* makes an angle η with the vertical given by

Where *i* is the angle of surcharge.

The angle α which the line *AC* makes with the horizontal is given by,

When *i* = 0, the value of α is equal to (450 + Φ′/2)

While checking the stability, the weight of soil (W_s) above the heel in the zone *ABC* should also be taken into consideration, in addition to the earth pressure (P_a) on the vertical plane AB and the weight of the wall (W_e).

Coulomb's theory can also be used for the determination of earth pressure (Fig. 20.5). as the Coulomb theory gives directly the lateral pressure on the back face (P_a), the forces to be considered are only P_a (Coulomb) and the weight of the wall (W_e). In this case, the weight of soil (Ws) is not to be considered separately.

Once the forces acting on the wall have been determined, the stability is checked using the procedure discussed in the preceding section. For convenience, the section of the retaining wall is divided into rectangles and triangles for the computation of weight and the determination of the line of action of the weights.

Semi-gravity Retaining walls. The base width of the semi-gravity retaining walls is slightly smaller than that of a corresponding gravity wall. The rest of the design procedure is the same as that for gravity retaining walls.

Cantilever retaining walls

Fig. 20.6 shows a cantilever retaining wall. The general proportions for an overall height of H are also shown. The top width of the stem is at least 0.3m. The width of the base slab is kept about 2H/3. The width of the stem at bottom, the thickness of the base slab and the length of the toe projection, each is kept about 0.1 H.

The earth pressure is computed using Rankine's theory on the vertical plane AB, provided the shear zone bounded by the line AC is not obstructed by the stem of the wall. The line AC makes an angle η with the vertical given by Eq. 20.9.

Fig. 20.7 shows the forces acting on the wall. The Rankine pressure Pa acts an angle i with the horizontal. It is resolved into the vertical and horizontal components P_v and P_h, as shown. The passive pressure Pp is also shown, but generally it is neglected. For convenience, the weight of soil (W_s) over the slab is divided into two parts (1) and (2). Likewise, the weight of stem is divided into two parts (3) and (4).

(a) Factor of safety against sliding

The factor of safety against sliding may be expressed as

Where ΣF_R = sum of the horizontal resisting forces,

and ΣF_d = sum of the horizontal driving forces.

Where b = base width, ΣV = sum of all the vertical forces, W_c, W_s and P_v.

$P_v = P_a \sin i$ and $P_h = P_a \cos i$.

P_p = passive force in the front of the wall (=$1/2\ K_{p2}\, \gamma_2\, D^2 + 2_{c2}$

Where c_2, γ_2 and Φ_2 are parameters of the foundations soil.

The factors of safety can also be determined from Eq. 20.3 if μ is given. If the required factor of safety of 1.5 against sliding is not obtained, a base key is generally provided (Fig. 20.8). The key increases the passive resistance to P_p' where

Where D_1 is the depth of the bottom o the key wall from soil surface.

Generally, the base key is constructed just below the stem and some of the main steel of the stem is extended into the key.

The friction angle Φ_2 and c_2 are generally reduced to about one-half to two-thirds of the values for extra safety, as the full passive resistance is doubtful.

Factor of safety against Overturning

Eq. 20.4 can be used to obtain the factor of safety against overturning.

Where ΣM_R = sum of the resisting moments about toe,

ΣM_o = sum of the overturning moments about toe.

The only overturning force is Ph, acting at a height of H/3.

$M_o = P_h \times H/3$

The resisting moments (M_R) are due to weights W_1, W_2, W_3, W_4 *and* W_5 of the soil and the concrete. The vertical component of pressure P_v also helps in resisting moment. Its resisting moment is given by

$M_v = P_v \times b$

Therefore

Where $M_1, M_2, \ldots M_5$ are the moments due to $W_1, W_2 \ldots W_5$ about toe.

Factor of safety against bearing capacity failure

The sum of the vertical forces acting on the base is equal to ΣV. The horizontal force is P_h. The resultant force (R) is given by

The net moment of these forces about toe *B* is given by

$\Sigma M = \Sigma M_R - \Sigma M_O$

The distance of the point E, from the toe, where R strikes the base is given by

Hence, the eccentricity e of R is given by

$e = b/2 -$

If e > b/6, the section should be changed, as it indicates tension. The pressure distribution under the base slab is determined as

The factor of safety against bearing capacity failure is given by Eq. 20.7.

COUNTERFORT RETAINING WALLS

For counter fort retaining walls, the general proportions of the stem and the base slab are almost the same as that in the cantilever walls. The counterforts are about 0.3 m thick and have the centre-to-centre spacing of 0.3 H to 0.7. H.

The analysis is also similar to that of a cantilever retaining wall. The pressure p_{max} and p_{min} are determined, as in the case of cantilever walls.

The basic difference between the counter fort retaining wall and the cantilever retaining wall is in the determination of the bending moment and shear forces.

(1) In cantilever retaining walls, the stem acts as a vertical cantilever fixed at base whereas in the counter fort retaining walls, it acts, as a continuous slab supported between the counter forts. The slab has positive moments in the middle and the negative moments at the supports. The reinforcement is provided in the horizontal direction on the front side of the stem in the middle and on the rear side at the supports. In cantilever walls, the main reinforcement is in the vertical direction at the rear face.

(2) In cantilever walls, the toe slab and the heel slab both act as cantilevers subjected to the upward pressure. The reinforcement is provided at the bottom face.

In counter fort retaining walls, although the toe slab acts as a cantilever, the heel slab acts as a continuous slab supported on the counter forts. Te main reinforcement is at the top face in the middle portion and at the bottom face near the supports.

(3) In counter fort retaining walls, the counter forts are designed as cantilever of varying section and fixed at the base. The main reinforcement is provided at the back face of the counter fort.

In addition, the vertical and horizontal ties are provided in the counter forts to join the base and the stem to the counter forts.

The structural design of the counter fort and cantilever retaining walls is outside the scope of this text.

Other modes of failure of retaining walls

In addition to the three types of failures, viz, sliding, overturning and bearing failures, a retaining wall may fail in the following two modes if the soil below is weak.

(1) Shallow Shear Failure. This type of failure occurs along a cylindrical surface ABC passing through the heel of the retaining wall. The failure takes place because of excessive shear stresses along the cylindrical surface within the soil mass. However, it has generally been found that the factor of safety against horizontal sliding discussed in Sect. 20.3 is lower than that for the shallow shear failure. Consequently, if the factor of safety against sliding (F_s) is greater than about 1.5, shallow shear failure is not likely to occur.

(2) Deep Shear Failure. This type of slope failure occurs along a cylindrical surface ABC when there is a weak layer of soil underneath the wall a depth of about 1.5times the height of the wall. The critical failure surface is determined by trial and error procedure.

For the backfills having slope i less than 10^0, it has been found that the critical failure surface *DEF* passes through the edge of the heel slab. The minimum factor of safety is found by trial and error, taking different circles, and determining the resisting forces and the driving forces along the failure surface.

When a weak soil layer is located at a shallow depth below the retaining wall, the possibility of deep shear failure should be investigated. The possibility of excessive settlement should also be looked into. Sometimes, piles are used to transmit the foundation load to a firm layer below the weak layer. However, care shall be taken in the design of piles so that the thrust of the sliding wedge of soil does not cause bending of the piles.

DRAINAGE OF THE BACKFILL

When the backfill becomes wet due to rainfall or any other reason, its unit weight increases. It increases the pressure on the wall and may create unstable conditions. Further, if the water table also rises, the pore water pressure *(u)* develops and it causes excessive hydrostatic pressure on the wall. To reduce the development of excessive lateral pressures on the wall, adequate drainage must be provided.

Weep holes are generally provided in the walls. The weep holes are of about 0.1 m diameter. The spacings of the holes generally varies between 1.5 m to 3 m in the horizontal direction. As the backfill material may be washed into weep holes and may clog them, filter material is placed around the weep holes.

Perforated pipes are also frequently used for the drainage of the backfill. These pipes are laid near the base. The water is collected from the backfill and discharged at a suitable place at the ends. The filter material is placed around the pipes. These days, a filter cloth or a geotextile fabric is also used to serve the purpose of a filter material. All drain pipes should be provided with clean-outs for cleaning when clogged.

Fine-grained soils cause large earth pressure against retaining walls and re, therefore, rarely used as a backfill material. As far as possible, good draining, granular materials should be used, at least in the sliding wedge portion of the wall. In case a fine-grained material cannot be avoided, some form of filter of coarse permeable material is placed behind the retaining walls to prevent the development of excessive pore water pressure. Fig. 20.13 shows two types of drainage filters commonly used. The water percolating into the filter is discharged through the weep holes. The inclined filter is found to be more effective than the vertical filter.

BULK HEADS

TYPES OF SHEET PILE WALLS

Sheet piles are generally made of steel or timber. However, sometimes reinforced cement concrete sheet piles are also used. The use of timber piles is generally limited to temporary structures in which the depth of driving does not exceed 3m. For permanent structures and for depth of driving greater than 3m, steel piles are more suitable. Moreover, steel sheet are relatively water tight and can be extracted if required and re-used. However, the cost of steel sheet piles is generally more than that of timber piles. Reinforced cement concrete piles are generally used when these are to be jetted into fine sand or driven in very soft soils, such as peat. For tougher soils, the concrete piles generally break off.

Fig. 20.14 shows the plan of a typical steel sheet pile wall, in which 2 sheet piles are shown with joints.

Based on its structural from and loading system, sheet pile walls can be classified into 2 types : (1) Cantilever Sheet piles and (2) Anchored Sheet Piles.

(1) Cantilever Sheet Piles

Cantilever sheet piles are further divided into 2 types :

(a) Free cantilevers sheet pile. [Fig. 20.15 (a)]. It is a sheet pile subjected to a concentrated horizontal load at its top. There is no backfill above the dredge level. The free cantilever sheet pile derives its stability entirely from the lateral passive resistance of the soil below the dredge level into which it is driven.

(b) Cantilever sheet pile. [Fig. 20.15 (b)]. A cantilever sheet pile retains backfill at a higher level on one side. The stability is entirely from the lateral passive resistance of the soil into which the sheet pile is driven, like that of a free cantilever sheet pile.

(2) Anchored Sheet piles

Anchored sheet piles are held above the driven depth by anchors provided at a suitable level [Fig. 20.15 (c)]. The anchors provide forces for the stability of the sheet pile, in addition to the lateral passive resistance of the soil into which the sheet piles are driven. The anchored sheet piles are also of two types:

(a) Free-earth support piles. An anchored sheet pile is said to have free-earth support when the depth of embedment is small and the pile rotates at its bottom tip. Thus there is no point of contra flexure (or inflexion point) in the pile.

(b) Fixed-earth support piles. An anchored sheet pile has fixed earth support when the depth of embedment is large. The bottom tip of the pile is fixed against rotations. There is a change in the curvature of the pile, and hence, an inflexion point occurs.

FREE CANTILEVER SHEET PILE

The free cantilever sheet pile about a point *O* below the dredge level. The actual pressure distribution is shown in Fig. 20.16(a). Bulm (1931) gave a simple solution. The passive resistance of the soil on the left side is idealized as a right angled triangle AOE [Fig. 20.16(b)]. The disturbed pressure acting on the right side below the pivot *O* is replaced by an equivalent concentrated load P1 acting at point *O*. In calculations that follow, however, the magnitude of the force P1 is not required.

For equilibrium, the moment of all the forces about *O* must be zero, *i.e*.

Where F is the horizontal force, *h* is the height of wall above the dredge level,

d is the depth of embedment.

Eq. 20.20 can be solved for *d*. The actual depth to be provided is generally taken as 1.2 d.

The point of the maximum bending in the sheet pile can be determined as under.

The bending moment at depth x below the dredge level is given by

The maximum B.M. (M_{max}) is obtained by substituting the value of x from Eq. 20.22 into Eq. 20.21.

The section modulus of the sheet pile can then be determined as

Where σ_a = allowable bending stress in the pile. Fig. 20.16 (c) shows the bending moment diagram.

CANTILEVER SHEET PILE IN COHESIONLESS SOILS

(a) shows a cantilever sheet pile in a cohesionless soil deposit. The pile rotates about the point O′. The pressure above O′ is passive in the front and active on the back side. However, the pressures below the point O′ are reversed i.e. there is active pressure in the front and passive on the back side. Fig. 20.17 (b) shows the actual pressure distribution. As the analysis taking actual pressure distribution is quite complicated, the pressure distribution is generally simplified as shown in Fig. 20.18. In Fig. 20.18, the pressure is zero at point O_1 at a depth ***a*** below the dredge level.

The pressure diagram BCO_1 shows the active pressure. The pressure at the dredge level is given by

The depth a of point O_1 of zero pressure is given by

$$P_1 = \gamma a\,(K_p - K_a) = 0$$

Let the total active pressure above point O_1 be P_1 acting at a height of above O_1.

The passive pressure is given by the diagram O_1 EO. The passive pressure intensity at the bottom tip A can be expressed as

$$P_2 = \gamma(K_p - K_a)\,(d - a) = \gamma(K_p - K_a)\,b$$

Where b = d-a, in which d is the depth of point A below the dredge level.

The passive pressure is indicated by the diagram OAF on the back side. The intensity of pressure at the tip A is given by

$$P_3 = \gamma(h + d)\,K_p - \gamma d K_a$$

or

$$P_3 = \gamma(h + b + a)\,K_p - \gamma\,(b+a)K_a$$

From the equation of equilibrium in the horizontal direction,

$$P_1 + P_3 - p_2 = 0$$

The total pressure P_3 and P_2 can be expressed in items of p_3 and p_2 as follows :

Eq.20.28 is solved by trail and error to determine b. The value of d is equal to (b+a). The depth d is for a factor of safety of unity. The required depth (D) is usually taken as 1.2d to 1.4d.Thus

D = 1.2d to 1.4d

This gives a factor of safety of about 1.50 to 2.0

Alternatively, a factor of safety can be applied to the passive resistance. In that case ,the value of k_p is usually taken as ½ to 2/3 of the normal value while computing b from eq.20.28 and the required depth D is taken equal to d.

In the above discussions, the depth of water table is not considered. If the water table on the front side is at the same level as on the rear side, the analysis remains unaltered except that the submerged unit weight (γ') should be used for the soil below the water table. However, if the difference in the two levels is greater than 1m, the pressure due to water on the sheet pile should be found from the flow net and properly accounted for in the analysis.

Approximate Analysis. The exact analysis of the cantilever sheet pile as discussed above is quite involved. An approximate value of d can be obtained using a simplified pressure diagram as shown in Fig. 20.20. In this analysis, the resistance of the pile below the point *O* is replaced by a concentrated force P3. (Note that the pressure distribution extends up to tip A).

From the equilibrium in horizontal direction.

$$P_1 - P_2 + P_3 = 0$$

Eq. 20.31 is solved by trial and error for d.

The value of d so obtained is usually increased by 20 to 40%. Thus

$$D = 1.2\ d \text{ to } 1.4\ d.$$

CANTILEVER SHEET PILE PENETRATING CLAY

A cantilever sheet pile penetrating clay ($\Phi = 0$) below the dredge level. The backfill is of cohesionless soil (c=0). Let the bulk unit weight of the backfill material and clay be, respectively, γ_1 and γ. The cohesion intercept of clay is c.

The pressure p1 at the dredge line on the back side is given by

$$P_1 = \gamma_1 h K_{a1}$$

Below the dredge level but above the point of rotation O, the passive pressure acts from left to right and the active pressure acts from right to left. Therefore, the pressure at depth Z below the dredge level is given by

$$P_2 = P_p - P_a$$

Eq. 20.35 can be solved for d. The actual depth D is kept 40% to 60% more. Thus

$$D = 1.4\ d \text{ to } 1.6\ d$$

Alternatively, the depth d can be computed using a reduced value of c/2 or 2c/3 in Eq. 20.35. In this case, the actual depth D would be equal to the computed value of d, as the factor of safety has already been applied to c.

If the water table exists on both the sides, modification can be done as in the case of cohesionless deposits. The submerged weights are used below the water table (see Illustrative Example 20.8).

ANCHORED SHEET PILE WITH FREE-EARTH SUPPORT

The stability of anchored sheet pile depends upon the anchor force in addition to that upon the passive earth pressure. The embedment depth is considerably smaller than that in a cantilever sheet pile. Therefore, by this method, the total length of the sheet pile is reduced. Of course, the additional cost of anchors is also to be considered while judging the economy of the two types construction.

Fig. 20.22 (a) shows an anchored sheet pile with free earth support. The deflected shape is also shown. As already mentioned, there is no point of contra flexure below the dredge level. Thus, below the dredge level, no pivot point exists for the statical system. The statical analysis is based on the assumption that the soil into which the pile is driven does not produce effective restraint to induce negative bending moment at its support.

The equations for the depth d are derived separately for the cohesionless and cohesive soils.

(a) Cohesionless Soils.

Fig. 20.22 (a) shows the forces acting on the pile, assuming that the material above and below the dredge level is cohesionless.

From equilibrium, $T+P_2-P_1=0$

Where T is the tensile force in anchor.

The depth a to the point of zero pressure can be determined as under.

$$\gamma K_a\ (h=a) - \gamma K_p\ a = 0$$

$$or\ \gamma K_a\ (h=a) - \gamma K_p\ a = 0$$

(b) Cohesive Soils.

Let us now consider the case when the anchored sheet pile is driven in clay (Φ = 0), but has the backfill of cohesionless, granular material (Fig. 20.23). The pressure is given by

Eq. 20.42 can be solved for d. The actual depth (D) provided is 20 to 40% more than D.

It may be noted that the wall becomes unstable when $P_2 = 0$ *i.e.,* $4c - \gamma H = 0$

The left hand side is equal to the stability number (S_n) defined in chapter 18. In other words, the walls become unstable when S_n is equal to or less than 0.25. If the adhesion of clay with the sheet pile (C_a) is considered, Eq. 20.43 is modified as

Therefore, the minimum stability number (S_n) required is 0.31. If the factor of safety required is F, the stability number *(S_n)* should be equal to 0.31 *F* or more.

Rowe's moment reduction curves

As sheet piles are relatively flexible, these deflect considerably. Their flexibility causes a redistribution of lateral earth pressure. The net effect is than the maximum bending moment is considerably reduced below the value obtained for the free-earth supports discussed in the preceding section.

Rowe (1952) developed a theoretical relation between the maximum bending moment and the flexibility of the sheet pile and gave moment reduction curves. The relative flexibility (ρ) is defined as

Where h = retained height (m), D = actual driving depth (m),

E = Young's modulus of the pile material (MN/m2) and I = moment of inertia of the pile (M4/m),

H = total length of the pile.

For anchored sheet piles in cohesionless soils, the relative density is important. The relative depth of anchor factor, β = e/H is also relevant.

For anchored sheet piles in cohesive soils, the stability number (Sn), as given below, is also required.

The relative height of piling factor α = h/H is also important for cohesive soils.

Fig. 20.24 shows a typical moment reduction curves for cohesionless soils. The ratio M_d/M_{max} is determined directly for the given value of ρ. The curve (a) is for loose sand (relative density = 0) and the curve (b) for dense sand (relative density = 100%). The value of M_{max} being known from the free-earth support analysis, the design moment M_d can be computed.

(For more details, the original paper may be consulted).

ANCHORED SHEET PILE WITH FIXED-EARTH SUPPORT

Fig. 20.25 (a) shows the deflected shape of an anchored sheet pile with fixed-earth support. The elastic line changes its curvature at the inflexion point I. the soil into which the sheet is driven exerts a large restraint on the lower part of the pile and causes a change incurvature. Fig. 20.25 (b) shows the pressure distribution. Bulm (1931) gave a mathematical relationship between (i/h) and Φ (Fig. 20.26), where I is the depth of the point of inflexion I below the dredge level and h is the height of sheet pile above the dredge level. Thus inflection point I is located.

For simplicity, the lower portion of the pressure diagram on the right hand side in Fig. 20.25 (b) is replaced by a concentrated force R_k at point K and the diagram shown in Fig. 20.27 (a) is used in the analysis. The magnitude of R_k is initially unknown, but it is automatically excluded from calculations when the moments are taken about *K*. Once the depth has been found, Rk can be determined from the equilibrium equation in the horizontal direction.

As the exact analysis of the anchored sheet pile with fixed-earth support is complicated, an approximate method, known as *equivalent-beam method* is generally used. It is assumed that the sheet pile is a beam which is simply supported at the anchor point *M* and fixed at the lower end K. Fig. 20.27 (b) shows the bending moment diagram the bending moment is zero at the inflexion point I. Theoretically, lower part *IK* of the pile can be removed and the sheer force can be replaced by a reaction R_I. Thus, a simply-supported beam *BI* is obtained [Fig. 20.27 (c)].

The following procedure is used for the analysis of the sheet pile with fixed-earth support, using equivalent beam method.

(a) Upper Beam BI

(1) Determine the pressure *P1* at the dredge level.

(2) Estimate the angle of shearing resistance Φ′ of the soil.

(3) Determine the distance I of the point of inflexion from Fig. 20.26.

(4) Determine the distance a of the point of zero pressure from the equation,

(5) Determine the pressure p_o at the point of inflexion from the relation,

(6) Determine the reaction R_1 for the beam *IB* by taking moments about the point *M* of anchor of all the forces acting on *IB* [Fig. 20.28 (a)].

(b) Lower Beam IK

(7) Determine the pressure P2 from the relation

$$P2 = \gamma(K_p-K_a)\ (d-a)$$

Alternatively,

(8) Determine the distance (d-a) by taking moments of the forces on the beam IK about K [Fig. 20.28 (b)]. The reaction *RI* on the lower beam is equal and opposite to that on the upper beam.

(9) Calculate d from Eq. 20.49 and hence find D = 1.2 d.

(10) Determine the tension T in anchor by considering the equilibrium of beam IB. Thus

$$T = P_1 - R_I$$

Where P1 = total force due to pressure on IB.

DESIGN OF ANCHORS

The anchors used in sheet pile walls are of the following types:

(1) Anchor plates and Beams (also, known as dead man).

(2) Tie backs.

(3) Vertical Anchors piles.

(4) Anchor beams supported by batter piles.

The design of anchor plates and beams is discussed below.

Anchor plates and beams are made of cast-concrete blocks. A wale (horizontal beams) is placed at the front (or back) face of the sheet pile and a tie rod is attached to it. The other end of the tie rod is connected to an anchor plate or a beam.

The resistance offered by an anchor plate or a beam is derived from the passive resistance of the soil in front of the plate. For full passive resistance to develop, the anchor plate must be located in zone CDE. Teng (1962) gave the following equations for the ultimate resistance of anchor plates in granular soils located at or near the ground surface.

Let B be the length of the anchor perpendicular so the cross section and let h be the height of the anchor. (a) For continuous plates or beams with B/h 5, the ultimate resistance is given by

$$P_u = B(P_p - P_a)$$

Or

Where is the depth of the lower face of the anchor beam from the ground surface.

(b) For plates or beams with B/h < 5, the ultimate resistance is given by

Where K_o = coefficient of earth at rest) = 0.40).

The allowable resistance is taken as

Where FS = factor of safety (generally taken equal to 2.0).

The center-to-center spacing of anchors is obtained from the relation,

$S = Pa / T$

Where T = tension in sheet pile per unit length as obtained from the analysis of anchored sheet pile.