Expansive soils are one of the most problematic soils, from the civil engineering viewpoint. The problem with them is their volume change behavior. They increase in volume (swell) when water is imbibed by them and decrease in volume (shrink), when water gets removed from it, by evaporation and evapotranspiration. This alternate swelling and shrinking occur in monsoon and summer seasons respectively. Normally, structures are not designed to undergo large displacements. Structures, especially light structures, founded on desiccated expansive soils suffer severe distress consequent to alternate swelling and shrinkage. R. Earl Jones and Wesley G Holtz (1973) had estimated that annual losses in the US from floods, earthquakes, hurricanes and tornados are less than one-half of the damages from expansive soils.

Expansive soils are available in various parts of the world mostly in arid and semi-arid climatic regions. It occupies about 20 percent of the land area in India. They are called as Pot clays in South Africa, Bay of Biscay soils in Australia, Expansive clay soils in South Western United states, and Black Cotton soils in Burma and India.
‘Regurs’ in India come under this category, and occur in the Deccan Plateau of Maharashtra, parts of Andhra Pradesh, Tamilnadu, Karnataka, Southern parts of Gujarat, and parts of Madhya Pradesh and Uttar Pradesh.

Various factors influencing volume change behaviour of clays can be summarized as

1. Mineral type and proportion of the clay mineral in the given soil
2. Placement or in situ conditions-dry density (soil structure) and water content (suction)
3. Thickness and depth of the expansive soil layer
4. Sources of moisture variation - e.g., climate, position and variation of water table, vegetation, drainage leaks
5. In-situ conditions and

These swell-shrink cycles due to moisture fluctuations with changing seasons result in polygonal cracks near the surface in the expansive soils. The depth of crack indicates the depth of the active zone, which is defined as the thickness in which moisture deficiency exists (Snethen, 1980). This is also the depth up to which the volume changes occur. In India, the active zone is mainly confined to a depth of about 3.5 m below ground level (Mohan, 1977), though depths of more than 3.5 m were also reported in certain areas (Ramaswamy, 1990). It was observed by Osman and Sharif (1987) that shrinkage cracks were found to greatly reduce heave in cracked zone and hence reduce the accumulated surface heave. Ground heave was found to decrease exponentially with depth.

The main periods during which cracks start developing are (a) at the end of rainy season due to swelling and (b) at the peak of the dry season due to
shrinkage. Cracking in building is most noticeable at the end of the long dry period following a wet season.

**2.2 FAILURES IN STRUCTURES**

1. The cracks developed in any building as a result of swelling and shrinkage stresses from the soil may be grouped into the following six categories:
   1) Diagonal and vertical cracks in the interior and exterior walls oriented towards the short span direction of roof slab
   2) Horizontal cracks in the interior and exterior walls
   3) Longitudinal cracks in roof slab due to the cantilever action
   4) Separation of roof slab from interior walls
   5) Leaning out of exterior walls
   6) Hogging moments are caused by moisture migration from high temperature zones at the edges to the low temperature zones beneath the shaded area of the building

2. Utilities buried in the soil such as water pipes and sewage lines get damaged due to displacement in the soil in which they are buried. The ensuing leakage of their contents would result in further wetting of the soil and enhance the chances of swelling.

3. Canal lining and canal beds constructed in expansive soils heave and as a result cracks are formed and seepage losses will occur.

4. Cracking in retaining walls and abutments of bridges occur if proper facilities like weep holes are not provided in the retaining walls. Expansive backfills will exert large swelling pressure on the back of the retaining wall and abutment of bridges and thus cause heavy damage.
5. Cracking in earth dams at the interface of shoulder and the central clay core occur if an expansive core is provided to minimise seepage through the body of the dam to bring the phreatic line into the downstream horizontal filter. If the soil below a pavement has high shrinkage and swelling properties, it creates problems in the maintenance of highways and runways.

2.3 IDENTIFICATION OF EXPANSIVE SOILS

Expansive soils can be identified by
- mineralogical composition
- knowledge of index properties
- direct measurement of swelling characteristics

The presence of mineral Montmorillonite is the main cause of high swelling, though other minerals like Illite also contribute to swelling, but in a small measure. The presence of clay minerals can be investigated by different techniques like X-ray diffraction (XRD), Differential Thermal Analysis (DTA), Dye adsorption, and Chemical analysis and Electron Microscope resolution (EMR).

2.3.1 IDENTIFICATION BASED ON INDEX PROPERTIES

Skempton (1953) used the term activity to indicate the volume change potential of a soil. It is defined as the ratio of plasticity index of the soil to the percentage of the soil fraction finer than $2\mu$ (0.002mm). This definition has been later modified by Seed et al (1962) for natural soils as

$$\text{Activity} = \frac{\text{Plasticity Index}}{\left(\% \text{clay fraction finer than } 2\mu\right) - 5\%} \quad (2.1)$$

An activity value of less than 0.75 suggests low volume change potential, whereas a value of more than 1.25 indicates high volume change potential. Values lying in between suggest medium potential (Skempton, 1953)

The index properties can be taken only as a guide to distinguish an expansive soil from a non-expansive soil. It is a better option to determine the free swell capacity as a means of classifying the expansive soil.

One important parameter ‘Free Swell’ for determining the potential to swell was proposed by Holtz and Gibbs(1956), defined as the ratio of the change in volume of 10 ml of dry soil placed in 100 ml of water, to its initial volume, expressed as a percentage after complete swelling. Soils with Free swell of 100% or more were graded as those that cause severe damage to lightly loaded structures. However, this test was found to be crude and subject to many limitations like personal factors etc. A more convenient method has been later suggested by Mohan and Goel(1959). In this test, the volumes of 10g of oven-dried sample were measured in 100cc of water and kerosene separately. The Differential Free Swell, which was later re-designated as Free Swell Index (FSI), is defined as

$$\text{FSI} = \frac{\text{Final vol. of soil in water} - \text{final vol. of soil in kerosene}}{\text{final volume of soil in kerosene}} \quad (2.2)$$
The above test was standardised and adopted by the Bureau of Indian Standards in IS: 2720 (part XL) – 1977. Documented values of FSI and corresponding swell potential are adopted by IS: 1498-1970 (revised Ed.1982). Table 2.1 gives the degree of expansion based on the Free Swell Index value.

Table 2.1: Degree of expansion based on Free Swell index (BIS:1498-1970)

<table>
<thead>
<tr>
<th>Liquid Limit (%)</th>
<th>Ip</th>
<th>Shrinkage Index</th>
<th>Free Swell Index (%)</th>
<th>Degree of expansion</th>
<th>Degree of Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>70-90</td>
<td>&gt;32</td>
<td>&gt;60</td>
<td>&gt;200</td>
<td>Very High</td>
<td>Severe</td>
</tr>
<tr>
<td>50-70</td>
<td>23-32</td>
<td>30-60</td>
<td>100-200</td>
<td>High</td>
<td>Critical</td>
</tr>
<tr>
<td>35-50</td>
<td>12-23</td>
<td>15-30</td>
<td>50-100</td>
<td>Medium</td>
<td>Marginal</td>
</tr>
<tr>
<td>20-35</td>
<td>&lt;12</td>
<td>&lt;15</td>
<td>&lt;50</td>
<td>Low</td>
<td>Non-Critical</td>
</tr>
</tbody>
</table>

A method of identification of the clay mineral present in a soil is by using the plasticity chart, used in the classification of fine-grained soils (IS:1498-1970), with the superimposition of U-line (Casagrande's personal correspondence, Holtz and Kovacs,1981).

Fig. 2.1. Plasticity chart
The A-line drawn with equation \( I_p = 0.73 (W_L - 20) \), differentiates clays and silts. Another line, called the U-line with equation \( I_p = 0.9 (W_L - 8) \) gives an idea of the minerals present in the soil. Montmorillonite which is a high swelling mineral lies in a band immediately below the U-line and Illite lies between the U-line and the A-line, whereas Kaolinite which is a non-swelling mineral lies below the A-line. (Fig. 2.1).

2.3.2 IDENTIFICATION BASED ON SWELLING CHARACTERISTICS

Direct measurement of the swelling characteristics, namely, the swelling potential and the swell pressure, give an idea of the degree of probable expansion of a swelling soil.

Swelling Potential

The ratio of increase in thickness to the original thickness of a soil sample placed in a consolidation ring, compacted at its Proctor's maximum dry density and optimum moisture content, when soaked in water under a surcharge of 6.9 kPa (1 psi) is called the swelling potential (S%). This is expressed as a percentage.

Swell Pressure

As the soil swells, if the structure also moves correspondingly, there would not be any pressure on the structure. But if the swelling is to be arrested, a certain amount of pressure needs to be applied. This pressure is termed as the swell pressure \( (p_s) \). There are at least three definitions for the swelling pressure (Jennings, 1963). Swell Pressure may be defined as that pressure developed by dead load, for which there will be neither compression nor expansion of the sample on saturation.
- which must be developed to keep the volume of the soil constant when free water is supplied
- which must be applied to compress fully a swollen sample to its original void ratio.

For definition (1) – a series of tests are to be conducted on identical samples with exactly the same initial conditions, while the tests corresponding to definition (2), and (3) require testing of only one sample.

2.4 EXISTING FOUNDATION PRACTICES IN EXPANSIVE SOILS

While shear strength and settlement are the prime criteria in the design of foundations in ordinary soils, which are in-expansive, swelling pressure and heave, which is vertical swelling, are the main governing criteria in the design of foundations in expansive soils.

A safe foundation practice shall be that which minimises the effect of heave in expansive soil, though not totally curtail it. Some popular methods may be

a) To avoid expansive soil
b) Changing the nature of expansive soil
c) Alteration of properties by mechanical and chemical means
d) To adopt special foundation techniques
   - Designing the structure to remain undamaged in spite of swelling
   - Isolating the structure from the swelling ground.

2.4.1 AVOID EXPANSIVE MATERIAL

In an expanding urbanisation, this may not be an economically viable proposition. Development of technology helps to effectively deal with the problems posed by expansive soils, and hence this option is seldom used now a day.
2.4.2 CHANGING THE NATURE OF EXPANSIVE SOIL

2.4.2.1 COMPACIlON

Heave is found to decrease if the soil is compacted to a lower unit weight on the wet side of the optimum moisture content.

2.4.2.2 PRE-WETTING

The moisture content of the soil is increased by ponding to achieve most of the heave before the construction. But it is a time-consuming process.

2.4.3 ALTERATION OF PROPERTIES

The ground conditions may not always permit direct resting of the foundation on the soil. It may be necessary to ‘doctor’ the ground to improve its engineering performance before the foundation is laid.

2.4.3.1 MECHANICAL ALTERATION

When the depth of the expansive soil layer is small, it can be excavated and replaced with a suitable material, provided the latter is available as required.

Two popular methods are referred to below.

Sand Cushion Method: In this method (Satyanarayana, 1966), the entire depth of the expansive clay stratum, if it is thin, or, a part thereof, if it is deep, is removed and

![Fig 2.2: Sand Cushion Method](image-url)
replaced by sand cushion compacted to the desired density and thickness. Swell pressure varies inversely as the thickness of sand layer and directly as its density. Hence sand cushions are formed in their loosest state without violating the criteria of bearing capacity. In monsoon, the saturated sand occupies less volume, accommodating some of the heave of the underlying soil and, in summer, partially saturated sand bulks and occupies the extra space left by the shrinkage of the expansive soil.

The limitations of this method are the high permeability of sand and design of thickness of sand cushion. The former creates accumulation of water and the latter depends on the depth of active zone which itself is difficult to determine. Besides, effects of fatigue in swelling have to be considered in designing the thickness of sand cushion.

Cohesive Non-Swelling (CNS) layer method: According to Katti(1978), cohesive forces develop up to a depth of 1.0m to 1.2m with saturation of expansive soil, which help to counter heave in the soil beneath eventhough the soil within the zone itself swells. The surface electric charges of clay particles produces adsorbed water bonds and develops cohesion, resulting in creation of an effective overburden pressure that
is about 10 – 18 times the physical overburden pressure.

In this method, the top 1.0m – 1.2m of expansive soil is removed and replaced by cohesive non-swelling soil (CNS) layer, which is later saturated. A CNS layer creates an environment similar to that which prevails within a depth of 1 m in an expansive soil with equivalent cohesion to counter heave. Moorum is a good example of CNS material.

Katti’s specifications for a CNS material are hard to meet. Besides, the cohesive soil upon saturation is rendered soft and may cause failure of the footing. So, Katti (1996) recommended the use of a Mechanically Stabilized Mix (MSM) to be placed on the top of the CNS cushion to make the soil strong enough to bear the load.

However, there is no evidence to show the use of CNS for any building foundation, even though it is claimed to have been used with success in solving problems of heaving of canal beds and linings.

2.4.3.2 PHYSICAL ALTERATION:

A granular material like cohesionless soil or rock flour is mixed with expansive soil to minimise heave. The faster ingress of water due to increased permeability is a disadvantage of this method.

2.4.3.3 CHEMICAL ALTERATION

In this method, the properties of expansive soil are altered to reduce the heave by adding chemicals. Lime was found to be the most effective in terms of efficacy and economy. By adding lime, the liquid limit and plasticity index are reduced, making the soil more friable and less susceptible to volume change. Other methods include alteration with cement, bitumen, asphalt, flyash, etc. These methods
have been proved to be costly and practically difficult, and standardisation of procedures could not be made. Hence they are not popular.

2.4.4 REINFORCED SOIL COLUMNS

2.4.4.1 LIME-SOIL COLUMNS

These two types of foundations serve both as a drain and as reinforcement in the soil. In this practice, which is very rare in India, columns of 0.5m diameter are formed by using quick (unslaked) lime mixed with *in-situ* clay using a tool. The tool is pushed to the desired depth, and rotated to and fro, simultaneously adding lime through the arrangement provided in the tool. Clay clods are formed which act as a column of high strength and good permeability to act as drain. About 10% - 12% of lime by dry weight of the clay is required.

Srirama Rao (1984) tried lime stabilization methods using lime and lime-soil columns. Only slaked lime is used in the stabilising the soil. Venkata Ratnam et al (1985), Babu Shankar et al (1989), reported that diffusion of lime is effective up to a radial distance of about 3 times the diameter of the lime-soil column. The limitation is that only the top few layers of soil could be modified.

2.4.5 ADOPTING SPECIAL CONSTRUCTION PRACTICES

Though the properties of expansive soils are unaltered, foundation techniques have been developed to counter the effect of heave in expansive soils as explained below:

2.4.5.1 RIGID CONSTRUCTION

The structure in this case is built on a rigid raft and, as a result, differential movement of foundations are not transmitted to the structure. The stiffened slabs on the ground can handle edge lift or central lift easily. Various
materials and construction techniques can be employed for the construction of this type of foundations.

**Stiffened Mat**, referred to as structural slab-on-ground or reinforced and stiffened slab is a slab which receives and transmits all the structural loads to the soils beneath. The slab is designed to resist both the positive and negative moments. Positive moments occur due to dead load and the swelling of the foundation soils induces the live load pressures exerted on the slab with negative moments. Generally,

![Fig.2.4. Stiffened Mat](image)

it is the negative moment consideration that controls the design of the mat foundation. If all structural elements are based on the stiffened slab, the slab movement will not affect the stability of the structure. Even if there is tilting, it does not affect the performance of the structure.

**Waffle raft** (Fig.2.5) are another example of this type of construction. The ribs hold the structural load. The waffle voids allow the expansion of the soil.

![Fig.2.5: Waffle slab](image)
Boucill Raft (Williams et al 1991) is an improved cellular raft-like structure, which resembles a traditional raft with another slab at the bottom, connected by brick webs. This foundation is several times stiffer, particularly in torsion, than any other types of raft with stiffening beams because of the composite action of top and bottom slabs with the integral webs. The differential movements are not transmitted to the structure and the damage is minimised. The walls of the building can be designed as deep beams adding to the rigidity of the structure. This is being mostly used in South Africa.

2.4.5.2 FLEXIBLE CONSTRUCTION

Studded brick strip footing is a type of foundation in which the structure is flexible enough to tolerate the differential settlements (Bhandari et al, 19??). In this, volume changes are permitted in expansive soil through the open voids provided for the purpose expressly in the studded brick foundation. A studded brick is of a special shape measuring 304mm × 100mm × 80mm, with a burnt size of 280mm × 90mm × 75mm. No binding material is required for this construction. However, the heave that can be tolerated by this foundation is limited.
2.4.5.3 PAD FOUNDATIONS

They are a series of individual footing pads placed on the upper soil and spanned by grade beams (Chen, 1975). They are used with advantage where bedrock is deep, use of friction piles is unfeasible and upper soil possesses moderate swell potential but high bearing capacity.

2.4.5.4 FRICTION PIERS

Where the bedrock is very deep and the upper layers of the soil are expansive, friction piers can be used (Chen, 1975). But it can be used with advantage only if soft soil is not encountered within the length of the pier.

2.4.6 ISOLATING STRUCTURE FROM SWELLING SOIL

2.4.6.1 DRILLED PIERS

Piers of small diameter but of considerable length are drilled (Fig 2.8) into the expansive soils up to the zone unaffected by moisture change (Chen, 1975). Both the end-bearing resistance and skin friction resistance increase with depth at the rate of about 10% per meter.

2.4.6.2 BELLED PIERS

These piers are drilled into materials other than bedrock and are enlarged at the bottom of the hole for the purpose of increasing the bearing area, thus increasing the total load carrying capacity (Fig 2.9). Such piers are commonly called as Belled-piers (Chen, 1975)
Foundation piers with a suspended floor slab for the construction of the structure, independent of soil movements, are a case in point. The piers are anchored below the active zone. An air gap of at least 150mm is maintained between the grade beam and the foundation soil to prevent swell pressure being exerted on the beam.

The uplift force, which tends to pull the pier out of the ground, is a direct function of the swell pressure. The withholding force consists of the dead load on the pier and the skin resistance along the unwetted portion of the pier. Thus, in the figure, the total uplift force $U$ is given by

$$U = 2\pi r \alpha \sigma_s (L - d_l)$$  \hspace{1cm} (2.3)

where, $r = $ Radius of the pier
$d_l = $ Depth of soil unaffected by wetting
$L = $ total length of the pier
$\sigma_s = $ Swell Pressure
$\alpha = $ Adhesion factor

The withholding force $F_w$ that keeps the pier from being pulled out of the ground is

$$F_w = (\pi r^2 p) + (2\pi r f d) + (F_w)$$  \hspace{1cm} (2.4)

where, $p = $ Unit dead load pressure
$f = $ Unit skin friction between the pile material and soil
$F_w = $ Weight of soil above the bell

$$F_w = \pi (R^2 - r^2) \gamma$$  \hspace{1cm} (2.5)

where, $R = $ Radius of the bell
$\gamma = $ Unit weight of the soil

May not resist uplift forces, causing tension forces.
The main advantage of the belled pier is that the resistance against uplift will not be affected by loss of friction in the zone unaffected by wetting.

2.4.6.3 GRANULAR PILE ANCHORS

Granular piles, also called stone columns, are being used during the last few decades, as a technique to improve the carrying capacity of soft cohesive soils and loose cohesionless deposits (Hughes and Withers, 1974; Datye, 1982).

Loose or medium dense sands are compacted by vibro-flotation to the desired relative density, resulting in a well compacted cylindrical column of desired diameter improving the strength characteristics. In case of soft cohesive soils, the soil is replaced by selected granular material (like crushed stone, preferably graded angular stones (of 75 mm to 100 mm size) compacted by vibro-replacement method to the densest possible state resulting in a stone column.

Stone columns are used where moderate increase in bearing capacity is required. They are used with advantage for lightly loaded structures, eliminating need for deep foundations such as piles.

In expansive soils, heave becomes an important consideration in foundation design. Hence a mere granular pile may not be sufficient when the swelling soil is likely to shear off the granular pile, which in particular, hence, may not resist uplift forces, causing tension forces. To make the granular pile...
tension-resistant and as a member which counter heave, it is anchored at the bottom to a mild steel plate through a mild steel rod. This system is called the granular pile anchor system. (Phani Kumar and Srirama Rao, 1996).

In this technique, the uplift force caused by the swelling soil on the foundation is resisted by

i) The weight of the granular pile

ii) The frictional resistance developed along the pile-soil interface which in turn has two components, namely

   a) The lateral pressure on account of the vertical overburden

   b) The lateral swell pressure

Thus, the resistance to uplift, \( F_u \), is given by

\[
F_u = W_p + [(\sigma_s K + \sigma_v) \tan \phi' + c] \frac{2 \pi r L}{2}
\]

where \( W_p \) = weight of the granular pile

\( \sigma_s \) = mean swell pressure

\( K \) = coefficient of lateral swell pressure

\( \sigma_v \) = mean effective vertical overburden pressure

\( K_s \) = coefficient of lateral earth pressure

\( \phi' \) = effective angle of friction between the column material and the swelling soil

\( c' \) = effective cohesion between the column material and the swelling soil

\( r \) = radius of the granular pile

\( L \) = length of the granular pile

This should be greater than the uplift force, \( U \), given by

\[
U = \rho (M - M_0)
\]

(2.7)
Where \( p_s \) = vertical swell pressure

\( A_f \) = area of the footing

\( A_g \) = area of the granular pile

The advantage of this technique is that, besides being cost-effective, the bottom of the pile need not rest in the inactive zone. It can be rested in the active zone also, provided the length of the pile can mobilise the necessary withholding capacity. The technique is also simple and does not require deployment of skilled labour.

They were found to be extremely effective in not only arresting heave but also in increasing the load carrying capacity of the composite ground and improving the engineering properties of the surrounding expansive soil. (Phani Kumar and Srirama Rao, 1996)

The lateral swell pressure of the expansive soil is an additional factor, apart from the overburden stress, that builds up enormous shearing resistance along the pile-soil interface, which is the main cause of reduction in heave.

Granular pile-anchors can be used either singly under isolated footings or in a row under a continuous footing or in a cluster.

2.4.7 PILE FOUNDATIONS

Of the various foundation practices, pile foundations are normally considered to be the best for adoption in clayey soils. They are used at places where good bearing stratum is not available at shallow depths, and open excavation becomes practically difficult.

Driving, vibrating, jacking pre-cast members install them either as cast-in-situ members or. Based on the method of installation, piles may be classified as displacement (e.g.: driven vibrated etc..) or replacement (e.g.: bored cast-in-situ,
bored pre-cast etc.,) piles. Normally, pile foundations are designed to carry compressive, uplift and lateral loads coming from the superstructure and, sometimes, to act as anchors and protection against scour. Occasionally, a pile has to transmit twisting moment also about its vertical axis. In the case where large lateral loads have to be resisted, in addition to vertical loads, piles are driven at an angle and are called batter piles. They are found to act efficiently when they are established in the direction of the resultant of vertical and horizontal loads.

The mechanism of load transfer by a pile to the surrounding soil depends upon the type of load, subsoil characteristics and the type of pile itself. The load coming onto the foundations is transferred to the soil through skin friction or end-bearing or combined action. Piles are classified as friction piles (floating piles), or end-bearing piles or combined action piles respectively depending on whether the load is by skin friction only, or point bearing only or a combination of the two.

Normally, a single pile is not used under a column. Piles are installed in groups provided with a cap. The numbers of piles depend on the load coming from the superstructure.

2.4.7.1 METHODS OF ANALYSIS OF PILE FOUNDATIONS

Starting with frictional piles, different methods of analysis (Coyle and Reese, 1966; Butterfield and Banerjee, 1971; Ellison et al, 1971; Desai, 1974; Meyerhoh, 1976; Thurman and D'Appolonia, 1965; Jeong et al, 1997) have been proposed to estimate the load-displacement behaviour of piles, which transfer load to soil essentially by skin friction.
2.4.6 CONVENTIONAL PLASTIC EQUILIBRIUM ANALYSIS

The various methods of determination of the load carrying capacity of piles normally consider plastic equilibrium conditions between the pile surface and the soil around it. They may be grouped as

i) Static formula

ii) Dynamic formula

iii) Pile load testing

No single method gives a complete idea on the performance of the pile under loading. A combination of two, or preferably three and judgement, helped by experience, is required to establish safe and economic pile design. However, the first method is more appropriate in this context and is discussed here.

2.4.8.1 STATIC FORMULA:

The total resistance of the pile foundations (Fig. 2.11) is divided into two components: (i) skin friction resistance \( Q_s \) and (ii) point bearing resistance \( Q_p \).

\[ Q_{si} = Q_s + Q_p \]  \hspace{1cm} (2.8)

Fig 2.11. Load carrying capacity of Pile Foundations

Thus, the total load carrying capacity \( Q_{si} \) is given by
Where

\[ Q_u = \text{Ultimate skin resistance} \]
\[ Q_{pu} = \text{Ultimate point-bearing resistance} \]
\[ A_s = \text{surface area} \]
\[ A_p = \text{bearing area of the pile} \]

Static formula for a single pile in cohesive soils

The ultimate static pile capacity in point bearing can be computed using the bearing capacity equations for shallow foundations. The breadth term \( N_p \) is, however, neglected since the width of the pile is insignificant in comparison with the length.

Thus,

\[ Q_{pu} = c_u N_c \tag{2.9} \]

Where \( N_c \) is normally taken as 9 for deep foundations.

For piles embedded in saturated cohesive soils, the static load capacity is estimated using undrained soil parameters (\( \phi = 0 \) analysis)

The ultimate skin resistance may be determined using the following three approaches.

(i) \( \alpha \) - method (total stress approach)
(ii) \( \beta \) - method (effective stress approach)
(iii) \( \lambda \) - method (pseudo - effective stress approach)

\( \alpha \) - method (Tomlinson, 1971)

According to this method, the unit skin resistance is given by

\[ q_s = \alpha c_u \tag{2.10} \]

where \( \alpha = \text{adhesion factor, which depends on the undrained cohesion of the soil} \)

\( \approx 1.0 \) for very soft clays \((c_u < 40 \, \text{KPa})\)
For very stiff clays \((c_u > 300 \text{ KPa})\)

The value of \(\alpha\) is interpolated for intermediate values of \(c_u\)

\(\beta\) - method (Burland, 1973)

According to one school of thought, piles in clay should be analysed by the effective stress approach, because

- the excess pore water pressure due to pile loading develops over a small zone surrounding the pile. This dissipates quickly through the cracks in the soil.
- for piles in stiff over-consolidated clays, the drained load capacity may be more critical than the undrained load capacity.

The skin resistance is expressed as

\[
q_s = \sigma_v' K_s \tan \phi' = \beta \sigma_v' \tag{2.11}
\]

- mean effective overburden stress
- \(\phi'\) = effective friction angle between the pile and the soil (taken as approximately equal to \(\phi\) for clayey soils.)
- \(K_s\) = coefficient of lateral earth pressure

\[
= K_s = (1 - \sin \phi') \text{ for normally consolidated clays}
\]

\[
= (1 - \sin \phi') \sqrt{OCR} \text{ for over consolidated clays}
\]

where \(OCR\) is over consolidation ratio

\(\lambda\) - method (Vijayvergia and Focht, 1972)

The skin resistance is given by

\[
q_s = \lambda (\sigma_v' + 2c_u) \tag{2.12}
\]

where \(\lambda\) = coefficient depending on pile embedded length.
For clays possessing both \( c' \) and \( \phi' \)

\[
q_p = c'N_c + \sigma'_vN_q
\]  

(2.13)

Where \( c' \) = effective cohesion

\( \sigma'_v \) = effective vertical stress at the pile tip level

\( N_c \) & \( N_q \) are bearing capacity factors

( different from those for shallow foundations)

**Static Formula for Single Pile in Cohesionless Soils**

For such a case \( c' = 0 \).

The unit frictional resistance is given by the formula

\[
q_s = K_s\sigma'_v\tan\phi'_v
\]  

(2.14)

\( K_s \) is coefficient which is approximately equal to \( K_p \) at the pile top and may be less than \( K_o \) at the pile tip, where \( K_o \) and \( K_p \) are coefficient of earth pressure at rest and passive earth pressure coefficient respectively. \( K_s \) also depends on the method of pile installation.

For bored or jetted piles: \[ K_s = K_o = 1 - \sin \phi' \]

For small displacement piles: \[ K_s = K_o \text{ to } 1.4K_o \]

For large displacement piles: \[ K_s = K_o \text{ to } 1.8K_o \]

Based on 'N' values, Meyerhof (1976) has suggested that the average skin resistance,

for large displacement piles: \[ q_s(kPa) = 2\bar{N} \]  

for small displacement piles: \[ q_s(kPa) = \bar{N} \]  

(2.16)

where \( \bar{N} \) is the average of \( N \) values over the length of the pile.

When piles are driven to the underlying rock layer, the ultimate point resistance is
where

\[ q_u = \text{unconfined compression strength of rock} \]

\[ N_\phi = \tan^2(45 + \phi'/2) \]

\[ \phi' = \text{drained angle of friction.} \]

The unit point resistance \( q_p \) is given by

\[ q_p = \sigma'_v N_q \]

\( \sigma'_v = \text{effective overburden pressure at the pile tip} = \gamma' L \)

\( \gamma' = \text{submerged unit weight of the soil} \)

Bearing capacity factor \( N_q \) is usually taken from the values given by Berezantsev et al(1961).

The above formula shows that the ultimate carrying capacity increases continuously with embedded depth. However, research has shown (Vesic, 1973; Meyerhof,1976) that the point bearing resistance does not increases indefinitely with \( L \) but reaches a limiting value at a certain depth, called critical depth. (fig.2.12).

Similarly, the frictional resistance along the pile shaft also increases upto certain depth and then remains constant. This depth is different for point resistance and skin resistance. However, the cause of this limiting resistance is attributed to soil arching, crushing below the pile tip etc.,

\[ \text{Fig. 2.12. Critical depth} \]
Unlike in cohesive soils, the point bearing resistance contributes more to the ultimate load carrying capacity of the pile, than the skin friction. Hence, shorter piles are more efficient and economical in cohesionless soils.

Based on uncorrected $N$ values, (Meyerhof, 1976).

For piles driven into fine to medium sand:

$$q_p = 40N \frac{L}{D} \times 400 \ kPa$$

For piles driven into coarse sand and gravel:

$$q_p = 40N \frac{L}{D} \times 300 \ kPa$$

For bored piles:

$$q_p = 14N \frac{L}{D}$$  \hspace{1cm} (2.19)

Piles are normally used in groups rather than as isolated piles. If the number of piles in the group are $n_g$, the load carrying capacity of the group is less than $n_g q_{sl}$, where $q_{sl}$ is the safe load on a single isolated pile. This is because of the overlapping of the pressure bulbs of the individual piles in a group.

The behaviour of the group is best represented by ‘group efficiency ($\eta$)’ as

$$\eta = \frac{\text{Ultimate load capacity of pile group}}{\text{Sum of the individual load capacities of N piles}}$$ \hspace{1cm} (2.20)

Different empirical formulae are defined to evaluation of $\eta$. One such equation often referred to is the Converse - Labarre formula, given below.

$$\eta = 1 - \theta \left(\frac{(m' - 1) n'}{m' n'} + \frac{n (m - 1)}{m' n'}\right)$$

where $\theta (\text{deg}) = \tan^{-1} \left( \frac{d}{s} \right)$

where $s = \text{spacing of piles in the group}$

$m' = \text{number of rows in the group}$
For piles driven into a cohesive soil, \( \eta \) has been found to be less than 100%, whereas for those installed in granular soils, \( \eta \) has been found to be more than 100%. But for design purpose, \( \eta \) should be treated always as less than 100%.

Another method of assessing load carrying capacity is by assuming block failure of the entire group, as given by Terzaghi and Peck (1948). The Ultimate group capacity \( Q_{pw} \) is given by

\[
Q_{pw} = c_n N_c A_g + 2(L_x + B_g)Lc_n
\]

where

- \( A_g \) = area of the group = \( L_g \times B_g \)
- \( N_c \) = Bearing capacity factor using Skempton's formula
- \( L \) = length of the pile
- \( L_g \) and \( B_g \) are the length and width of the pile group respectively

2.4.9 ELASTIC ANALYSIS

With the advent of computers, more sophisticated methods for evaluation of distribution of load and settlement along the pile have been developed. The elastic approach is one such development.

2.4.9.1 ELASTIC APPROACH FOR PILES IN INEXPANSIVE CLAYS

The earlier analyses assuming full mobilisation of the stresses along the pile-soil interface is an extreme case and is far from reality. In actual practice, at working conditions, the situation is close to elastic conditions. In the true sense, the soil is not elastic in any range of stresses, i.e., the stress-strain curve is not a straight line. Neither the displacements occurred during the loading are recovered totally when the loads are removed. But on the observation that the stress increases continuously with the increasing strain, though not linearly, it may be taken that elastic conditions may be assumed for the analysis of the piles, especially in saturated clays. However,
some empirical coefficients are to be affixed to suit the practical conditions before actually using in the field.

The basic principle in the elastic analysis is that both the soil and the pile are assumed as elastic bodies. When the load is applied on the pile, surface stresses are developed along the surface of the pile and, simultaneously, in the surrounding soil. The pile and the soil act as a composite body, resisting the load applied on it. Depending on their properties, displacements occur both in the pile and the soil, which are written as functions of the surfaces forces acting at the interface of pile and soil. For elastic compatibility, the displacements in the pile as well as in the soil should be equal. Solving the compatibility equation, the surfaces forces on the pile-soil interface can be obtained.

There are essentially three types of loading which may conveniently be considered to represent the shear stress on a pile element

i) a vertical point load acting on the axis of element

ii) a uniform vertical line load acting down the axis on element

iii) a vertical stress uniformly distributed around the outer circumference of the element.

Model (i) is adopted by D'Appolonia and Romualdi (1963), Salas and Belzunce (1965), Thurman and D'Appolonia (1965). D'Appolonia and Ramualdi (1963) employed equations of Mindlin in solving the problem of an embedded pile.

The shear stress on each of the elements is assumed to act as a point load at its centre. The pile is divided into a number of elements and the solutions are obtained by imposing compatibility conditions between the displacement of the pile and that of the soil. The pile is assumed to be compressible and the mirror image technique is employed to ensure the zero vertical displacement condition at the pile tip.
An analysis of rigid axially loaded pile has been presented by Salas and Belzunce (1965). The approach is same as, but also included pile-soil slip in a purely cohesive soil.

Model (ii) was given little attention by researchers. Model (iii) is accurate, but complex. But the use of computer programmes make the analysis no more difficult to work with.

Poulos, and his associates contributed significantly in this direction. Poulos (1968) first made an analysis for the study of behaviour of singly axially loaded incompressible piles and piers. Poulos and Davis (1969) studied the settlement interaction between two identical piles in an elastic mass. The increase in settlement of each pile in an elastic mass and the increase in settlement of each pile due to the interaction is expressed in terms of interaction factors ($\alpha$). It was shown that for symmetrical pile groups (for piles with equal settlement or those which equally loaded), the increase in settlement due to interaction may obtained by superposition of the values of ‘$\alpha$’ for the individual piles in the group. They studied the influence of pile spacing, pile length, type of group, depth of layer and Poisson’s ratio, on the settlement of group. It is observed that major portion of settlement is obtained as immediate settlement, as in a single pile.

Poulos & Mattes (1969) studied the rate of development of negative skin friction on compressible end-bearing pile resting in a consolidating soil, underlain by a rigid layer which is submerged. Various possible practicable cases were analysed. It is observed that ‘$L/d$’ and ‘$K$’ have a major influence on the down-drag force.

Poulos & Davis (1972) extended the above analysis to find the rate of development of negative skin friction on compressible pile resting in a
consolidating soil, which is submerged. The effects of consolidation of the soil is studied with reference to various possible practical cases.

Poulos and Davis (1975) performed an analysis for end-bearing piles for full pile-soil slip along with some parametric study.

In his analysis, for long compressible piles, Poulos (1982) observed that the load required to cause a head settlement of 10% of pile diameter may be significantly less than the ultimate load capacity. A dimensionless solution is provided to enable rapid estimation of the combined effects of length and compressibility on the practicable load capacity of the pile.

In all these analyses, the effect of the disturbance caused by the driving or the installation of the pile has not been taken into account. Holloway et al. (1975) suggested a method to evaluate these stresses. Poulos (1980) suggested the use of a suitably modified soil modulus ‘$E_s$’ to simplify matters.

Chow (1986) presented an analysis of linear and non-linear response of vertically loaded pile groups, with the soil behaviour modelled using load-transfer curves and the pile-soil-pile interaction is determined based on Mindlin’s solution.

While some others analyzed the pile foundations for negative skin friction using Finite Element Method. Ito & Matsui (1976) investigated the growth mechanism of negative skin friction and method of its estimation, influence of pile settlement and groups on its development using Finite Element Method. Sangseom et al. (1997) made a limited parametric study and observed that down-drag on piles in a group is much less than that on single pile and that it is more influenced by group spacing, number of piles and relative position of piles in the group.

Kuwabara and Poulos (1989), using the same approach followed by Poulos and Davis (1975) for a single pile, have done analysis for end-bearing group
piles arranged in square pattern, subject to negative friction. The analysis is carried out for homogeneous and non-homogeneous conditions, both for slip and no-slip conditions between pile and soil. They observed that the inner piles are less influenced by down-drag and that the length on them over which full slip occurs is less than that of the outer piles of the group. They further observed that larger soil surface movement is required to mobilise full slip for a group rather than for a single isolated pile and that for a group with a rigid cap, possible tensile forces are developed at the top of the outer piles.

The analysis of axially loaded piles in isotropic or cross-anisotropic soils is studied by Lee and Small (1991) by finite layer analysis, whose results were quite close to some field observations. However, the method is more tedious and involves more labour to work with.

2.4.9.2 ELASTIC APPROACH FOR PILES IN EXPANSIVE SOILS

Poulos and Davis (1973, 1980) have made an attempt at the study of piles resting in expansive soils using the elastic approach. They have made a study on a single incompressible pile resting in an expansive clay:

A method of assessment of performance of piles in expansive clays is made by Challa and Poulos (1991), by conducting a series of laboratory tests. The comparison between theory using the boundary layer approach and practical values have shown that variation of pile movement and pile force with time could not be predicted accurately.

Joram et al (1976) and Yahia E-A Mohamedzein et al (1999) also studied the behaviour of short piles in expansive soils using FEM. The latter have assumed pile as linearly elastic and soil as a non-linear elastic model. From their study, they concluded that increase in pile length decreases the upward vertical
movement of pile and with increase in vertical load, both vertical movement and
tensile stress decrease. Further, they observed that tensile stresses for loaded piles
occur along most portions of the pile within the active zone with maximum stress
developing near the mid height of the pile.

Model tests were conducted on piles embedded in clay specimens
studied by Komornik(1973), to assess the effect of swelling clay on piles and a
design procedure for the piles resting in swelling clays is developed.

2.4.10 UNDER -REAMED PILES

Under-reamed piles, are an extension of the belled-pier technique and
are most efficacious in isolating the structure from the foundation soil. Sharma et al
(1978) brought out a handbook on under-reamed piles.

Under-reamed piles are bored cast-in-situ piles with enlarged bases. A
single under-reamed pile has enlargement at the base while a multi-underreamed pile
has more than one bulb along the length of the pile and connected at their top by
plinth beams. They are used effectively to resist uplift loads for the foundation of
transmission lines, antennas and other elevated towers, dry docks, under-ground tanks
etc. They are practically useful to resist uplift forces caused by swelling of expansive
clays. Due to enlarged bases, these piles provide good anchorage to light structures
against their being pulled up during the swelling of the soil. The enlarged base
increases the end-bearing area of the pile in the stable zone and thus improves its
bearing capacity.

The drawback with under-reamed piles is that it is difficult to form
bulbs in soils that do not have cohesion. The construction technique is highly
specialised and, as such, untrained labour cannot install them properly.
2.4.10.1 LOAD-CARRYING CAPACITY OF UNDER-REAMED PILES

Based on the approach of slip at the pile-soil interface, a systematic study of these piles was first attempted in South Africa (Jennings and Henkel, 1949). This was followed by a report from Australia by Tasker et al. (1950) which gave an empirical expression $R$ for the anchoring capacity in tons is given as

$$R = 21 \pi c_s (R_0^2 - 1.5r^2)$$

(2.23)

where

- $c_s =$ Cohesion in tons / ft$^2$
- $R_0 =$ Radius of the under-ream in ft and
- $r =$ radius of the pile stem in ft.

Skempton's (1959) static formula for bored piles can be conveniently used in finding out the ultimate bearing capacity of under reamed piles.

$$Q_s = A_p N_c c_{ub} + A_s \alpha c_{us}$$

(2.24)

where

- $A_p =$ Area of base in m$^2$
- $A_s =$ Surface area of the pile shaft in m$^2$
- $N_c =$ Point bearing capacity factor
- $c_u =$ Undrained cohesion of the soil at base of the pile in k Pa
\[ c_2 = \text{undrained cohesion of the soil around the shaft in k Pa} \]

\[ \alpha = \text{Shaft adhesion factor} \]

The bearing capacity factor \( N_c \) is found to be 9 for cohesive soils and the adhesion factor \( \alpha \) is found to be of the order of 0.50 (Mohan and Chandra, 1961).

Based on the studies (Meyerhof and Adams, 1968) regarding the uplift resistance of a circular plate embedded in a \( c-\phi \) soil, Tomlinson (1977) has suggested the use of the following equation to estimate the uplift capacity of piles with base enlargements.

\[ Q_u = \pi D_u H + \left( \frac{1}{6} \right) \pi \gamma D_u (2d_f - H) K \tan \phi + W \]  
\[ (2.25) \]

where

\[ Q_u = \text{Ultimate uplift resistance of pile} \]

\[ c = \text{cohesive strength of the soil} \]

\[ D_u = \text{diameter of the enlarged base} \]

\[ H = \text{Height of block of soil uplifted by the pile} \]

\[ s = \text{shape factor} \]

\[ \gamma = \text{Unit weight of soil} \]

\[ d_f = \text{Depth of pile below ground level} \]

\[ K = \text{Coefficient, dependent on angle of internal friction \( \phi \)} \]

\[ W = \text{Weight of soil and pile within a cylinder of diameter } D_u \]

Soneja & Garg (1980) have studied the effect of various aspects of under-reamed piles, e.g., the shaft diameter, the no. of bulbs, depth of pile, location of the bulb along the height, etc., Prototype and model piles were tested under lateral loads in cohesionless soils.
2.4.10.2 LOAD-CARRYING CAPACITY OF MULTI-UNDERREAMED PILES

The following conventional method is adopted for estimation of the ultimate load capacity of multi-bulb under-reamed pile foundation.

In the case of a multi-underreamed pile, frictional resistance is mobilised over a larger perimeter and reduces the uplift due to swelling.

The soil between two bulbs serves as a single unit provided the vertical spacing between the centres of the bulbs is restricted to $1.25 D_u$ to $1.5 D_u$. For piles with stem diameter greater than 300mm, the spacing is $1.25 D_u$, while for smaller diameters it is $1.5 D_u$. The frictional resistance is mobilised over an enlarged perimeter corresponding to the diameter of the bulb as shown in Fig. 2.13.

The Bureau of Indian Standards has also brought out a code (IS: 2911 - Part III, 1980), in which a table is given for the safe load without reference to the soil properties, which is unreliable. However, in the above code, the Ultimate bearing capacity $Q_u$ in clays, given in kN, calculated from soil properties is given as

$$Q_u (KN) = A_p N_c c_p + A_u N_c c_u + c' u A_s + \alpha c_a A,$$  \hspace{1cm} (2.26)

Where

$$A_s = \frac{\pi}{4} (D_u^2 - d^2) \quad m^2$$

where

$c_{tp}$ = undrained cohesion at the pile tip

$c_{ub}$ = Average cohesion of soil around under-ream bulb (k Pa)

$A_s'$ = Surface area of the cylinder circumscribing the under-ream bulbs

$c_a$ = Average cohesion of soil along the pile stem (k Pa)

$\alpha$ = Reduction factor (usually taken as 0.5 for bored piles)

$N_c = 9$ for pile foundations
‘First term’ will be absent, for the determination of load on the pile in uplift, and ‘third term’ will be absent if only one bulb were present in the pile foundation.

The depth of the active zone plays an important role in the determination of load carrying capacity. The soil within the active zone should not be considered as contributing to the safe load either in compression or uplift. In India, mainly the active zone is confined to a depth of about 3.5m below ground level, but may occasionally exceed 5m. Since the bulbs are to be anchored below the active zone, it influences greatly the cost of the foundation.

The elastic approach is adopted in this thesis for analysing uniform diameter piles and enlarged base piles, as single and in group of piles in homogeneous and non-homogeneous soils, and is presented in the subsequent chapters.