2.1Depth of foundation:

Many factors affect the depth of foundation. such as type of soil, ground water table, loads from structure, bearing capacity and density of soil and other factors. The minimum depth of foundation is calculated by Rankine's formula when the bearing capacity of soil is known by soil investigation report.

General factors to be considered for determining depth of foundation are:

1. Load applied from structure to the foundation
2. Bearing capacity of soil
3. Depth of water level below the ground surface
4. Types of soil and depth of layers in case of layered soil
5. Depth of adjacent foundation

The minimum depth of foundation should be considered to ensure that the soil is having the required safe bearing capacity as assumed in the design. However, it is advised to carry out soil investigation before deciding on depth of foundation.

Soil investigation report will suggest the foundation depth based on the type of structure, soil properties, depth of water table, and all other variable that should be considered. Soil investigation report provides bearing capacity of soil at different levels and at different locations.

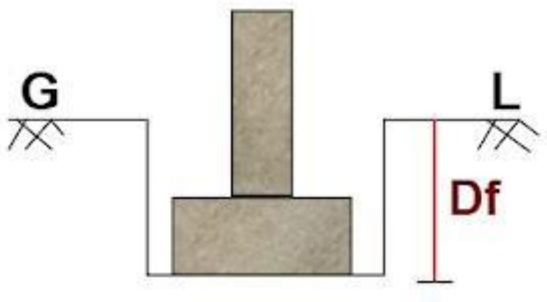


Fig 1.Depth of Foundation

[Fig 1 https://www.paramvisions.com/2021/04/calculating-depth-of-foundation-by.html]

When the soil investigation report is not available, the depth of foundation should be selected such that it is not affected by swelling and shrinking of soil due to seasonal changes. Depth of foundation should also consider the depth of water table to prevent and scour below the ground.

For foundation near existing foundation, it must be ensured that pressure bulbs of foundations do not coincide if the depth of new foundation has to be taken below the depth of existing foundation.

The foundation should not be contracted at shallow depth considering the frost action in cold countries.

Rankine's formula provides the guidance on minimum depth of foundation based on bearing capacity of soil.

Rankine's Formula

$$h = \frac{p}{\gamma}\left(\frac{1 - sin\varphi}{1 + sin\varphi}\right)^2$$

Where, h = minimum depth of foundation

p= gross bearing capacity

γ= density of soil

φ= angle of repose or internal friction of soil.

The above formula does not consider the factors discussed above and just provides the guidance on minimum foundation depth, assuming that the foundations are not affected by factors such as water table, frost action, types and properties of soil etc. as discussed above. This formula does not consider the loads from the structure on the foundation.

In the Rankine's formula, it can be seen that foundation depth depends on the bearing capacity of soil, so, if the bearing capacity of soil increases, the depth of foundation also increases.

FACTORS AFFECTING DEPTH OF FOUNDATION

- Before calculating depth of shallow foundation, the following factors have to be considered well in advance.
- Foundation should be placed at such a depth so that it is safe against damages due to swelling, shrinkage or freezing of sub soil.
- Bearing capacity of soil beneath the foundation must be adequate to support the load coming from foundation.
- If foundation has to be placed on cohesive soil, then the settlement due to consolidation should not be excessive.
- Never place foundation on loose or disturbed soils which have a tendency to erode by wind or flood.
- If possible then foundation should be placed above ground water table as this can avoid cost of pumping, and can prevent instability of soil due to seepage of water into the bottom of an excavation.
- Make an investigation on foundation soil to know its physical and chemical properties, because presence of sulphate can damage foundation.

To perform its function properly a footing must be laid at a suitable below the ground surface. The vertical distance between ground surface and the base of the footing is known as the depth of the footing(D_f). The depth of the footing contains the ultimate bearing capacity and the settlement. While fixing the depth of footing, the following points should be considered.

1.Depth of top soil:

The footing should be located bellow the top soil consisting the organic materials which eventually decompose. The top soil should be removed over an area slightly larger than the footing.

2.Frost depth:

The footing should be carried bellow the depth of frost penetration. If the footing is located at insufficient depth, it would be subjected to the frost damage due to formation of ice lenses and consequent frost heave. During summer, thawing occurs from the top downwards and the melted water is entrapped

3. Zone of soil volume change:

Some clay, especially clays having high plasticity, such as black cotton soil, undergoes excessive volume changes. Such soil shrinks upon drying and swells upon wetting. The volume changes are generally greater near the ground surface and decreases with increase in depth. Large volume change beneath a footing may cause lifting and dropping. The footing should be placed bellow as strata that are subjected to large volume change.

4. Adjacent footing and property lines:

- The footing should be so located that no damage is done to the existing structure. The adjacent structure may be damaged by construction of a new footing due to vibrations, undermining or lowering of the water table. The new footing may also impose additional load on the existing footing which may cause settlement.
- In general, deeper the new footing and closer to the existing structure the greater is the potential damage to the existing structure. This is this is particularly more severe if the new footing is lower than the existing footing.
- As far as possible, the new footing should be placed at a small depth as the old ones and the sites of excavation adjacent to the existing structure should be suitably supported. If the footings are placed at the different levels, the slope of the line joining the two footings should not be steeper than two horizontal to one vertical as per IS: 1904-1978.

5. Sloping ground:

If a footing is located adjacent to a sloping ground, the sloping ground surface should not encroach upon a frustum of bearing material under the footing having sides making an angle of 30‟ with the horizontal. Moreover, the minimum distance from lower edge of the footing to be sloping ground surface should be 90cm.

6.Water table:

The footing should be placed above ground water table as far as possible. The presence of ground water within the soil immediately around a footing in undesirable as it reduces the bearing capacity of the soil and there are difficulties during construction. The water proofing problem also arises due to dampness.

7.Scour depth:

The footings located in streams, on water fronts or other locations where there is a possibility of scouring should be placed below the potential scour depth.

8.Underground defects:

The depth of footing is also affected by the presence of underground defects such as faults, causes and mines. If there are manmade discontinuities, such as sewer lines, water mains, underground cables, these should be shifted or footing should be relocated.

9.Root holes:

If there are root holes or cavities caused by burrowing animals or worms, the footing should be placed bellow such a zone of weakened soil.

10.Minimum depth:

IS 1904 – 1978 specifies that all foundations should extend to a depth of a least 50cm below the natural ground surface. However, in case of rocks, only its top soil should be removed and the surface should be cleaned and if necessary sepped.

2.2 Bearing capacity:

Bearing capacity: It is the load carrying capacity of the soil.

The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure.The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil. There are three modes of failure that limit bearing capacity: general shear failure, local shear failure, and punching shear failure. It depends upon the shear strength of soil as well as shape, size, depth and type of foundation.

Basic definitions:

1. Ultimate bearing capacity or Gross bearing capacity (q_u):

It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

2.Net ultimate bearing capacity (q_{nu}):

It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it.

3.Safe bearing capacity:

It is the bearing capacity after applying the factor of safety (FS). These are of two types,

a. Safe net bearing capacity (q_{ns}) :

It is the net soil pressure which can be safety applied to the soil considering only shear failure.

(or)

Net ultimate bearing capacity is divided by certain factor of safety will give the net safe bearing capacity.

$$q_{ns} = q_{nu}/ F$$

Where F = factor of safety = 3

b. Gross Safe bearing capacity (q_s):

It is the maximum gross pressure which the soil can carry safely without shear failure.

(or)

When ultimate bearing capacity is divided by factor of safety it will give gross safe bearing capacity.

$$qs = qu/F$$

4.Allowable Bearing Pressure(q_a):

It is the maximum soil pressure without any shear failure or settlement failure.

5. Net safe settlement pressure (q_{np})

The pressure with which the soil can carry without exceeding the allowable settlement is called net safe settlement pressure.

6. Net allowable bearing pressure (q_{na})

This is the pressure we can used for the design of foundations. This is equal to net safe bearing pressure if $q_{np} > q_{ns}$. In the reverse case it is equal to net safe settlement pressure.

Methods of Improving the Bearing Capacity of soils:

The bearing capacity of a soil mainly depends on the closeness of its particles. The bearing capacity of a soil can be increased by the following methods:

1.By increasing the depth of foundation.

The compactness of the foil increases as we go below the ground level. As the bearing capacity directly depends on the compactness of the soil, it will go on increasing as the depth of foundation is increased.

2.By draining of the sub-soil under.

Water reduces the cohe-sive properties and hence reduces the bearing capacity of the soil. By draining off water from the sub-soil the bearing capacity of the soil is certainly increased.

3.By compacting the soil.

If the soil is compacted thoroughly, the voids are decreased and bearing capacity is increased.

4.By confining the soil and preventing it from spreading and lateral movement.

Spreading soils, if confined by sheet piling will resist more leads, that is, their bearing capacity will increase.

5.By increasing the width of foundation.

By increasing the width of foundations, the intensity of load is decreased and on the same soil more loads can be placed.

6.By hardening the soil by grouting, i.e. pumping in the cement -grout into the ground.

7. By grouting, the cohesive properties are in-creased and the soil will be able to take up more loads.

8.By solidifying the ground by chemical processes.

In this case also the soil is compacted by mixing certain chemicals such as ‘calcium chloride etc.

Factors influencing bearing capacity of soils

1. Soil Strength
2. Foundation Width
3. Foundation Depth
4. Soil Weight and Surcharge
5. Spacing Between Foundations
6. Earthquake and Dynamic Motion
7. Frost Action
8. Subsurface Voids
9. Expansive and Collapsible Soils
10. Potential Heave
11. Soil Reinforcement
12. Soil Erosion and Seepage

1.Soil Strength:

Bearing capacity of cohesionless soil and mixed soil increases unproportionally with the increase of in the effective friction angle. However, bearing capacity of cohesive soil varies linearly with the soil cohesion provided that the effective friction angle is zero.

2. Foundation Width

- Foundation width affects bearing capacity of cohesionless soil. The bearing capacity of a footing placed at the surface of cohesionless soil, where the soil shear strength is considerably dependent on internal friction, is proportional to the width of the foundation. Bearing capacity of cohesive soil of constant shear strength and infinite depth is independent of foundation width.

3. Foundation Depth

- The greater the bearing capacity the deeper the foundation. This is specifically obvious in a uniform cohesionless soil. In contrary, if the foundation is carried down to a weak soil layer, then bearing capacity is declined.

- Foundations placed at depths where the structural weight equals the weight of displaced soil usually assures adequate bearing capacity apart from the case where the structure supported by under-consolidated soil and collapsible soil subject to wetting.

4. Soil Weight and Surcharge

The contribution of subsurface and surcharge soil, which are influenced by water table, to the bearing capacity cannot be ignored. The water table should not be above the base of the foundation to avoid construction, seepage, and uplift problems. If the water table is below the depth of the failure surface, then it has no influence on the bearing capacity.

5. Spacing between foundations

It is recommended to consider minimum spacing between footings, which 1.5 times foundation width, during the design of foundation in order to avoid reduction in bearing capacity.

6. Earthquake and Dynamic Motion

Repeated movements could increase pore pressure in foundation soil and consequently bearing capacity is decreased. Sources of cyclic movements are earthquakes, vibrating machinery, and other sources like vehicular traffic, blasting, and pile driving.

The foundation soil can liquify when pore pressures equal or exceed the soil confining stress. Liquefaction reduces effective stress to zero and causes gross differential settlement of structures and loss of bearing capacity.

7. Frost Action

Frost heave in certain soils in contact with water and subject to freezing temperatures or loss of strength of frozen soil upon thawing can alter bearing capacity over time. Low cohesion materials containing a high percentage of silt-sized particle are mostly susceptible to frost action.

8. Subsurface Voids

Bearing capacity of soil decreases due to subsurface voids which are within a critical depth beneath the foundation. The critical depth is that depth below which the influence of pressure in the soil from the foundation is negligible.

9. Expansive and Collapsible Soils

Collapsible and expansive soil can have large strength and bearing capacity when they are fairly dry. However, the volume of these soils changes due to changes in water content. This leads to total and differential foundation movements. Seasonal wetting and drying cycles may cause soil movements that often lead to excessive long-term deterioration of structures with substantial accumulative damage.

10. Potential Heave

The potential heave can be determined from results of consolidometer test which can be performed in accordance with ASTM D 4546. The results of this test is considered in determining preparation of foundation soils to reduce destructive differential movements and to provide a foundation of sufficient capacity to withstand or isolate the expected soil heave.

11. Soil Reinforcement

Bearing capacity of soft or weak soil can be increased greatly by installing various forms of reinforcement in the soil like metal ties, strips, or grids, geotextile fabrics, or granular materials.

12. Soil Erosion and Seepage

Erosion of soil around and under foundations and seepage can reduce bearing capacity and can cause foundation failure.

Types of shear failure of foundation soils:

Depending on the stiffness of foundation soil and depth of foundation, the following are the modes of shear failure experienced by the foundation soil.

- General shear failure (Fig.1(a))
- Local shear failure (Fig.1(b))
- Punching shear failure (Fig.1(c))

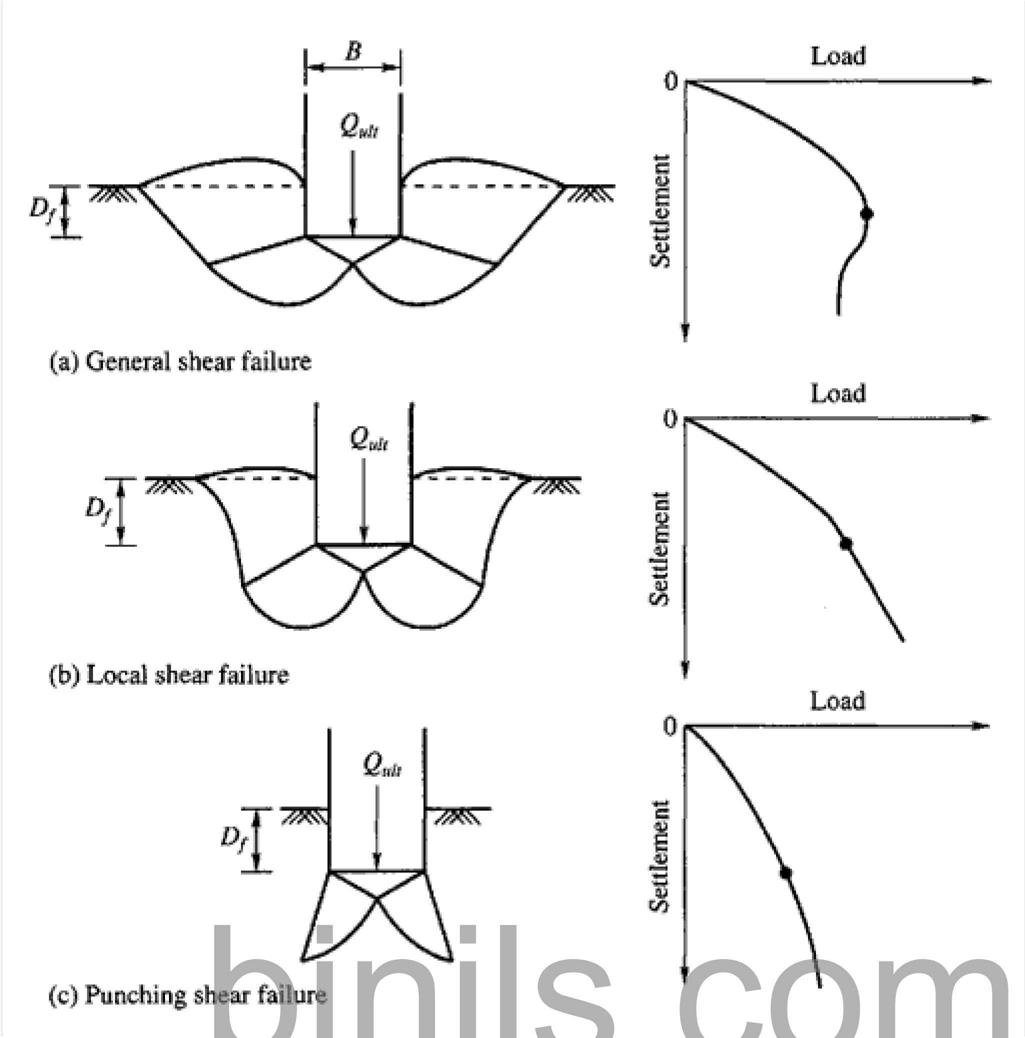


Fig 1 Different modes of failure

[Fig1 http://www.abuildersengineer.com/2012/11/types-of-failure-in-soil.html]

General Shear Failure:

- This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.
- Dense or stiff soil that undergoes low compressibility experiences this failure.
- Continuous bulging of shear mass adjacent to footing is visible.
- Failure is accompanied by tilting of footing.
- Failure is sudden and catastrophic with pronounced peak in curve.
- The length of disturbance beyond the edge of footing is large.
- State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.
- General shear failure is accompanied by low strain (<5%) in a soil with considerable (>36^o) and large N (N > 30) having high relative density (I_D> 70%).

Local Shear Failure:

- This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure. A significant compression of soil below the footing and partial development of plastic equilibrium is observed.
- Failure is not sudden and there is no tilting of footing.
- Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.
- Failure surface is not well defined.
- Failure is characterized by considerable settlement.
- Well defined peak is absent in curve.
- Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low (<28^o) and low N (N < 5) having low relative density (I_D> 20%).

Punching Shear Failure of foundation soils:

- This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure. This type of failure occurs in a soil of very high compressibility.
- Failure pattern is not observed.
- Bulging of soil around the footing is absent.
- Failure is characterized by very large settlement.
- Continuous settlement with no increase in P is observed in p-Δ curve.

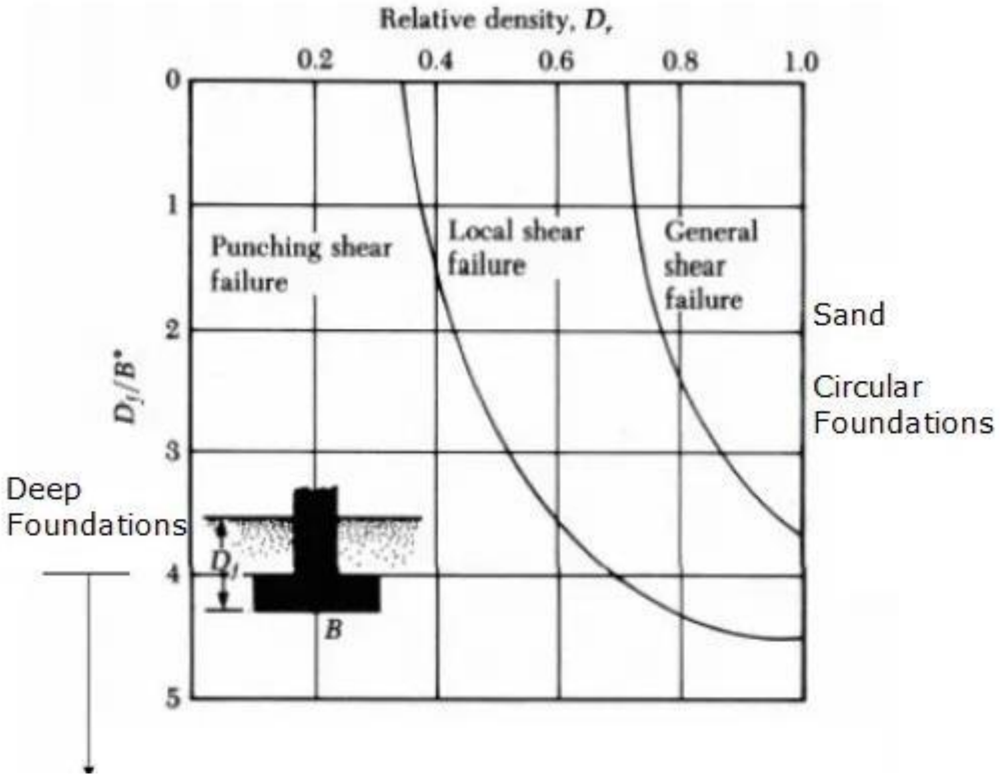


Fig. 2 presents the conditions for different failure modes in sandy soil carrying circular footing based on the contributions from Vesic (1963 & 1973)

[Fig 2 https://theconstructor.org/geotechnical/types-of-shear-failure-of-foundation-soils/7492/]

2.3 Terzaghi equation:

Terzaghi (1943) was the first to propose a comprehensive theory for evaluating the safebearing capacity of shallow foundation with rough base.

Assumptions:

1. Soil is homogeneous and Isotropic.
2. The shear strength of soil is represented by Mohr Coulombs Criteria.

The footing is of strip footing type with rough base. It is essentially a two dimensionalplane strain problem.

3. Elastic zone has straight boundaries inclined at an angle equal to the horizontal.
4. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soilabove the base of footing is neglected.
5. Method of superposition is valid.
6. Passive pressure force has three components (P_{PC} produced by cohesion, P_{Pq} produced bysurcharge and $P_{P\gamma}$ produced by weight of shear zone).
7. Effect of water table is neglected.
8. Footing carries concentric and vertical loads.
9. Footing and ground are horizontal.
10. The properties of foundation soil do not change during the shear failure
11. Limit equilibrium is reached simultaneously at all points. Complete shear failure ismobilized at all points at the same time.

Limitations:

1. The theory is applicable to shallow foundations
2. As the soil compresses, increases which is not considered. Hence fully plastic zone maynot develop at the assumed.
3. All points need not experience limit equilibrium condition at different loads.
4. Method of superstition is not acceptable in plastic conditions as the ground is near failurezone.

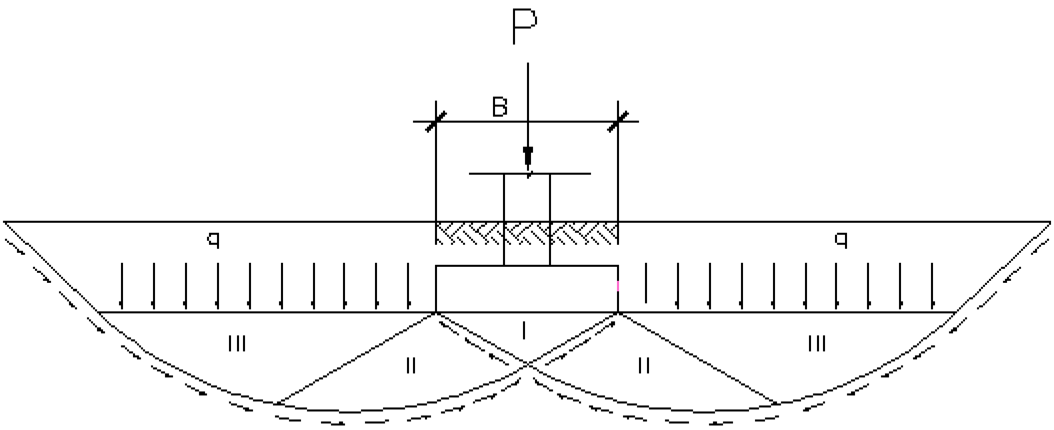


Fig 1 Shear stresses based on Terzaghi's soil bearing capacity theory.

[Fig 1https://civilengineeringbible.com/subtopics.php?i=1]

Terzaghi's concept of Footing with five distinct failure zones in foundation soil

- The soil is semi-infinite, homogeneous and isotropic
- The problem is two-dimensional
- The base of the footing is rough
- The failure is by general shear
- the load is vertical and symmetrical
- The ground surface is horizontal
- the overburden pressure at foundation level is equivalent to a surcharge load
- the principle of superposition is valid,

Coulomb's law is strictly valid, that is

$$\tau = C + \sigma tan\varphi$$

Mechanism of Failure:

- The shapes of the failure surfaces under ultimate loading conditions are given in Fig.
- The zones of plastic equilibrium represented in this figure by the area *gedcf* may be subdivided into three zones:

1 . Zone I of elastic equilibrium

2. Zones II of radial shear state

3. Zones III of Rankine passive state

- When load q_u per unit area acting on the base of the footing of width B with a rough base is transmitted into the soil, the tendency of the soil located within zone I is to spread but this is counteracted by friction and adhesion between the soil and the base of the footing.
- Due to the existence of this resistance against lateral spreading, the soil located immediately beneath the base remains permanently in a state of elastic equilibrium, and the soil located within this central Zone I behaves as if it were a part of the footing and sinks with the footing under the superimposed load.

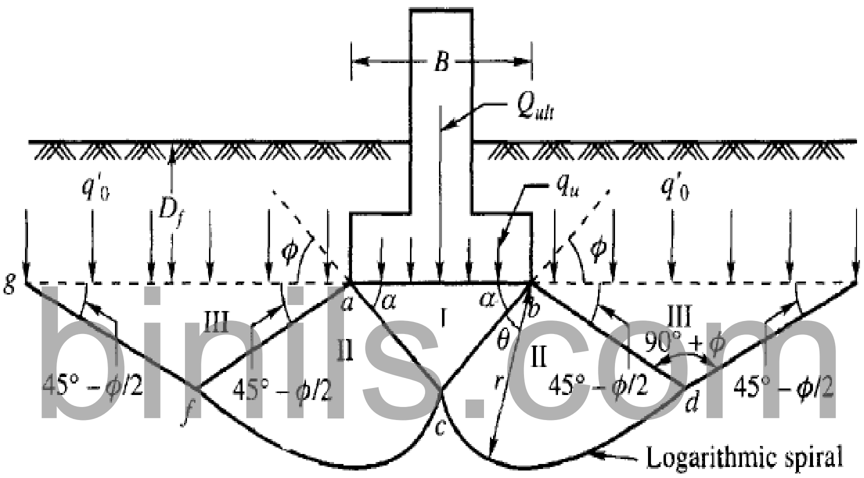


Fig 2 Shear stresses based on Terzaghi's soil bearing capacity theory

[Fig 2 https://www.pinterest.com/pin/682013937310128083/]

- The depth of this wedge shaped body of soil abc remains practically unchanged, yet the footing sinks.
- This process is only conceivable if the soil located just below point c moves vertically downwards. This type of movement requires that the surface of sliding *cd* (Fig.) through point c should start from a vertical tangent. The boundary be of the zone of radial shear bed (Zone II) is also the surface of sliding.
- As per the theory of plasticity, the potential surfaces of sliding in an ideal plastic material intersect each other in every point of the zone of plastic equilibrium at an angle (90° - ϕ). Therefore, the boundary *be must rise at an angle ϕ to the horizontal provided the friction* and adhesion between the soil and the base of the footing suffice to prevent a sliding motion at the base.

- The sinking of Zone I creates two zones of plastic equilibrium, II and III, on either side of the footing. Zone II is the radial shear zone whose remote boundaries bd and af meet the horizontal surface at angles (45° - ϕ/2), whereas Zone III is a passive Rankine zone. The boundaries de and fg of these zones are straight lines and they meet the surface at angles of (45° - ϕ/2). The curved parts cd and cf in Zone II are parts of logarithmic spirals whose centers are located at b and a respectively.

Downward force:

i)weight of soil wedge ABC

$$w = \frac{1}{4}\gamma B^2 tan\varphi$$

ii) Total load on footing $q_f B$

Upward force:

i)passive force

ii)cohesion(c)

Length of AC and CB

$$cos\varphi = \frac{adj}{hypo} = \frac{\frac{B}{2}}{AC}$$

$$AC = \frac{B}{2} x \frac{1}{cos\varphi}$$

Vertical component $C = (\frac{B/2}{cos\varphi}.C)\ sin\varphi$

$$C = (\frac{B}{2}.C)\, tan\varphi$$

$$vertical\ component\ of\ C = \frac{B}{2} Ctan\varphi$$

i)$2P_p$

ii)$\frac{B}{2} xtan\varphi x2C$

Upward =Downward

$$2P_p + BCtan\varphi = q_f B + \frac{1}{4}\gamma B^2 tan\varphi$$

$$q_f B = 2P_p + BCtan\varphi - \frac{1}{4}\gamma B^2 tan\varphi$$

The resultant passive earth pressure has 3 component

i)$P_{P\gamma} \rightarrow Produced\ by\ weight\ of\ shearzone\ BCDE$

ii) $P_{Pc} \rightarrow Produced\ by\ cohesion$

iii) $P_{Pq} \rightarrow Produced\ by\ surcharge\ q$

$$q_f B = 2(P_{P\gamma} + P_{Pc} + P_{Pq}) + BCtan\varphi - \frac{1}{4}\gamma B^2 tan\varphi$$

$$q_f B = 2P_{P\gamma} + 2P_{Pc} + 2P_{Pq} + BCtan\varphi - \frac{1}{4}\gamma B^2 tan\varphi$$

$$q_f B = [2P_{P\gamma} - \frac{1}{4}\gamma B^2 tan\varphi] + [2P_{Pc} + BCtan\varphi] + 2P_{Pq}$$

$$Let,\ 2P_{P\gamma} - \frac{1}{4}\gamma B^2 tan\varphi = Bx\frac{1}{2}\gamma BN_\gamma$$

$$2P_{Pc} + BCtan\varphi = BcN_c$$

$$2P_{Pq} = B\gamma DN_q$$

Substitute in above equation

$$q_f B = Bx\frac{1}{2}\gamma BN_\gamma + BcN_c + B\gamma DN_q$$

$$q_f B = B[\frac{1}{2}\gamma BN_\gamma + cN_c + \gamma DN_q]$$

$$q_f = [\frac{1}{2}\gamma BN_\gamma + cN_c + \gamma DN_q]$$

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

N_c, N_q, N_γ = Bearing Capacity factor which are dimensionless depend on angle of shear resistance

$$\boldsymbol{N_q = [\frac{a^2}{2\cos^2(45 + \frac{\varphi}{2})}]}$$

$$a = e^{(\frac{3\pi}{4} - \frac{\varphi}{2})\tan\varphi}$$

$$N_c = (N_q - 1)\cos\varphi$$

$$N\gamma = \frac{1}{2}[\frac{K_\varphi}{\cos\varphi} - 1]\tan\varphi$$

Ultimate bearing capacity,

$$\mathbf{q_f = cNc + \gamma DNq + 0.5\gamma BN\gamma}$$

If the ground is subjected to additional surcharge load q, then

$q_f = cNc + (\gamma D + q)\ Nq + 0.5\gamma BN\gamma$

Net ultimate bearing capacity,

$q_n = cNc + \gamma D\ (Nq - 1) + 0.5\gamma BN\gamma - \gamma D$

$qn = cNc + \gamma D\ (Nq - 1) + 0.5\gamma BN\gamma$

Safe bearing capacity,

$q_s = cNc + \gamma D\ (Nq - 1) + 0.5\gamma BN\gamma 1/F + \gamma D$

Here, F = Factor of safety (usually 3)

c = cohesion

γ = unit weight of soil

D = Depth of foundation

q = Surcharge at the ground level

B = Width of foundation

Nc, Nq, Nγ = Bearing Capacity factors

$$Nc = \cot\varphi\ (Nq - 1)$$

$$Nq = e^{2(3\pi/4 - \varphi/2)}\tan\varphi / [2\ \cos2(45 + \varphi/2)]$$

$$N\gamma = (1/2)\tan\varphi\ (Kpr/\cos2\varphi - 1)$$

Kp=passive pressure coefficient.

$$K_p = \frac{1 + sin\varphi}{1 - sin\varphi}$$

K_p= coefficient of passive earth pressure.

Strip footings: $q_f = c\ Nc + \gamma\ D\ Nq + 0.5\ \gamma\ B\ N\gamma$

Square footings: $q_f = 1.3\ c\ Nc + \gamma\ D\ Nq + 0.4\ \gamma\ B\ N\gamma$

Circular footings: $q_f = 1.3\ c\ Nc + \gamma\ D\ Nq + 0.3\ \gamma\ B\ N\gamma$

Rectangular footing: $q_f = [1 + 0.3\frac{B}{L}]c\ Nc + \gamma\ D\ Nq + [1 - 0.3\frac{B}{L}]\gamma\ B\ N\gamma$

Note:

Local shear failure($\Phi<28^{o}$)) ------ N'c,N'q,N' γ

General shear failure($\Phi>36^{o}$)) ------Nc,Nq,N γ

Terzaghi's Problems:

1.A square footing 2.5 m x 2.5 m is built in a homogeneous bed of sand of unit weight 20 KN/ m^3 and having an angle of shearing resistance of 36^0. The depth of the base of footing is 1.5 m below the ground surface. Find the safe load that can be carried by a footing with a factor of safety of 3 against complete shear failure. Use Terzaghi's analysis.

Given Data;

L=B=2.5m

D=1.5m

Unit weight(γ)=20KN/m^3

Φ=36^0

C=0

FOS=3

From the graph

N_c`=60 ,$N_q$`=42 ,N_γ'=50

To find:

Safe load=?

Solution:

Here Φ=36^0 therefore it is general shear failure. The equation can be written as

Bearing capacity of soil,

$$q_f = [cN_c' + \gamma D N_q' + 0.4\gamma B N_\gamma']$$

$$q_f = [(ox60) + (20x1.5x42) + (0.4x20x2.5x50)]$$

$$= 2260 \text{ KN/m}^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 2260 - (20x1.5)$$

$=2230$ KN/m^2

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{q_{nf}}{F} + \gamma D$$

$$q_s = \frac{2230}{3} + 20x1.5$$

$=743.33+30$

$=773.3$KN/m^2

$$q_s = \frac{load}{Area}$$

$$safe\ load(W) = q_s x Area$$

Area=B^2=(2.5)^2=6.25m^2

W=773.3x6.25=4833.3KN

2. A square footing located at a depth of 1.5 m below the ground surface in Cohesion less soil carries a column load of 1280 kN. The soil is submerged having an effective unit weight of 11.5 kN/m3 and an angle of shearing resistance of 30^0. Show and find the size of the footing for Fos = 3 by Terzaghi's theory of general shear failure.

Given Data;

B=?

D=1.5m

Unit weight(γ)=11.5KN/m^3

$\Phi=30^0$

Cohesion less,C=0

Load=1280KN

From the graph

N_c=37.2 ,N_q= 22.5 ,N_γ=19.7

To find:

Size of footing (B)=?

Solution:

Bearing capacity of soil,

$$q_f = [cN_c + \gamma DN_q + 0.4\gamma BN_\gamma]$$

$q_f =$ [0x37.2+11.5x1.5x22.5+0.4x11.5xBx19.7]

$$q_f = 388.125 + 90.62B$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 388.125 + 90.62B - (11.5x1.5)$$

$$q_{nf} = 388.125 + 90.62B - 17.25$$

$$q_{nf} = 370.875 + 90.62B$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{370.875 + 90.62B}{3} + (11.5x1.5)$$

$$q_s = \frac{370.875 + 90.62B}{3} + 17.25$$

$$q_s = \frac{370.875 + 90.62B + (3x17.25)}{3}$$

$$q_s = \frac{370.875 + 90.62B + 51.75}{3}$$

$$q_s = \frac{422.625 + 90.62B}{3}$$

$$q_s = \frac{load}{Area}$$

$$safe\ load(W) = q_s x Area$$

$$safe\ load(W) = q_s x B^2$$

$$1280 = \frac{422.625 + 90.62B}{3} x B^2$$

$$3840 = (422.65 + 90.62B)B^2$$

$$3840 = 422.65B^2 + 90.62B^3$$

$$90.62B^3 + 422.65B^2 + 0B - 3840 = 0$$

B=2.44m

Size of footing=2.44x2.44m

3.A rectangular footing (2x3m) rest on a C-φ soil which is base and 1.5m below the ground surface. Calculate the safe bearing capacity using FOS=3, C=10KN/m^3, φ=30 degree. Nc = 31.2, Nq = 22.5 and Nγ = 19.7 and also soil has following properties voids ratio=0.55, degree of saturation=50%, specific gravity=2.67.

Given data:

B=2m

L=3m

D=1.5m

FOS=3

C=10KN/m^3

φ=30

Nc = 31.2, Nq = 22.5 and Nγ = 19.7

voids ratio(e)=0.55,

degree of saturation(S_r)=50%=0.5

specific gravity(G)=2.67

To find:

Safe bearing capacity(q_s)=?

Solution:

$$\gamma = \frac{(G + eS_r)\gamma_w}{1 + e}$$

$$\gamma = \frac{(2.67 + 0.55x0.5)9.81}{1 + 0.55}$$

$$\gamma = 18.639 KN/m^3$$

Bearing capacity of soil,

$$q_f = [(1 + 0.3)\frac{B}{L}cN_c + \gamma DN_q + (1 - 0.3)\frac{B}{L}x0.5\gamma BN_\gamma]$$

$$q_f = [(1 + 0.3)\frac{2}{3}10x31.2 + 18.639x1.5x22.5 + (1 - 0.3)\frac{2}{3}x0.5x18.639x2x19.7]$$

$$q_f = 1158.82 KN/m^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 1158.82 - (18.639x1.5)$$

$$q_{nf} = 1130.86 KN/m^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{1130.86}{3} + (18.639x1.5)$$

$$q_s = 404.90 KN/m^2$$

Effect of water table:

Ultimate bearing capacity with the effect of water table,

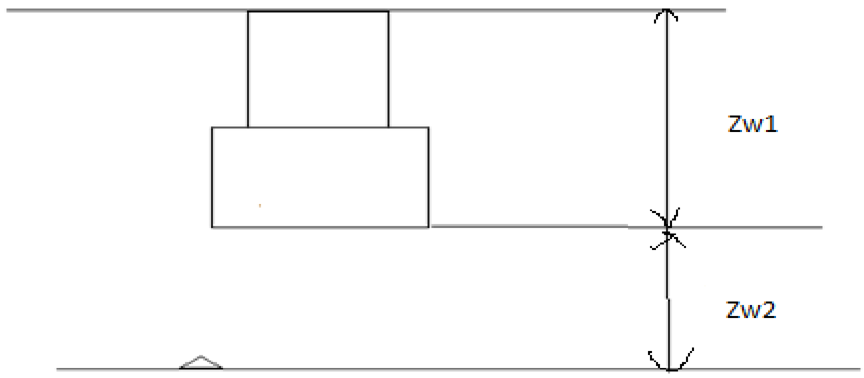
$$q_f = cNc + \gamma DNq R_{w1} + 0.5\gamma BN\gamma R_{w2}$$

$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

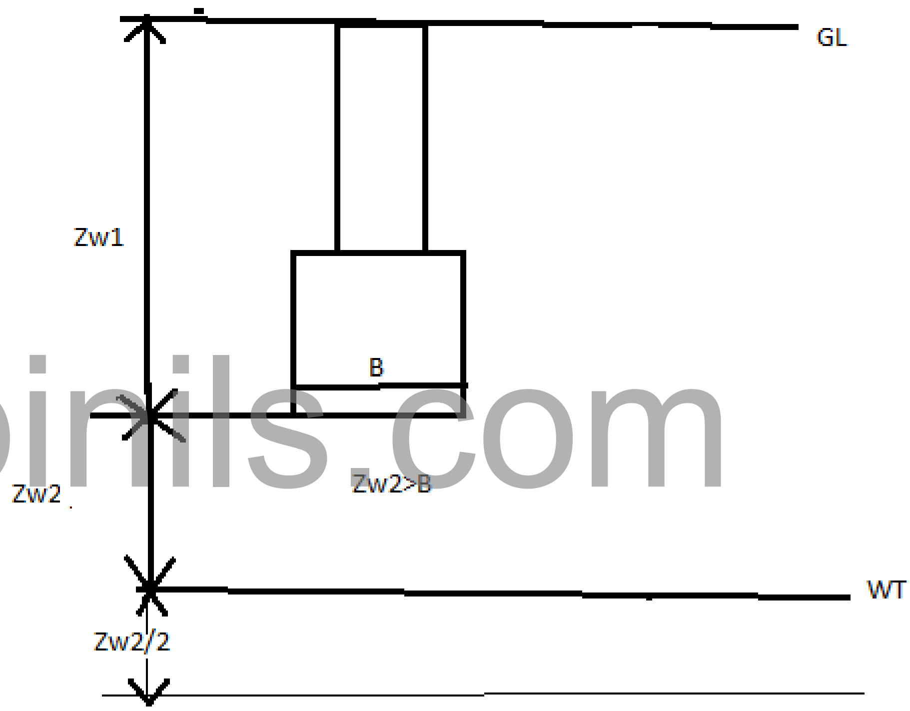
$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}$$

Case1: water level Below the Footing:

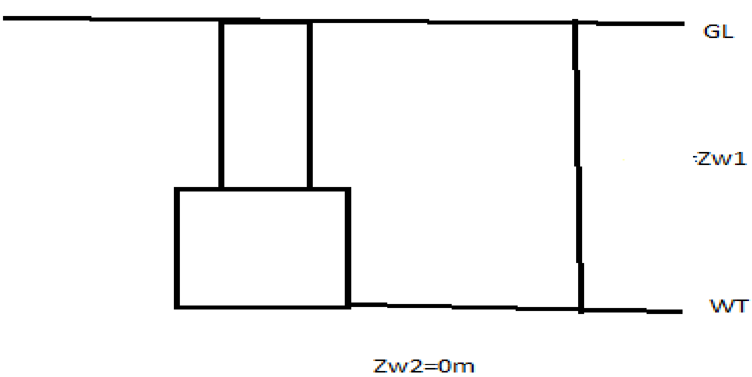
a)Z_{w2}<B



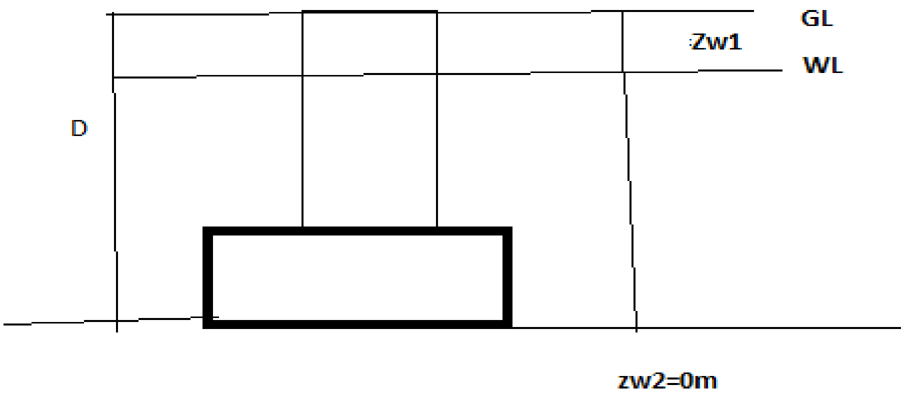
b)Z_{w2}>B



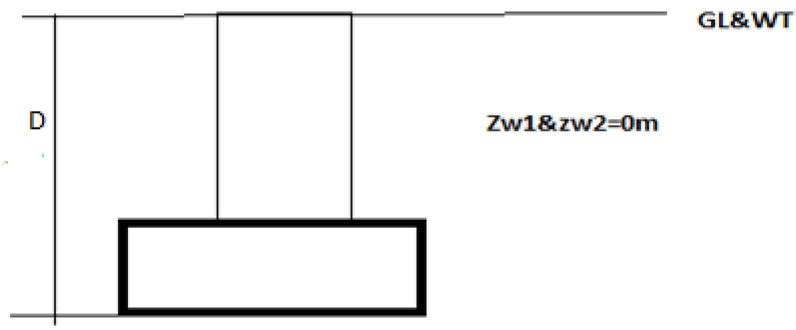
Case2: water level at the base of Footing:



Case3: water level above the Footing:



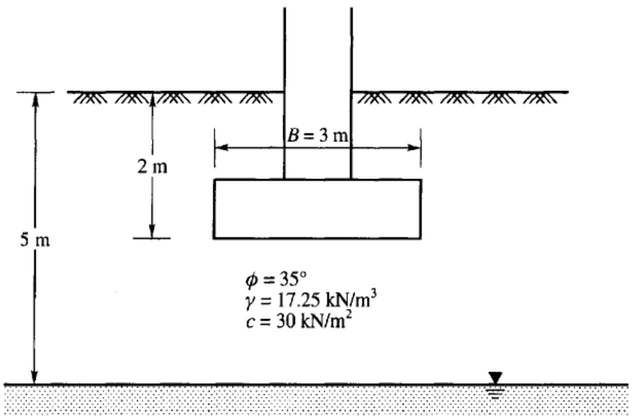
Case4: water level at Ground level:



Terzaghi's Problem with water table:

1.A strip footing of width 3 m is founded at a depth of 2 m below the ground surface in a (c - ϕ) soil having a cohesion c = 30 kN/m^2 and angle of shearing resistance ϕ = 35°. The water table is at a depth of 5 m below ground level. The moist weight of soil above the water table is 17.25 kN/m^3.

Determine (a) the ultimate bearing capacity of the soil, (b) the net bearing capacity, and (c) the net allowable bearing pressure and the load/m for a factor of safety of 3. Use the general shear failure theory of Terzaghi.



Given data:

strip foundation

Width=3 m

Depth of foundation D =2m

$\emptyset=35^0$

$C=30KN/m^3$

$\gamma=17.25KN/m^3$

Fos=3

Nc = 57.8, Nq = 41.4 and Nγ = 42.4

To find:

(a) the ultimate bearing capacity of the soil,

(b) the net bearing capacity, and

(c) the net allowable bearing pressure and the load/m

Solution:

$$q_f = [\frac{2}{3}cN_c + \gamma DN_q R_{w1} + 0.5\gamma BN_\gamma R_{w2}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{2}{2}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL=2m

Zw2= depth of foundation to water level=5-2=3m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{3}{3}]$$

$$R_{w2} = 1m$$

$$q_f = [((\frac{2}{3})x30x57.8) + (17.25x2x41.4x1) + (0.5x17.25x3x42.2x1.25)]$$

$$q_f = [1156.6 + 1428.3 + 1097.1]$$

$q_f = 3681KN/m^2$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 3681 - (17.25x2)$$

$$q_{nf} = 3681 - 34.5 = 3646.5 KN/m^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{3646.5}{3} + (17.25x2)$$

$$q_s = 1250 \ KN/m^2$$

2.A square foundation of size 1.8m×1.8m is to be built at a depth of 1.6m on a uniform clay strata having the following properties'=0^0,c=30KN/$m^3$and γ=18.2 KN/m^3.Find the safe load that the foundation can carry with a factor of safety of 3. Use Terzaghi's bearing capacity theory. If the ground water table subsequently rises from depth of 6m to the ground surface, find the load carrying capacity of the foundation. The submerged density of the soil is 10.5 KN/m^3.

Given data:

Square foundation size= 1.8m× 1.8m

Depth of foundation =1.6m

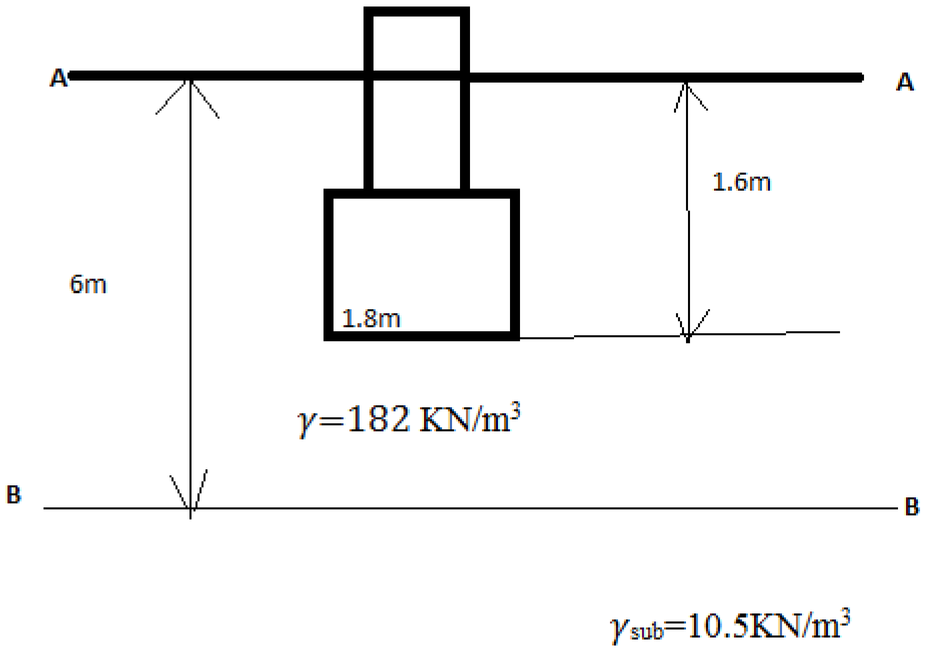
∅=0

C=30KN/m^3

γ=18.2KN/m^3

Fos=3

γ_{sub}=10.5KN/$m^{3.}$



Tofind:

i) case–i: water table at 6m from G.L. safe load=?

ii) case–ii: water table at the ground surface safe load=?

Solution:

Case (1):water table at 6m from ground surface.

Bearing capacity of soil

$$q_f = [cN_c + \gamma D N_q R_{w1} + 0.4\gamma_{avg} B N_\gamma R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{1.6}{1.6}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL=1.6m

Zw2= depth of foundation to water level=6-1.6=4.4m, Zw2>B

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{4.4}{1.8}]$$

$$R_{w2} = 1.72m$$

For Ø=0[*Terzaghi'sBearing capacity factor*]

N_C=5.7,N_q=1.0,N_r=0

$$\gamma_{avg} = \frac{(182x4.4) + [(\frac{4.4}{2}) x10.5]}{[4.4 + \frac{4.4}{2}]}$$

$$\gamma_{avg} = 124.8KN/m^3$$

Bearing capacity of soil

$$q_f = [cN_c + \gamma DN_qR_{w1} + 0.4\gamma_{avg}BN_\gamma R_{w2}]$$

$$q_f = [(30x5.7) + (182x1.6x1x1) + (0.4x124.8x1.8x0x1.875)]$$

$$q_f = 462.2KN/m^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = 462.2 - (182x1.6)$$

$$q_{nf} = 171KN/m^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{171}{3} + (182x1.6)$$

$$q_s = 348.2KN/m^2$$

$$q_s = \frac{load}{Area}$$

$$safe\ load(W) = q_s xArea$$

$$safe\ load(W) = q_s xB^2$$

$$safe\ load(W) = 348.2x(1.8)^2$$

W=1128.2KN

Case (2)if water table at the ground surface

Bearing capacity of soil

$$q_f = [cN_c + \gamma D N_q R_{w1} + 0.4\gamma B N_\gamma R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$R_{w1} = \frac{1}{2}[1 + \frac{0}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{0}{1.6}]$$

$$R_{w1} = 0.5m$$

Zw1= depth of foundation from GL=0m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{1.8}]$$

$$R_{w2} = 0.5m$$

$$q_f = [cN_c + \gamma D N_q R_{w1} + 0.4\gamma B N_\gamma R_{w2}]$$

$$q_f = [(30x5.7) + (182x1.6x1x0.5) + (0.4x182x1.8x0.5)]$$

q_f=[171+145.6+65.52]

q_f=391.12KN/m^2

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 391.12 - (182x1.6)$$

=99.92KN/m^2

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{99.92}{3} + (182x1.6)$$

$$q_s = 324.5KN/m^2$$

$$\text{safe load}(W) = q_s x\text{Area}$$

$$\text{safe load}(W) = q_s xB^2$$

$$\text{safe load}(W) = 324.5x(1.8)^2$$

W=1051.38KN

3. Afoundation,2.0 m square of depth 1.2 m is installed 1.2m Above the water table and a submerged density of $10kN/m^3$. The strength parameters with respect to effective stress c'=0 and φ=300.Find the gross ultimate bearing capacity for the following conditions.

1. Water table is 1.2m below the base of the foundation.
2. Water table raise to the level of the base of the foundation and
3. The water table rise to ground level. (For $\varphi = 30^0$, Assume $N_q = 22$ and $N_\gamma = 20$).

Solution:

Square footing(2m×2m)

$C = 0$, $\varphi = 30^0$

$N_q = 22$, $N_r = 20$

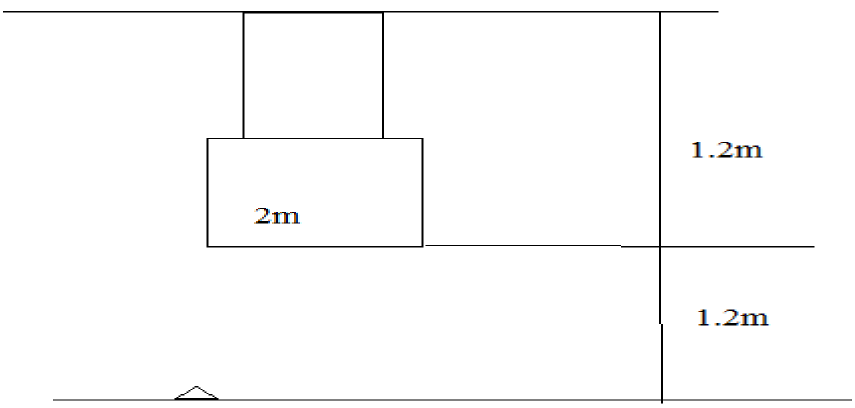
$\gamma_{sub} = 10kN/m^3$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$\gamma_{sat} = \gamma_{sub} + \gamma_w$$

$$\gamma_{sat} = 10 + 9.81 = 19.81KN/m^3$$

i) Water table is 1.2m below the base of the foundation:



$$q_f=1.3\,cN_c+\gamma D\,N_qRw_1+0.4\gamma BN_rRw_2$$

Bearing capacity of soil

$$q_f=[cN_c+\gamma DN_qR_{w1}+0.4\gamma_{avg}BN_\gamma R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf}=q_f-\bar{\sigma}$$

$$q_{nf}=q_f-\gamma D$$

Safe Bearing capacity of soil,

$$q_s=\frac{q_{nf}}{F}+\bar{\sigma}$$

$$R_{w1}=\frac{1}{2}[1+\frac{Z_{w1}}{D}]$$

$$R_{w1}=\frac{1}{2}[1+\frac{1.2}{1.2}]$$

$$R_{w1}=1m$$

Zw1= depth of foundation from GL=1.2 m

Zw2= depth of foundation to water level=1.2

$$R_{w2}=\frac{1}{2}[1+\frac{Z_{w2}}{B}]$$

$$R_{w2}=\frac{1}{2}[1+\frac{1.2}{2}]$$

$$R_{w2}=0.8m$$

$$q_f=1.3\,cN_c+\gamma D\,N_qRw_1+0.4\gamma BN_rRw_2$$

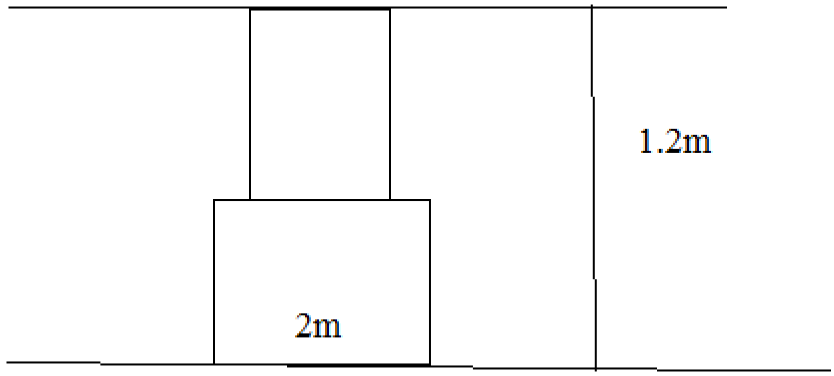
$$q_f=0+(19.8\times1.2\times22\times1)+(0.4\times19.8\times2\times20\times0.8)$$

$$q_f=776.16\ kN/m^2$$

Net ultimate bearing capacity(q_{nf})

$$q_{nf}=q_f-\gamma D=776.16-(19.8\times1.2)=752.4kN/m^2$$

ii) Water table at base of the foundation:



$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{1.2}{1.2}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL=1.2 m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{2}]$$

$$R_{w2} = 0.5m$$

q_f=1.3 cN_c +γD N_qRw_1+0.4γ_{sub} $BN_rR\ w_2$

q_f=0 +(19.8 ×1.2×22×1) +(0.4 ×10 ×2×20 ×0.5)

q_f= 602.72kN/m^2

q_{nf}=q_f-γD =602.72 – (19.8×1.2) = 578.96 kN/m^2

iii) water table rises the ground level

$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{0}{1.2}]$$

$$R_{w1} = 0.5m$$

Zw1= depth of foundation from GL=0 m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{2}]$$

$$R_{w2} = 0.5m$$

$$\gamma=\gamma_{sub}$$

q_f=1.3 cN_c +γ_{sub}D $N_q Rw_1$+0.4γ_{sub}BN_r R w_2

q_f=0 +(10 ×1.2×22×1) +(0.4 ×10 ×2×20 ×0.5)

q_f=344 kN/m^2

q_{nf}=q_f-γ_{sub}D

=344-10 x1.2 =332 kN/m^2

4. A footing 2.m square carries a gross pressure of 350kN/m^2 at a depth of 1.2m in sand. A saturated unit weight of sand is 20 kN/m^2 and the unit weight of sand above water table is 16 kN/m^3. The shear strength parameters are C' =0, $\emptyset$ = $\mathbf{30°}$ (for $\emptyset$ = $\mathbf{30°}$,N_q=22, N_γ=20). Determine the factor of safety with respect to shear failure for the following cases

i) W.T is 5mbelowtheground level

ii) W.T is1.2mbelow the ground level solution:

We will follow IS code method and terazaghi

For square footing in soil having c=0 q_f=$\bar{\sigma}$Nq+0.4γBNγW'

casei): W.T at 5 m below G.L

$\bar{\sigma}$=16x1.2

=19.2kN/m^2

D_W=5m

D+B =3+1.2=4.2m

Since D_W>(D+B),W''=1

Alsoγ=16kN/m^2

$$q_f = \bar{\sigma}N_q + 0.4B\gamma N_\gamma W''$$

=19.2 x22 +0.4 x16x3 x20x1

$=806.4KN/m^2$

$q_{nf}=q_f-\gamma D$

=806.4– 16 x1.2

$=787.2kN/m^2$

Safe bearing capacity,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

=787.2/F+16 x1.2

350=787.2/F+19.2F=2.38

Case ii): water table at 1.2m below the G.L

$D_w=D \Rightarrow W''=0.5$

$\gamma=\gamma_{sat}=20kN/m^3$

$\sigma=16\times1.2=19.2\ kN/m^2$

$$q_f = \bar{\sigma} N_q + 0.4 B\gamma N_\gamma W''$$

=19.2x22+0.4x(20-9.81)x3x20

$q_f = 666.96\ kN/m^2$

$q_{nf}=q_f-\gamma D$

=666.96-16x1.2

$=647.76kN/m^2$

Safe bearing capacity

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

350=67.76/F+19.2

F=1.96

5.A circular footing is resting on a stiff saturated clay with unconfined compression strength of 250 kN/m^2. The depth of foundation is 2m. Determine the diameter of the footing if the column load is 700 KN.

Assume a factor of safety as 2.5. the bulk unit weight of soil is20KN /m^3. What will be the change in ultimate,net ultimate and safe bearing capacity if the water table is at ground level ?

For stiff saturated clay, Ø=0

N_c=5.7,N_q=1and N_γ=0

q_u=250KN/m^2

$$C = \frac{q_u}{2}$$

∵c=250/2= 125 KN/m^2

q_f=1.3 $cN_r+\gamma DN_r+0.4\gamma BN_r$

=966KN/m^2

q_{nf}= $q_f-\gamma D$

=966-20 x2 = 926 KN/m^2

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

=926/2.5+40

=410.4KN/m^2

W =q_sxA

$700=410.4x\frac{\pi x d^2}{4}$

$$d = \sqrt{\frac{4x700}{\pi x 410.4}} = 1.47m$$

q_{nf}=1.3$cN_c+\gamma'' DN_q$

=1.3x125x5.7 +10 x2x1

=946.25KN/m^2

$q_n = 946.25 - 20$

$= 926.25\ KN/m^2$

$q_s = 526.25/2.5 = 390.5 KN/m^2$

6.A strip footing 2 m wide carries a load intensity of $400KN/m^2$ at a depth of 1.2 m on sand. A saturated unit weight of sand is $19.5KN/m^3$ and unit weight above water table is $16.8\ KN/m^3$ .The shear strength parameter $C=0,\varphi=36^0$,Determine the factor of safety for a following condition.

1)WT below 4m from GL

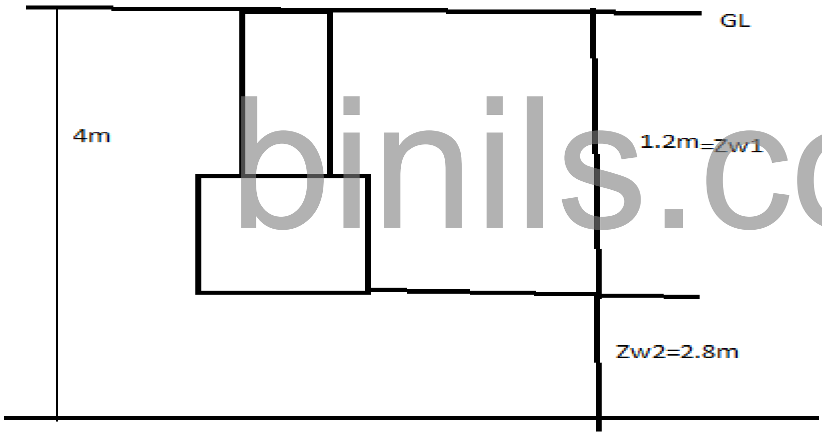
2)WT 1.2 m from GL

3) WT 2.5 m from GL

4) WT 0.5 m from GL

5) WT at GL

1)WT below 4m from GL



$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{1.2}{1.2}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL= 1.2m

Zw2= depth of foundation to water level=2.8m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{2.8}{2}]$$

$$R_{w2} = 1.2\ m$$

$$q_f = [\frac{2}{3}cN_c + \gamma DN_qR_{w1} + 0.5\gamma BN_\gamma R_{w2}]$$

$$q_f = [16.8x1.2x40.4x1 + 0.5x16.8x2x33.4x1.2]$$

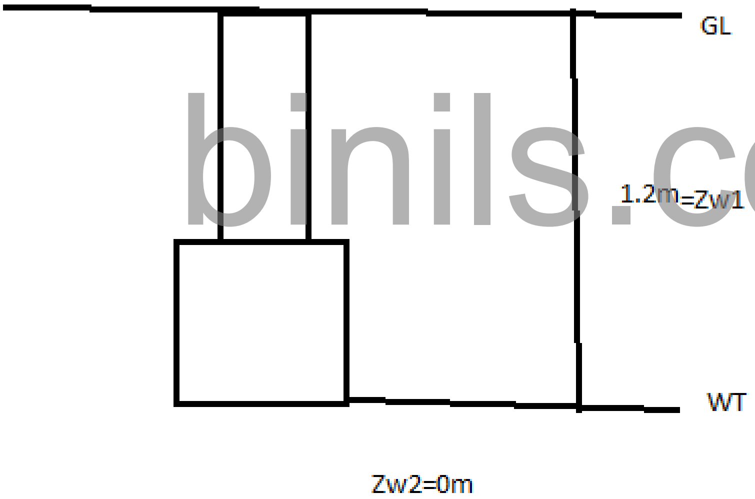
$$= 814.464 + 673.344$$

=1487.8KN/m^2

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1487.8}{400} = 3.7$$

2)WT 1.2 m from GL



$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{1.2}{1.2}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL= 1.2m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{2}]$$

$$R_{w2} = 0.5\ m$$

$$q_f = [\frac{2}{3}cN_c + \gamma D N_q R_{w1} + 0.5\gamma B N_\gamma R_{w2}]$$

$$q_f = [\frac{2}{3}cN_c + 16.8x1.2x40.4x1 + 0.5x16.8x2x33.4x0.5]$$

=1095KN/m^2

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1095}{400} = 2.7$$

3) WT 2.5 m from GL

$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{1.2}{1.2}]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL= 1.2m

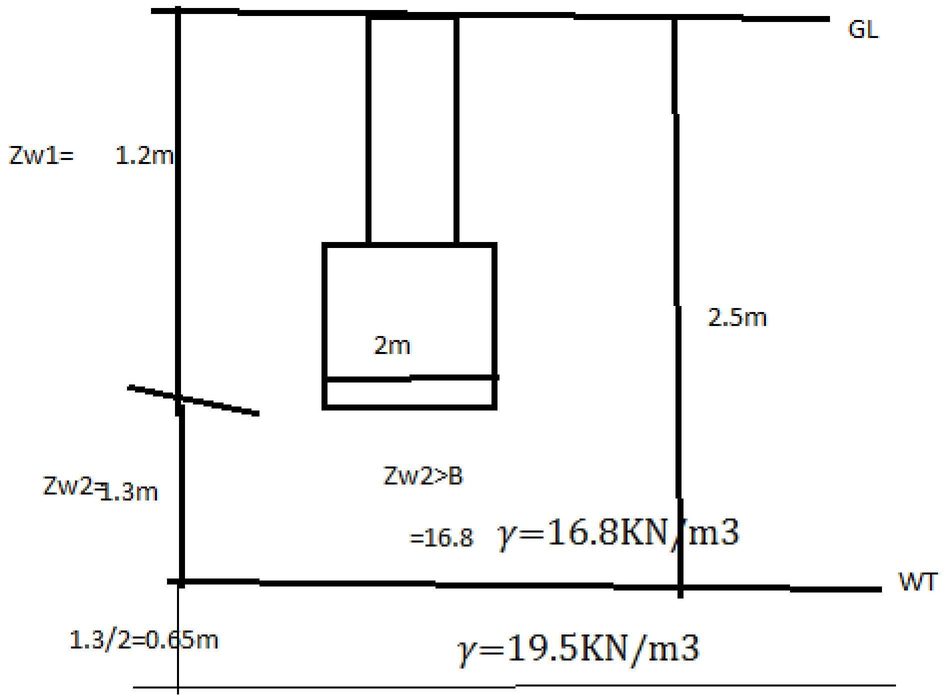
Zw2= depth of foundation to water level=1.3m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{1.3}{2}]$$

$$R_{w2} = 0.82m$$

$$q_f = [\frac{2}{3}cN_c + \gamma D N_q R_{w1} + 0.5\gamma_{avg} B N_\gamma R_{w2}]$$



$$\gamma_{avg} = \frac{(1.3x16.8) + (0.65x19.5)}{(1.3 + 0.65)} = 17.7KN/m^3$$

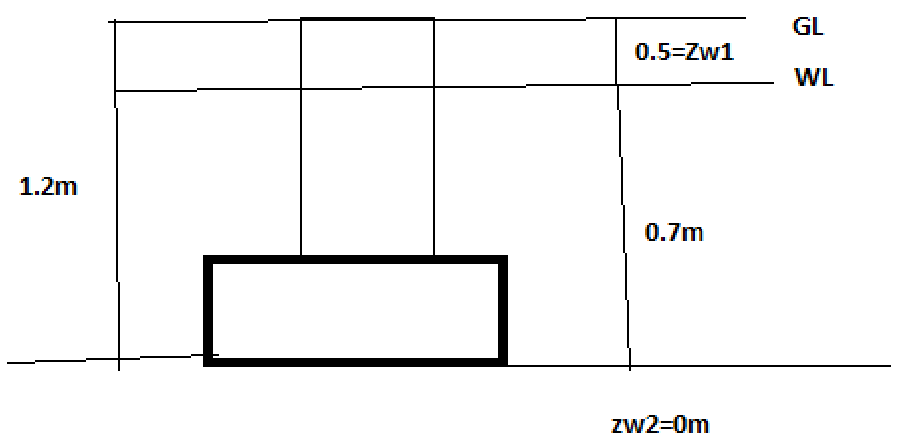
$$q_f = [(18.37x1.2x40.4x1) + (0.5x17.7x2x33.4x0.82)]$$

$$q_f = 1375.3KN/m^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1375.3}{400} = 3.4$$

4) WT 0.5 m from GL



$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{0.5}{1.2}]$$

$$R_{w1} = 0.708m$$

Zw1= depth of foundation from GL= 0.5m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{2}]$$

$$R_{w2} = 0.5\ m$$

$$\gamma_{avg} = \frac{(0.5x16.8) + (0.7x19.5)}{(0.5 + 0.7)} = 18.37KN/m^3$$

$$q_f = [\frac{2}{3}cN_c + \gamma_{avg}DN_qR_{w1} + 0.5\gamma BN_\gamma R_{w2}]$$

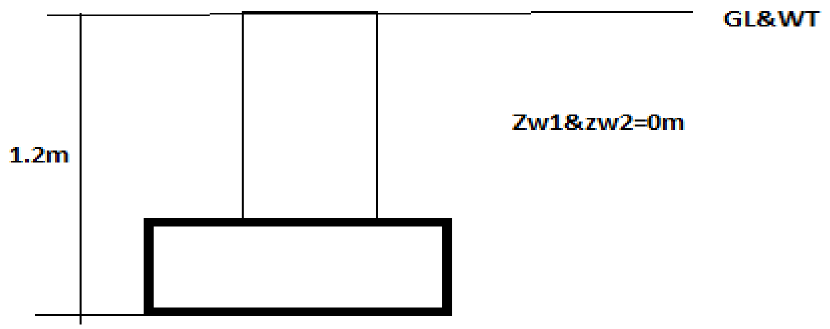
$$q_f = [(18.37x1.2x40.4x0.708) + (0.5x19.5x2x33.4x0.5)]$$

$$q_f = 956.178KN/m^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{956.178}{400} = 2.39$$

5) WT At GL



$$R_{w1} = \frac{1}{2}[1 + \frac{Z_{w1}}{D}]$$

$$R_{w1} = \frac{1}{2}[1 + \frac{0}{1.2}]$$

$$R_{w1} = 0.5m$$

Zw1= depth of foundation from GL= 0m

Zw2= depth of foundation to water level=0m

$$R_{w2} = \frac{1}{2}[1 + \frac{Z_{w2}}{B}]$$

$$R_{w2} = \frac{1}{2}[1 + \frac{0}{2}]$$

$$R_{w2} = 0.5\ m$$

$$q_f = [\frac{2}{3}cN_c + \gamma DN_q R_{w1} + 0.5\gamma BN_\gamma R_{w2}]$$

$$q_f = [(19.5x1.2x40.4x0.5) + (0.5x19.5x2x33.4x0.5)]$$

$$q_f = 798.33KN/m^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{798.33}{400} = 1.99$$

2.4 IS code method:

General shear failure

$$q_f = CN_cS_cd_ci_c + \gamma DN_qS_qd_qi_q + 0.5B\gamma N_\gamma S_\gamma d_\gamma i_\gamma$$

Local shear failure

$$q_f = \frac{2}{3} CN'_cS_cd_ci_c + \gamma DN'_qS_qd_qi_q + 0.5B\gamma N'_\gamma S_\gamma d_\gamma i_\gamma W'$$

S_c, S_q, S_γ= Shape factor

d_c, d_q, d_γ= Depth factor

i_c, i_q, i_γ= inclination factor

W'=water table factor

Shape	**S_c**	**S_q**	**S_γ**
Strip	1	1	1
Rectangle	$1 + 0.2\frac{B}{L}$	$1 + 0.2\frac{B}{L}$	$1 - 0.4\frac{B}{L}$
Square	1.3	1.2	0.8
Circle	1.3	1.2	0.6

For $\varphi = 0, i_c = i_q = i_\gamma$=1

$$i_c = i_q = (1 - \frac{\alpha}{90})^2$$

$$i_\gamma = (1 - \frac{\alpha}{\varphi})^2$$

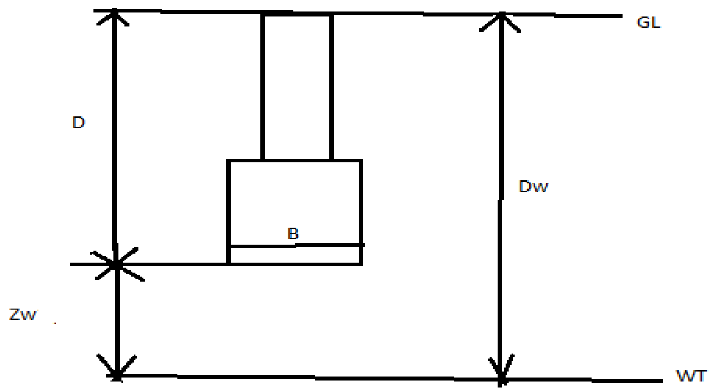
$$N\varphi = \tan^2 (45 + \frac{\varphi}{2})$$

$$\varphi > 10^0, d_q = d_\gamma = 1 + 0.1(\frac{D_f}{B})\sqrt{N\varphi}$$

$$\varphi < 10^0, d_q = d_\gamma = 1$$

$$d_c = 1 + 0.2\frac{D}{B}\sqrt{N\varphi}$$

Effect of water table(W'):



$$W' = 0.5\,[1 + \frac{(D_w - D}{B}]$$

$$W' = 0.5\,[1 + \frac{Z_w}{B}]$$

When Z_w=0,W'=0.5

Z_w=B,W'=1

Table1: Bearing Capacity Factor (Refer IS6403-1981 Page number8)

ϕ (Degrees)	Bearing Capacity Factors N_c	N_q	N_γ
0	5·14	1·00	0·00
5	6·49	1·57	0·45
10	8·35	2·47	1·22
15	10·98	3·94	2·65
20	14·83	6·40	5·39
25	20·72	10·66	10·88
30	30·14	18·40	22·40
35	46·12	33·30	48·03
40	75·31	64·20	109·41
45	138·88	134·88	271·76
50	266·89	319·07	762·89

Problems:

1.A rectangular footing has a size of 1.8mx3m has to transmit the load of a column at a depth of 1.5m. Calculate the safe load which the footing can carry at a factor of safety 3 against shear failures. Use Is code method. The soil has the following Properties.n=40%,G=2.67,w=15%,c=8KN/m^2,Φ=32.5^0

Solution:

$$e = \frac{n}{1-n}$$

$$= \frac{0.4}{1-0.4} = 0.667$$

$$\gamma_d = \frac{G\gamma_w}{1+e}$$

$$= \frac{2.67x9.81}{1+0.667} = 15.71KN/m^3$$

$$\gamma = \gamma_d(1+w)$$

$$=15.71(1+0.15)$$

$$=18.07KN/m^3.$$

For $\Phi=32.5^0$ for Is Method

$N_c=38.3, N_q=25.85$ and $N\gamma=35.21$

$$q_f = CN_cS_cd_ci_c + \gamma DN_qS_qd_qi_q + 0.5B\gamma N_\gamma S_\gamma d_\gamma i_\gamma$$

For Rectangular:

Shape Factor

$$s_c = 1 + 0.2\frac{B}{L}$$

$$= 1 + 0.2\frac{1.8}{3} = 1.12$$

$$s_q = 1 + 0.2\frac{B}{L}$$

$$= 1 + 0.2\frac{1.8}{3} = 1.12$$

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

$$= 1 - 0.4x\frac{1.8}{3} = 0.76$$

Depth factor:

$$\varphi > 10^0, d_q = d_\gamma = 1 + 0.1(\frac{D_f}{B})\sqrt{N\varphi}$$

$$d_c = 1 + 0.2\frac{D}{B}\sqrt{N\varphi}$$

$i_c = i_q = i_\gamma=1$

$N\varphi = \tan^2(45 + \frac{\varphi}{2})$ or $\sqrt{N\varphi} = tan\,(45 + \frac{\varphi}{2})$

$$\sqrt{N\varphi} = tan\,(45 + \frac{32.5}{2}) = 1.823$$

$$d_c = 1 + 0.2\frac{1.5}{1.8}x1.823 = 1.304$$

$$d_q = d_\gamma = 1 + 0.1\,(\frac{1.5}{1.8})\,x1.823 = 1.152$$

$$q_f = 8x38.13x1.12x1.304 + 18.07x1.5x25.85x1.12x1.152$$
$$+ 0.5x18.07x1.8x35.21x0.76x1.152$$

$$= 445.50 + 904.02 + 501.34$$

$$= 1850.86KN/m^2$$

$$q_{nf} = q_f - \gamma D$$

$$= 1850.86 - 18.07x1.5$$

$$= 1823.75KN/m^2$$

$$q_s = \frac{q_{nu}}{F} + \gamma D$$

$$= \frac{1823.75}{3} + (18.07x1.5)$$

$$= 635.02KN/m^2$$

Safe Load= q_s x area

=635.02x(1.8x3)

=3429KN

2.5 Bearing capacity from in-situ tests:

1. **Standard Penetration Test (SPT)**
2. **Cone Penetration Test (cPT)**
3. **Plate load Test**

1. Standard Penetration Test (SPT)

The standard penetration test is an in-situ test that is coming under the category of penetrometer tests. The standard penetration tests are carried out in borehole. The test will measure the resistance of the soil strata to the penetration undergone. A penetration empirical correlation is derived between the soil properties and the penetration resistance.

The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils. It can also be used to determine the unconfined compressive strength of cohesive soils.

The requirements to conduct SPT are:

- Standard Split Spoon Sampler
- Drop Hammer weighing 63.5kg
- Guiding rod
- Drilling Rig.
- Driving head (anvil).

Procedure for Standard Penetration Test

The test is conducted in a bore hole by means of a standard split spoon sampler. Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole.

By means of a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS - 2131:1963.

The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted.

Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm not taken into consideration. The number of blows recorded for last two 150mm intervals are added to give the standard penetration number (N). In other words,

N = No: of blows required for 150mm penetration beyond seating drive of 150mm.

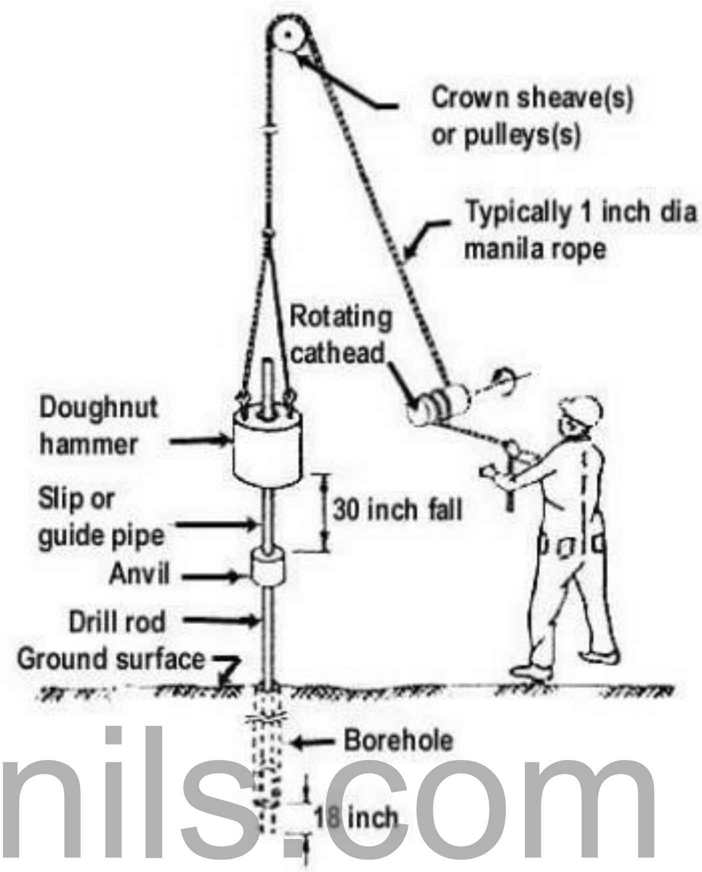


Fig.1: Standard penetration Test

[Fig 1 https://theconstructor.org/geotechnical/standard-penetration-test-procedure-precautions-advantages/4657/]

If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dilatancy correction and overburden correction.

Corrections in Standard Penetration Test:

Before the SPT values are used in empirical correlations and in design charts, the field 'N' value have to be corrected as per IS 2131 – 1981. The corrections are:

1. Dilatancy Correction
2. Overburden Pressure Correction

1. Dilatancy Correction

Silty fine sands and fine sands below the water table develop pore water pressure which is not easily dissipated. The pore pressure increases the resistance of the soil and hence the penetration number (N).

Terzaghi and Peck (1967) recommend the following correction in the case of silty fine sands when the observed value is N exceeds 15.

The corrected penetration number,

$$N_C = 15 + 0.5\ (N_R - 15)$$

Where NR is the recorded value and NC is the corrected value.

If N_R less than or equal to 15, then $N_c = N_R$

2. Overburden Pressure Correction

From several investigations, it is proven that the penetration resistance or the value of N is dependent on the overburden pressure. If there are two granular soils with relative density same, higher 'N' value will be shown by the soil with higher confining pressure.

With the increase in the depth of the soil, the confining pressure also increases. So the value of 'N' at shallow depth and larger depths are underestimated and overestimated respectively.

Hence, to account this the value of 'N' obtained from the test are corrected to a standard effective overburden pressure.

The corrected value of 'N' is

$$Nc = C_N\, N$$

Here C_N is the correction factor for the overburden pressure.

Precautions taken for Standard Penetration Test

- Split spoon sampler must be in good condition.
- The cutting shoe must be free from wear and tear
- The height of fall must be 750mm. Any change from this will affect the 'N' value.
- The drill rods used must be in standard condition. Bent drill rods are not used.
- Before conducting the test, the bottom of the borehole must be cleaned.

Advantages of Standard Penetration Test

The advantages of standard penetration test are:

- The test is simple and economical
- The test provides representative samples for visual inspection, classification tests and for moisture content.
- Actual soil behavior is obtained through SPT values
- The method helps to penetrate dense layers and fills
- Test can be applied for variety of soil conditions

Disadvantages of Standard Penetration Test

- The limitations of standard penetration tests are:
- The results will vary due to any mechanical or operator variability or drilling disturbances.
- Test is costly and time consuming.
- The samples retrieved for testing is disturbed.
- The test results from SPT cannot be reproduced
- The application of SPT in gravels, cobbles and cohesive soils are limited

Cone Penetration Test:

Dynamic Cone Penetrometer, or DCP, is a tool used for evaluating the strength of soils on site. It also helps with monitoring the condition of granular layers and subgrade soils in pavement sections over time. It can be used to determine the right solutions for the sites, especially when soft soils are involved.

It is also applied when the CBR value of compacted soil sub-grade beneath the existing road pavement is to be determined. Continuous readings can be taken down to a depth of 800 mm or, when an extension rod is fitted, to a depth of up to 1200 mm.

The DCP is a simple and portable instrument. It consists of a hardened conical tip, standard diameter steel rod, and a standard weight hammer(8kg), which is dropped from the top of the rod against an anvil to advance the tip into the ground.

Apparatus for DCP

The apparatus of the instrument involves the following parts:

- Handle
- Top Rod
- Hammer(8kg)

- Anvil
- Handguard Cursor
- Bottom Rod
- 1m rule
- 60 degree Cone
- bars and spanners(to ensure that the screwed joints are kept tight at all times)

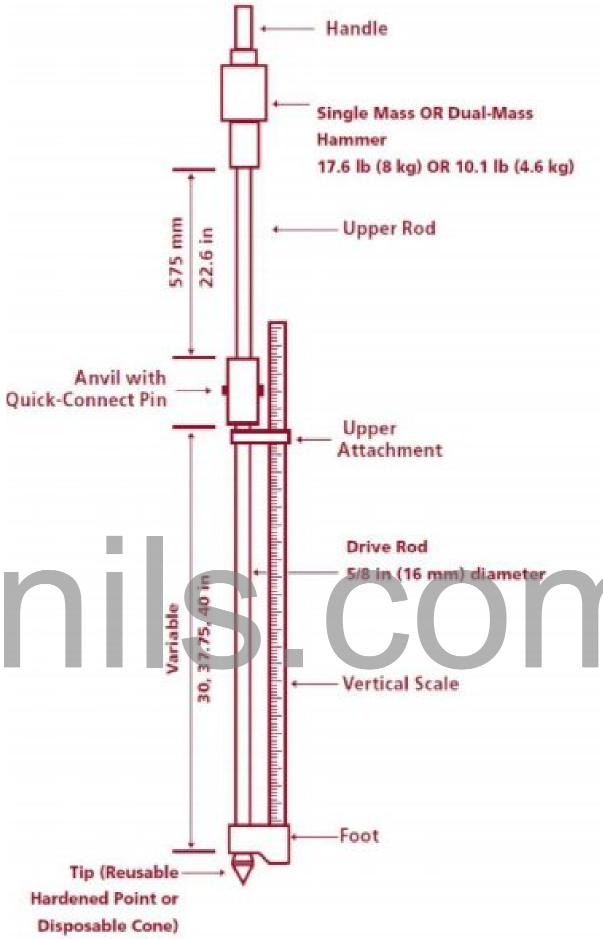


Fig 2 Dynamic Cone Penetrometer

[Fig 2 https://theconstructor.org/geotechnical/soils/what-dynamic-cone-penetrometer/40239/]

The following joints should be secured with a strong adhesive or similar non-hardening thread-locking compound prior to use:

(i) Handle/top rod

(ii) Anvil/bottom rod

(iii) Bottom rod/cone

The hammer is lifted to the top of the rod and released in order to drive the rod into the ground. With the help of the embedded vertical scale, the penetration (in inches or millimeters) is recorded after the blows of the hammer.

Correlations have been established between measurements with California Bearing Ratio (CBR) and DCP so that the outcome can be interpreted and compared with CBR specifications for pavement design.

Procedure

After the instrument is set up, the zero reading of the apparatus is recorded. This is done by placing the DCP on a hard surface, ensuring its verticality, and then noting down the zero reading.

The instrument is held vertical, and the weight is carefully raised to the handle. The weight should not touch the handle before it is allowed to drop, and that the operator should let it fall freely and does not lower it with his hands.

It is advised that a reading should be taken at increments of penetration of about 10mm. However, it is usually easier to take a scale reading after a set number of blows. It is, therefore, necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular bases, readings after every 5 or 10 blows are normally satisfactory, but for weaker sub-base layers and sub-grades, readings after every 1 or 2 blows may be appropriate.

After the completion of the test, DCP is removed by gently tapping the weight upwards against the handle. It should be done with caution as if done vigorously, the life of the instrument will be reduced.

Benefits of DCP:

- Soil information is often limited, and is often collected from within the extents of the foundation area, but one may also need to assess the soils somewhere else on the site.
- Information regarding the variation of soil strength with depth can be obtained, which can be critical for developing the best solution for unsuitable subgrade soils.
- One can collect information from a lot of points relatively quickly, so you can see how soil conditions vary across the site and respond accordingly.
- One gets accurate and precise information on the soil conditions in the field and at construction time.

Advantage:

1. Continuous resistance with depth is recorded
2. Static resistance is more appropriate to determine static properties of soil

Disadvantage:

1. If a small rock piece is encountered resistance shown is erratic and incorrect
2. 3.involves handling heavy equipment

Plate Load Test:

The allowable bearing pressure can be determined by conducting a plate load test at the site. The conduct a plate load test, a pit of the size 5Bp X 5Bp, where Bp is the size of the plate, is excavated to the depth equal to the depth of foundation(D_f).The size of the plate is usually 0.3m square. It is made of steel and is 25mm thick. Occasionally circular plates are also used. Sometimes large size plates of 0.6m square are used.

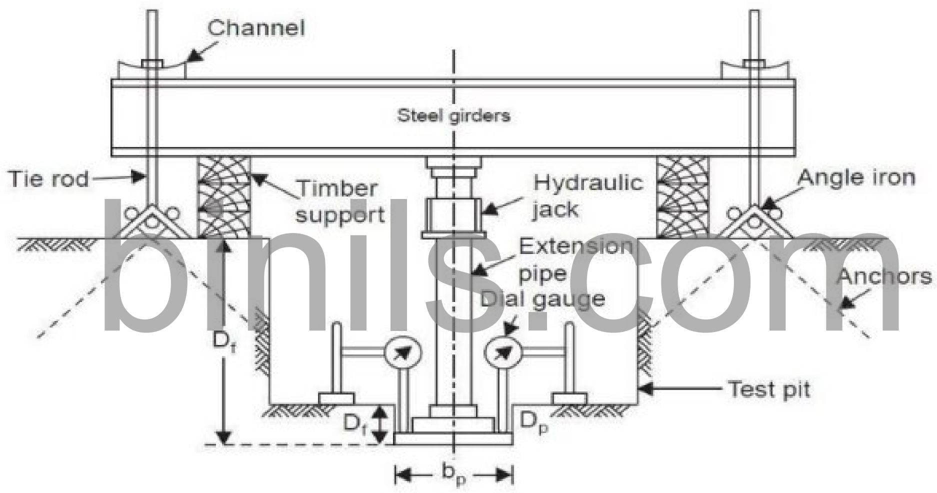


Fig 3 plate load test setup

[Fig3 https://structville.com/2021/03/how-to-determine-the-bearing-capacity-of-soils-from-plate-load-test.html]

A central hole of size Bp X Bp is excavated in the pit the depth of the centralhole(D_p)is obtained from the following relation

$$\frac{D_p}{B_p} = \frac{D_f}{B_f}$$

$$D_p = (D_f/B_f)B_p$$

$$= (B_p/B_f)D_f$$

Where,

B_f-width of the pit

B_p-size of plate

The conducting the plate load test, the plate is placed in the central hole and the load is applied by means of a hydraulic jack. the reaction to the jack is provided by means of a reaction beam. Sometimes truss is used instead of a reaction beam to take up the reaction. Alternatively, a loaded platform can be used to provide reaction.

Varieties in Plate load test and their durations: -

Plate load test is performed under two variations:

1) Gravity load test (Reaction Loading method)

2) Reaction truss method

The total duration required to perform a complete test varies from 6-7 days which includes installations, test, dismantling. The results of the test in case of soft strata can be obtained within a few hours whereas in case of hard strata it might take close to a couple of days.

1. Gravity load test

In this type of method, a rigid platform is utilized to transfer loads through loading of sandbags or concrete blocks. These blocks and sandbags act as a dead weight, and whole arrangement rests upon vertical columns. The hydraulic jack is provided in between the rigid plate and top of the column to transfer the load properly.

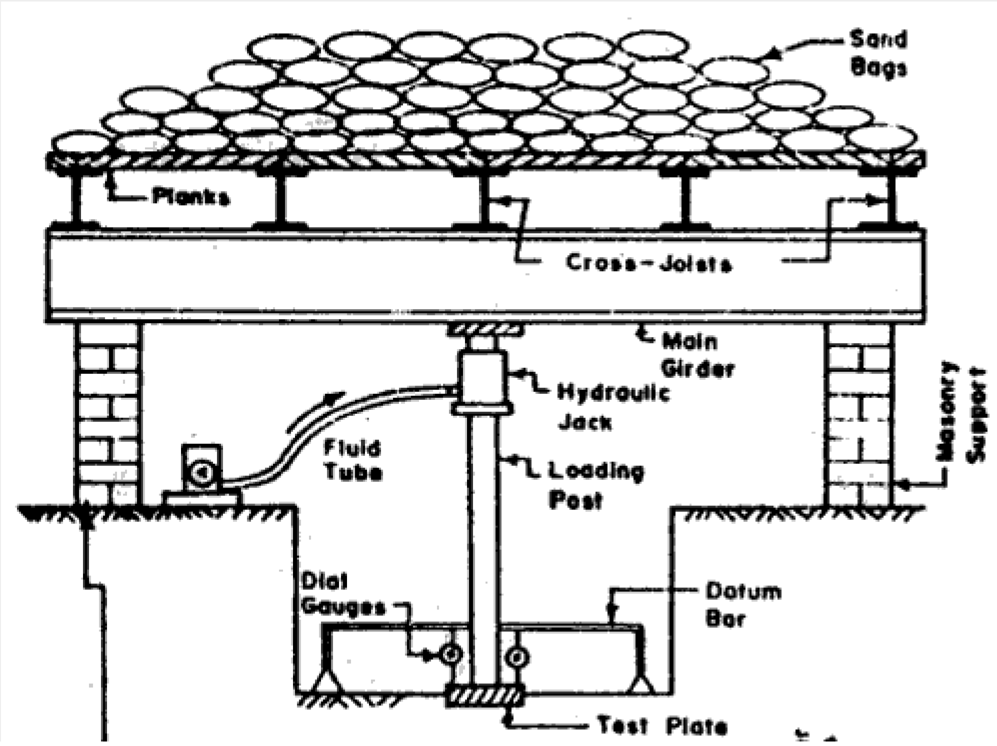


Fig 4 Sand bag method

[Fig4 http://www.abuildersengineer.com/2012/10/plate-load-test-foundation-site.html]

2. Reaction truss method

In this method, the reaction generated through jack is borne by reaction truss installed over it. The undesirable movement of truss is controlled by soil anchors or nails fixed into the soil with the help of hammers. The most commonly observed truss is made of mild steel sections. In order to curb later movement, truss is locked with guy ropes.

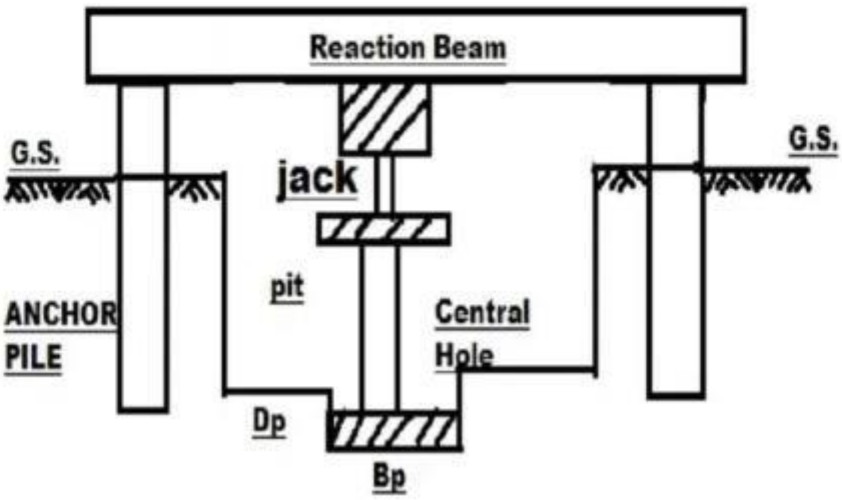


Fig 5Reaction truss method

[Fig5 https://www.ww-99.top/products.aspx?cname=plate+bearing+test&cid=6/]

A seating load of KN/m^2 is first applied, which is released after the sometimes. The is then applied in increments of about 20% of the estimated safe load or $1/10^{th}$ of the ultimate load. The settlement is recorded after 1,5,10,20,40,60 minutes and further after an internal of one hour. These hourly observations are continued for clayey soils, until the rate of settlement is less than 0.2mm per hour. The test is conducted until failure or at least until the settlement of 25mm has occurred

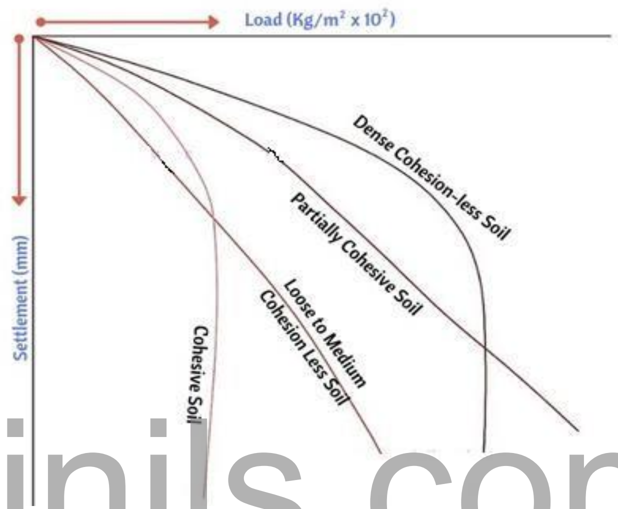


Fig 6 Load settlement Curve

[Fig 6https://civilread.com/plate-load-test/]

The ultimate load for the plate is indicated by a break on the log-log between the load intensity q and the settlements. If the break is not will defined the ultimate load is taken as the corresponding to the settlement of $1/5^{th}$ of the plate width(Bp) on the natural plot. The ultimate load is obtained from the intersection of the tangents drawn

Determination of bearing capacity:

1. The ultimate bearing capacity of the proposed foundation $q_u(f)$ can be obtained fromthe following relations

a) For sandy or gravel soil:

$$q_f = q_p \frac{B}{B_p}$$

b) For clay soil

$$q_f \cong q_p$$

c) For $C - \varphi\ Soil$:

$$Q = q.A + PS$$

where,

B_f –foundation width

B_p –Plate width

$q_f = bearing\ capacity\ of\ foundation$

$q_p = bearing\ capacity\ of\ plate$

Q=Total load

A=Area of footing or plate

P=perimeter of footing or plate

Q=bearing Pressure

S=perimeter area

Determination of settlement:

2.The plate load test can also be used to determine the settlement for a given intensity ofloading (q_0). The relations between the settlement of the plate (s_p)and that of the foundation (s_f) for the same load intensity

a) For clayey soils, $s_f = s_p(B_f/B_p)$ -----------(3)

where s_p is obtained from the load intensity settlement curve for q_0

b) For sandy soils

$$S_f = S_p \left[\frac{B_f(B_p+30)}{B_p(B_f+30)}\right]^2 \text{----------(4)}$$

Where B_f – width of foundation in meters

B_p – width of the plate in meters

3.For designing a shallow foundation for an allowable settlement of s_f,A trial and errorprocedure is adopted. First of all, a value of B_f is assumed and value of q_0 is obtained as

$$q_0 = Q/A_f \text{ ----------- } (5)$$

where A_f - area of footing

Q – Load

For the computed value of q_0 the plate settlement(s_p) is determined from the load – settlement curve obtained from the plate load test the values of s_f is computed equation 3if the soil is clay and using 4 if sand. The computed with the allowable settlement. The procedure is repeated till the computed value is equal to the allowable settlement

The plate load test is can be also be used for the determination of the influence factor I_f,

$$S_i = \left(\frac{1-\mu^2}{E}\right) qBI_f$$

The above graph shows a plot between settlements and the load qB,The slope of the line is equal to $\frac{1-\mu^2}{E}$

LIMITATIONS OF PLATE LOAD TEST:

1. SIZE EFFECT:

The results of the plate load test reflect the strength and the settlement characteristics of the soil within the pressure bulbs. As the pressure bulb depends upon the size of the loaded area it is much deeper for the actual foundation as compared to that of plate. The plate load test does not truly represent the actual conditions to a large depth.

2. SCALE EFFECT:

The ultimate bearing capacity of saturated clays is independent of the size of the plate but for cohesionless soils. It increases with the size of the plate to reduce scale effect, it is desirable to repeat the plate load test with plates of two or three different sizes and the average of the bearing capacity values obtained.

3. TIME EFFECT:

A plate load test is essentially a test of short duration for clayey soils it does not give the ultimate settlement. The load settlement curve is not truly representative.

4. INTERPRETATION OF FAILURE:

The failure load is not well defined except in the case of a general shear failure an error of personal interpretation may be involved in other type of failures

5. REACTION LOAD:

It is not practicable to provide a reaction of more than 250KN.Hence the test on a plate of size larger than 0.6m width is difficult.

6. WATER TABLE:

The level of water table affects the bearing capacity of the sandy soils.If the water table is above the level of the footing it has to be lowered by pumping before placing at the water table level if it is within about 1m below the footing.

Advantages of Plate Load Test:

- Bearing able to evaluate the actions of the base under loading conditions.
- Assessing soil capability at a certain depth and predicting settlement over a certain load.
- A shallow foundation could be determined on the basis of the permissible bearing size, which can be estimated in the context of a plate load test.
- Time and cost-effective
- It’s easy to execute.

Disadvantages of Plate Load Test:

- Depth of impact is small and can hardly offer soil power.
- It does not have details on the prospects for long-term consolidation of the base soil.
- The scale of the test plate is smaller than the real base, hence why there is a scale impact.
- Significant land disturbance happens after drilling is finished.

Problems:

1.The following data were obtained from a plate load test carried out on a 60cm square test plate at a depth of 2m below ground surface on a **sandy** soil which extends upto a large depth. Determine the settlement of foundation 3x3m carrying a load of 1100KN .

Load intensity(KN/m^2)	50	100	150	200	250	300	350	400
Settlement mm	2.0350	4.0	7.5	11.0	16.3	23.5	34.0	45.0

Given data:

B_p=60cm=0.6m

D=2m

sandy soil

B=3m

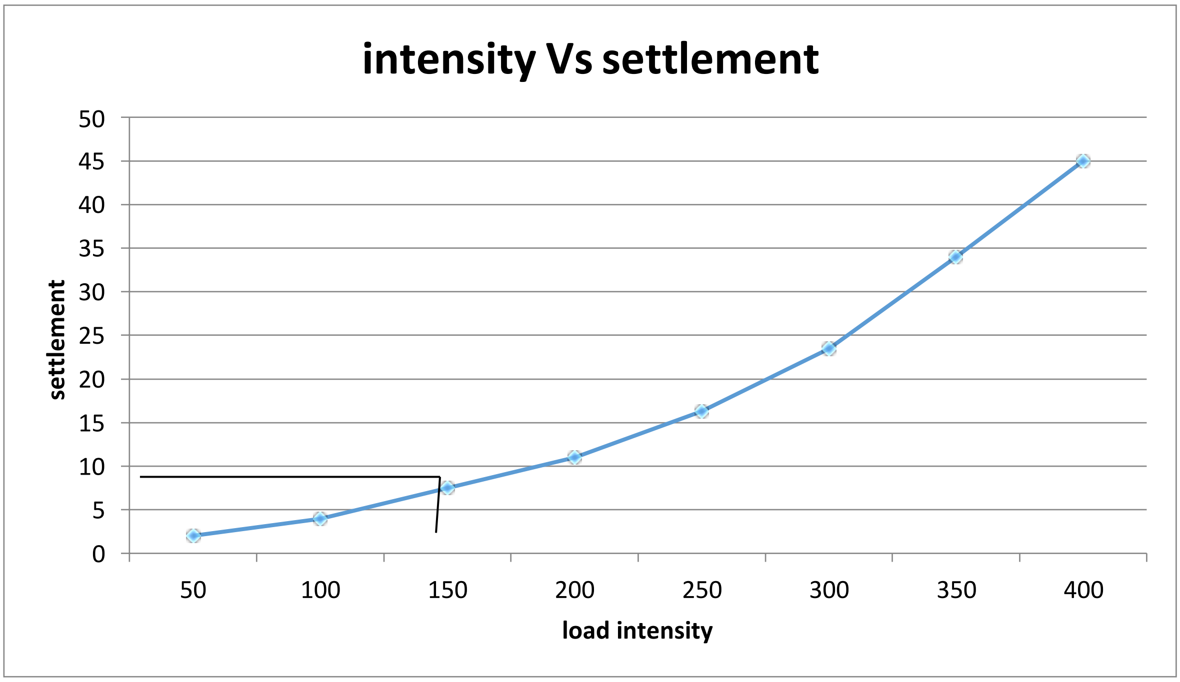
load = 1100KN

To find:

Settlement=?

Solution:

$$intensity = \frac{Load}{area} = \frac{1100}{3^2} = 122.22Kn/m^2$$



For load intensity 122.22 the settlement is 7mm

S_p=7mm

$$S_f = S_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)}\right]^2$$

$$S_f = 7 \left[\frac{3(0.6 + 0.3)}{0.6(3 + 0.3)}\right]^2$$

$$S_f = 13.01\text{mm}$$

2.A plate load test was conducted on a uniform deposit of sand at a depth of 1.5m below the natural ground level and the following data were obtained

Pressure(Kpa)	0	50	100	200	300	400	500
Settlement(mm)	0	2	4.5	10	17	30	50

The size of plate was 600x600mm and that of pit 3mx3mx1.5m

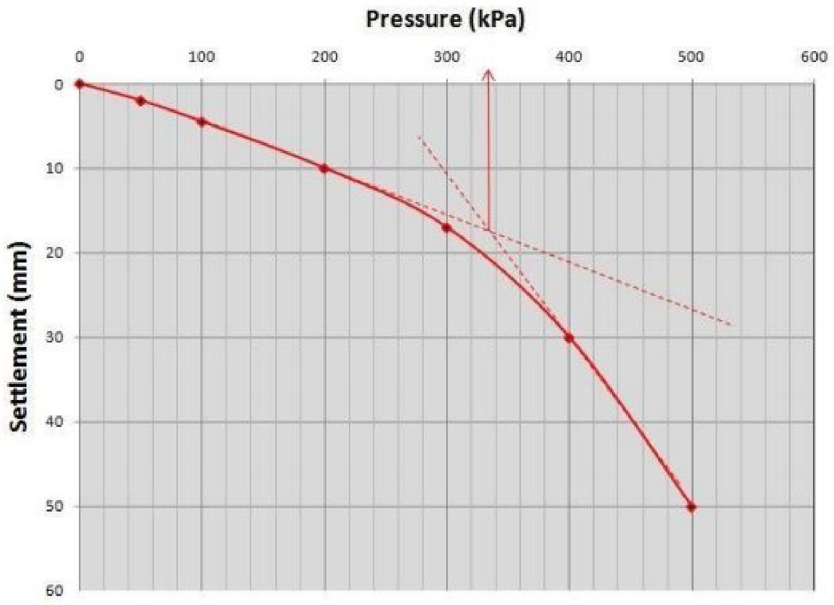
i)Plot the pressure settlement curve and determine the failure stress

ii) A square footing,1.5x1.5m is to be founded at 1.5m depth in this soil. Assuming the FOS against shear failure as 3 and maximum permissible settlement as 25mm Determine the allowable bearing pressure

iii) Design of footing for a load of 600KN if the water table is at a great depth.

Solution:

Draw a graph between load and settlement. The failure is obtained by tangent the line. From the graph the failure pressure is q_p=335KN/m^2.



For sandy or gravel soil:

$$q_f = q_p \frac{B}{B_p}$$

$$= 335x \frac{1.5}{0.6} = 837.5KN/m^2$$

$$q_a = \frac{q_f}{F} = \frac{837.5}{3} = 279.16KN/m^2$$

From settlement consideration:

$$S_f = S_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)}\right]^2$$

$$= 25 \left[\frac{1.5(0.6 + 0.3)}{0.6(1.5 + 0.3)}\right]^2 = 16mm$$

From the load settlement curve, the settlement corresponds to a pressure of 290KN/m^2

$$The\ maximum\ allowable\ service\ column\ load = 1.5\text{x}1.5\text{x}290$$

$$= 652.5\text{KN.}$$

This shows that a column load of 600KN can be safely supported on footing of 1.5x1.5m on the soil

Net allowable bearing pressure:

For settlement 25mm

$$q_p = 35(N - 3)\left(\frac{B + 0.3}{2B}\right)^2 . Rw_2 . R_d$$

For settlement 40mm

$$q_p = 55(N - 3)\left(\frac{B + 0.3}{2B}\right)^2 . Rw_2 . R_d$$

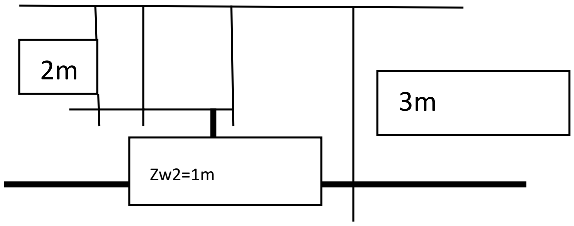
N=standard penetration number

$$Rw_2 = 0.5\left[1 + \frac{Zw_2}{B}\right]$$

$$R_d = depth\ factor = \left[1 + 0.2\frac{D}{B}\right] \leq 1.2$$

1.A strip footing 1.5m wide is located at a depth of 2m over a cohesionless soil.The standard penetration test was conducted having corrected N value of 20.If the depth of water table is 3m below the ground level Then determine the allowable bearing pressure for the soil.

Given data:



strip footing

B=1.5m

D=2m

cohesionless soil

standard penetration test

N=20

To find:

q_a=?

Solution:

$$q_f = 35(N-3)\left(\frac{B+0.3}{2B}\right)^2 .Rw_2.R_d$$

$R_{w2} = 0.5\left[1+\frac{Z_{w2}}{B}\right]$=1.3

$$R_d = depth\ factor = \left[1+0.2\frac{D}{B}\right] \leq 1.2$$

$R_d = \left[1+0.2\frac{2}{1.5}\right] = 1.26$

$$q_f = 35(20-3)\left(\frac{2+0.3}{2x1.5}\right)^2 .1.3x1.26$$

=573.35KN/m^2

2.6 Settlement:

Settlement is the vertical downward movement to the loaded base. As a result of settlement, the original depth of soil mass decrease due to soil grains coming closer together. Uneven settlement leads to cracks. The amount of settlement is different for different type of soil or rock

Types of foundation settlement

- Differential foundation settlement
- Uniform foundation settlement

Differential foundation settlement

- Settlement that occurs at differing rates between different portions of a building is termed differential settlement.
- Differential settlement occurs if there is difference in soils, loads, or structural systems between parts of a building. in this case, different parts of the building structure could settle by substantially different amounts.
- Consequently, the frame of the building may become distorted, floors may slope, walls and glass may crack, and doors and windows may not work properly.
- Uneven foundation settlement may force buildings to shift out of plumb which lead to crack initiation in foundation, structure, or finish.
- Majority of foundation failures are attributable to severe differential settlement.
- Lastly, for conventional buildings with isolated foundations, 20mm differential settlement is acceptable. And 50mm total settlement is tolerable for the same structures.

Uniform foundation settlement:

- when foundation settlement occurs at neraly the same rate throughout all portions of a building, it is called uniform settlement.
- If all parts of a building rest on the same kind of soil, then uniform settlement the most probable type to take place.
- Similarly, when loads on the building and the design of its structural system are uniform throughout, the anticipated settlement would be uniform type.
- Commonly, uniform settlement has small detrimental influence on the building safety.
- However, it influences utility of the building for example damaging sewer; water supply; and mains and jamming doors and windows.

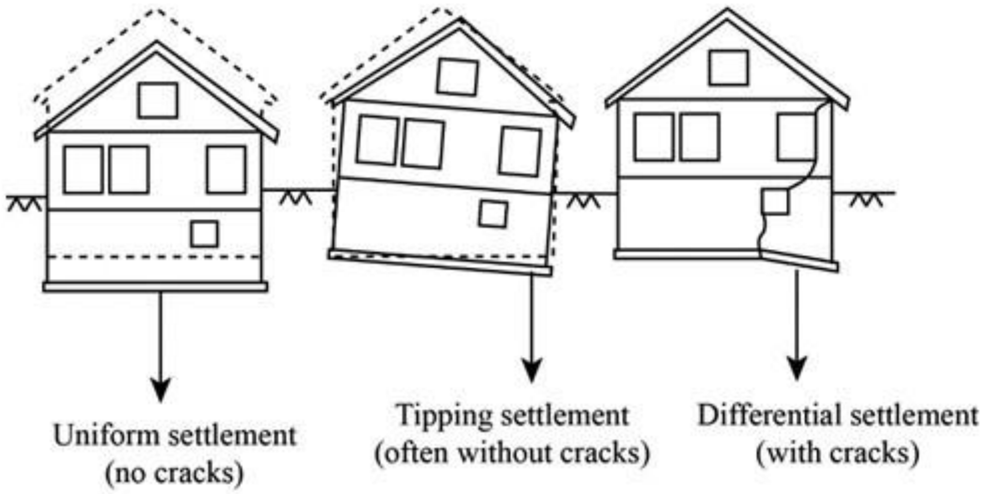


Fig.1: Difference between uniform and differential settlement

[Fig1https://www.chegg.com/homework-help/definitions/settlement-of-structure-8]

Foundation settlement causes

Direct causes

The direct cause of foundation settlement is the weight of building including dead load and live load.

Indirect causes

- Failure of collapsible soil underground infiltration
- Yielding of excavation done adjacent to foundation
- Failure of underground tunnels and mines
- Collapse of cavities of limestones
- Undermining of foundation while flood
- Earthquake induced settlement
- Finally, due to extraction of ground water and oil.

Components of total settlement of foundations

1.Immediate settlement:

- It is also called short term settlement.
- Immediate settlement take place mostly in coarse grained soils of high permeability and in unsaturated fine-grained soils of low permeability.
- Lastly, it occurs over short period of time which about 7 days. So, it ends during construction time.

Cohesive soil:

$$s_i = qB\left[\frac{1-\mu^2}{E_s}\right] I_f$$

Influence factor I_f:

I_f=0.82 for square footing

I_f=0.88 for circular footing

I_f=1.06 for rectangular footing $\frac{L}{B} = 1.5$

I_f=1.7 for square footing $\frac{L}{B} = 5$

Cohesionless Soil:

$$S_i = \frac{H}{C}\log_e\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

C=compressibility

H= depth of stratum

2.Primary settlement

- It also termed as primary consolidation
- Take place over long period of time that ranges from 1 to 5 years or more
- Primary settlement frequently occurs in saturated inorganic fine grain soil.
- Expulsion of water from pores of saturated fine grain soil is the cause of primary settlement.

$$S_c = \frac{HC_c}{1+e_o}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$S_c = m_v \Delta\bar{\sigma} H$$

$$S_c = \frac{\Delta e}{1+e_o} H$$

i)compression index:

$$C_c = \frac{e_0 - e_1}{\log_{10}\left(\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)\right)}$$

$$Or$$

$$C_c = 0.009(w_l - 10)$$

$$W_l = Liquid\ limit$$

$ii)Coefficient\ of\ volume\ change$:

$$m_v = \frac{\Delta e}{\Delta \bar{\sigma}} \cdot \frac{1}{1+e_0}$$

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}}$$

$$C_v = \frac{K}{m_v \gamma_w}$$

$$\bar{\sigma}_0 = over\ burden\ pressure$$

$$\bar{\sigma}_f = final\ stress\ or\ pressure$$

K=permeability

$$\bar{\sigma}_f = \bar{\sigma}_0 + \Delta \bar{\sigma}$$

$$\Delta e = e_0 - e_f$$

e_0= initial voids

e_f=final voids voids

3. Secondary settlement

Secondary settlement is the consolidation of soil under constant effective stress.

Frequently, it occurs in organic fine grain soil.

It continues over the life span of foundation structure similar to creep in concrete.

Total Settlement:

$$\mathbf{S = S_i + S_c + S_s}$$

$\mathbf{S_i}$ **= immediate or elastic settlement**

$\mathbf{S_c}$ **= Primary or consolidation settlement**

$\mathbf{S_s}$ **= secondary settlement**

Causes of settlement are: -

- Uneven bearing capacity of soil at foundation level.
- Different loads on different parts of foundation.
- Varying ground water table height.
- Compressible foundation soil.
- Earthquakes and floods.
- Expansive soil such as black cotton soil.

Various remedial measures:

- Compaction of soil over the complete area at foundation level.

- Proper designs so that large load difference does not exist on different parts of the foundation.
- Dewatering of foundation if ground water table interference with construction of foundation.
- Stabilization of soil of foundation level if it is compressible.
- Special type of foundation for expansive soils such as black cotton soil.
- Consideration of earthquake loads and other earthquake resisting methods during design and construction of buildings.

Problems:

1.A normal consolidated clay layer is 6m thick with a natural water content of 30%of clay has a saturated unit weight of 17.4KN/m^3, specific gravity of 2.67 and liquid limit of 40%. The ground water level is at surface of the clay. Determine the settlement of the foundation. If foundation level will subject to center of a clay layer to a vertical stress increase of 8KN/m^3.

Given data:

W=30%
H=6m
$\gamma_{sat} = 17.4KN/m^3$
$G = 2.67$
W_l=40%
Increase or additional $\Delta\bar{\sigma} = 8KN/m^3$

To find :

Settlement=?

Solution:

$$S_c = \frac{HC_c}{1+e_o}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$C_c = 0.009(w_l - 10)$$

$$\bar{\sigma}_0 = \gamma' Z$$

$$\gamma' or \gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$= 17.4 - 9.81 = 7.59KN/m^3$$

$$Z = \frac{H}{2} = \frac{6}{2} = 3m$$

$$\bar{\sigma}_0 = 7.59x3$$

$$= 22.7KN/m^3$$

$e = \frac{wG}{S_r}$[consider it as fully saturated S_r=1]

$$C_c = 0.009(40 - 10)$$

$$= 0.27$$

$$e = \frac{0.3x2.67}{1} = 0.801$$

$$S_c = \frac{6x0.27}{1 + 0.801} \log_{10}\left(\frac{22.7 + 8}{22.7}\right)$$

$$S_c = 0.117m = 117mm$$

2.A rectangular footing 2mx3m carries a column load of 600KN at a depth of 1m.The footing rest on a $C - \varphi\ soil$ strata 6m thick having Poisson's ratio of 0.25 and young's modulus E=20000KN/m^3.Calculate the immediate Settlement of footing.

Given Data:

B=2m

L=3m

Load=600KN

D=1m

H=6m

$\mu = 0.25$

E=20000KN/m^3

$$C - \varphi\ soil(cohesive\ soil)$$

To find:

immediate Settlement(S_i)=?

Solution:

$$s_i = qB\left[\frac{1 - \mu^2}{E_s}\right] I_f$$

$$q = \frac{load}{area} = \frac{Load}{BxL}$$

$$\frac{L}{B} = \frac{3}{2} = 1.5m$$

I_f=1.06 for rectangular footing$\frac{L}{B} = 1.5$

3.A rectangular footing 1.2mx1.5m rest at a depth of 1m in a saturated clay layer 4m deep the clay is normally consolidated having an unconfined compression strength of $40KN/m^2$. A soil has a liquid limit of 30% water content 23% Determine the load which the footing carrying safe with FOS=3 against shear. Also determine the settlement if the footing is loaded with safe load use Terzaghi analysis $\gamma = 17.8KN/m^2$

Given Data:

B=1.2m

L=1.5m

D=1m

H=4m

normally consolidated

Strength (q)=$40KN/m^2$

W_l=30%

W=23%

FOS=3

Here $\varphi = 0, N_c = 5.7, N_q = 1, N_\gamma = 0$

$$\gamma = 17.8KN/m^2$$

To find:

Load=?

Settlement=?

$$q_f = [1 - 0.3\frac{B}{L}]cN_c + \gamma DN_q + [1 + 0.3\frac{B}{L}]\gamma BN_\gamma]$$

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{Load}{area}$$

$$S_c = \frac{HC_c}{1 + e_o}\log_{10}(\frac{\bar{\sigma_0} + \Delta\bar{\sigma}}{\bar{\sigma_0}})$$

$$C_c = 0.009(w_l - 10)$$

$$\bar{\sigma} = \gamma Z$$

$e = \frac{wG}{S_r}$[consider it as fully saturated S_r=1]

$$\Delta\bar{\sigma} = \frac{maximum\ safe\ load}{area}$$

4.A 30cm square bearing plate settles by 10 mm in the plate load test conducted on sandy soil.The intensity of load applied on the plate causing the settlement is 200KN/m^2.Estimate the possible settlement of a square shaped shallow foundation of side 2m under the same intensity of loading.

Given data:

B_p=30cm=0.3m

S_p=10mm=0.01m

Intensity(q)= 200KN/m^2

B=2m

To find:

S_f=?

Solution:
For sandy or granular soil:

$$S_f = S_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)}\right]^2$$

$$S_f = 0.01 \left[\frac{2(0.3 + 0.3)}{0.3(2 + 0.3)}\right]^2$$

S_f=0.030m

5.The following data were obtained from a plate load test carried out on a 60cm square test plate at a depth of 2m below ground surface on a **sandy** soil which extends upto a large depth. Determine the settlement of foundation 3x3m carrying a load of 1100KN .

Load intensity(KN/m^2)	50	100	150	200	250	300	350	400
Settlement mm	2.0350	4.0	7.5	11.0	16.3	23.5	34.0	45.0

Given data:

B_p=60cm=0.6m

D=2m

sandy soil

B=3m

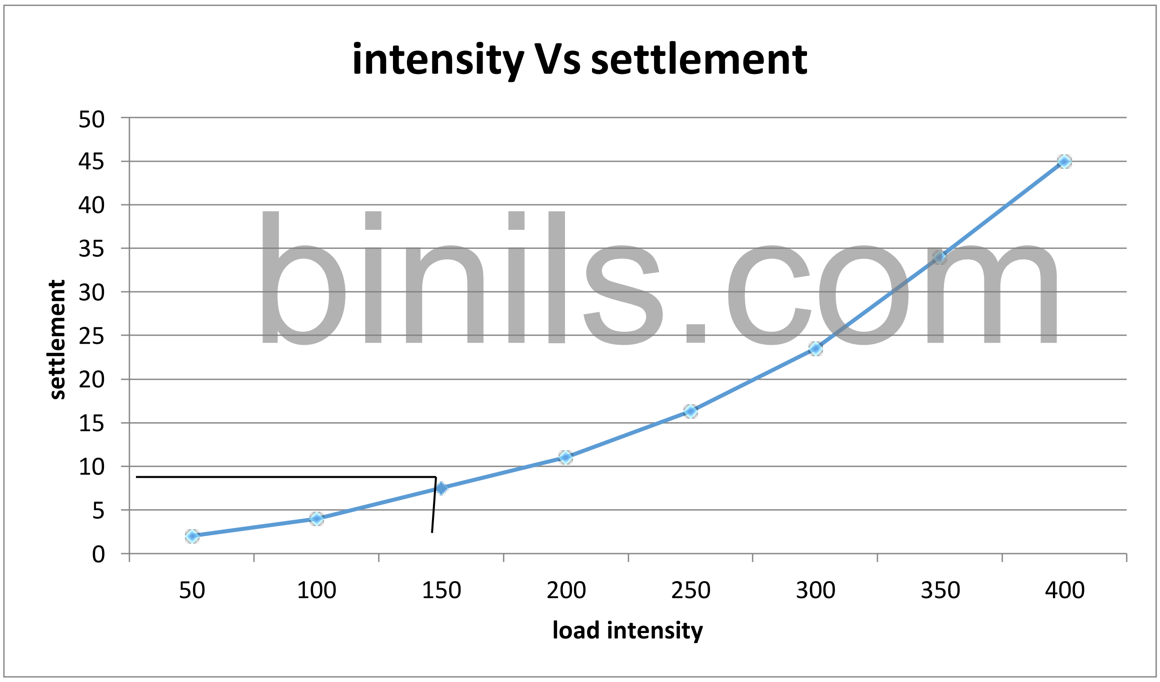
load = 1100KN

To find:

Settlement=?

Solution:

$$intensity = \frac{Load}{area} = \frac{1100}{3^2} = 122.22Kn/m^2$$



For load intensity 122.22 the settlement is 7mm

S_p=7mm

$$S_f = S_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)}\right]^2$$

$$S_f = 7 \left[\frac{3(0.6 + 0.3)}{0.6(3 + 0.3)}\right]^2$$

$$S_f = 13.01\text{mm}$$