Foundation Engineering
Prof. Dr N.K. Samadhiya
Department of Civil Engineering
Indian Institute of Technology Roorkee

Module - 01
Lecture - 07
Shallow Foundation

Welcome to the lecture series on Shallow Foundation again, I am going to continue with the estimation of settlement that is the total settlement. We have discussed the procedures to determines immediate elastic settlement, the consolidated settlement and the settlement for griller soils based on various laboratory and field investigations. But today, I am going to discuss the secondary consolidation settlement and as we know that the secondary consolidation settlement takes place after the consolidation, primary consolidation process is over.

(Refer Slide Time: 01:18)

Another component of total settlement is the secondary consolidation settlement. The secondary (secondary compression or creep) continues much beyond primary consolidation and occurs at a much slower rate. Though there is some doubt regarding the point of time at which secondary consolidation can be said to begin, it is quite clear that it occurs at a constant effective stress and is not associated with the dissipation of excess pore water pressure. One line of thinking is that the secondary compression is due to gradual readjustment of the soil skeleton after the disturbance during primary consolidation. The rate of secondary compression, perhaps, is controlled by the highly viscous adsorbed layer surrounding the clay particles.

So, another component of total settlement is the secondary consolidation settlement, the secondary compression or creep, continues much beyond primary consolidation and occurs at a slower rate, much slower rate than the primary consolidation. Though there is some doubt regarding the point of time at which secondary consolidation can be said to begin. It is quite clear that it occurs at a constant effective stress, and is not associated with the dissipation of pore water pressure.

A one line of thinking is that the secondary compression is due to the gradual readjustment of the soil skeleton after the disturbance during primary consolidation. The rate of secondary compression perhaps, is controlled by the highly viscous adsorbed layer surrounding the clay particles.

(Refer Slide Time: 02:05)

There is now a general agreement that the magnitude of secondary compression can be expressed in terms of the secondary compression index, c_α given by the following equation

$$C_\alpha = \frac{\Delta e}{\Delta \log t}$$

where Δe is the decrease in void ratio in a plot of void ratio *vs.* logarithm of time, corresponding to a time interval of Δt

There is now a general agreement that the magnitude of secondary compression can be expressed in terms of the secondary compression index c alpha, given by the following equation. And c alpha is given as delta e up on delta log of t, where delta e is the decrease in a void ratio in a plot of void ratio versus logarithm of time, corresponding to a time interval of delta t. So, if this delta t is corresponding to time delta t not corresponding to time delta sigma as that is the case of the primary consolidation.
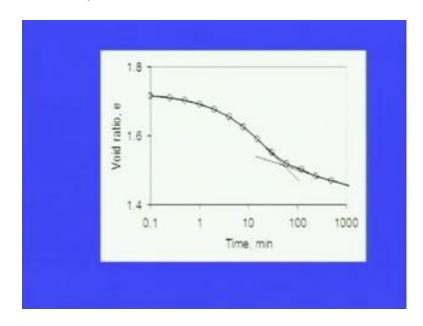
c_α is measured as the slope of the straight line portion of the dial reading vs. logarithm of time curve, which is obtained after the primary consolidation is over. The compression is noted from the plot usually for one log cycle of time, the corresponding Δe then gives the value of c_α. A typical plot is shown in next figure.

C alpha is measured as the slope of the straight line portion of the dial reading versus logarithm of time curve, which is obtained by the primary consolidation. This curve we used to determines the coefficient of consolidation and we find that after the primary consolidation is over there is decrease in the volume of the specimen. And that is primarily due to the secondary compression, the compression is noted from the plot usually for 1 log cycle of time.

So, whatever is the corresponding delta e that is nothing, but the value of c alpha, a typical plot is shown in the next figure.

So, this is the figure, what we do in the consolidation test, we take the deformation dial gauge reading and that corresponding to those deformation dial gauge reading, we can get the values of wide ratio at any particular time during in the that particular stress range. Let us say we have conducted our test, for a stress range of 25 kilo Newton per meter square to 50 kilo Newton per meter square. So, starting from 25 kilo Newton per meter square we can find out what is the wide ratio.

And, then with different time intervals we can find out the void ratio, and then we plot as a curve between the wide ratio and the log of time t. Now, from here to here, this is the portion which corresponds to the primary consolidation and this portion corresponds to the secondary consolidation. Now, theoretically this portion should be asymptotic to the time axis, but practically we see that, this at this part of the curve is not asymptotic to the time axis. And hence using the theoretical curve, we can find out the point corresponding to 100 percent degree of primary consolidation.

So, what we do we draw a tangent to these two straight lines, wherever it intersects we say that this point corresponds to 100 percent degree of consolation and whatever change in wide ratio is that is due to the secondary compression. It means that above this is the part of primary consolidation and below this is the part of secondary consolidation. And the line of this slope, line of this slope of this line will give you the value of c alpha and normally it is measured for 1 log cycle.

(Refer Slide Time: 05:25)

Separating the primary and secondary compression in a clear cut manner is rather tricky, especially if the soil is thick. In a thick layer, while parts of the layer which are near the drainage faces may have no excess pore water pressure in them and may thus already be under secondary compression whereas the parts of soil layer near the middle of the layer are still a far way off from completing their primary consolidation.

So, in this manner we can, find out the coefficient of consolidation, separating the primary and the secondary compression is in a clear cut manner is rather tricky, especially if the soil is thick. In a thick layer, while parts of the layer which are very close to the drainage surfaces may have no excess pore water pressure and in them may thus already be under secondary compression where as the parts which are away from that it means that in the middle of the layer you will find that they a are still a far way off from completing the primary consolidation.

(Refer Slide Time: 06:07)

c_α may decrease as the thickness of layer increases. Another factor, namely, the ratio of applied stress increment to the existing overburden pressure, $\Delta\sigma'/\sigma'_0$, also said to influence the significance of secondary compression relative to the primary compression. Greater the ratio $\Delta\sigma'/\sigma'_0$ smaller the significance of secondary compression. Also the value of c_α is itself, strictly, not a constant but may change somewhat, with time.

C alpha may decrease as the thickness of the layer increases, another factor namely the ratio of applied stress increment to the existing overburden pressure that is delta sigma dash by sigma zero bar. Also set to influence the significance of secondary compression relative to the primary compression. Greater the ratio smaller the significance of secondary compression, also the value of c alpha is itself not a constant, but may change somewhat with time.

(Refer Slide Time: 06:41)

In-spite of all these uncertainties, working values of c_a can be determined by assuming that the influence of the factors mentioned above is negligible. As long as the stress increment is large enough to go well beyond the pre-consolidation stress σ'_c, the ratio c_a/c_c can be considered approximately constant over the normal range of engineering stresses.

In-spite of all these uncertainties working value of c alpha can be determined by assuming that the influence of the factors mentioned above is negligible. As long as the stress increment is large enough to go well beyond the pre-consolidation stress sigma c dash. The ratio of c alpha upon cc that is the compression index can be considered approximately constant over the normal range of engineering stresses.

(Refer Slide Time: 07:13)

Mesri and Godlewski (1977) gave values of c_a/c_c ratio for a variety of natural soils. According to these authors, c_a/c_c has a median value of 0.05. The higher values of c_a/c_c (up to about 0.11) were exhibited by peats and certain organic soils while for inorganic soils, the range is from 0.025 to 0.06. Ladd (1976) suggested that for normally consolidated soils,

$$C_a(\%) = (4-6)\frac{C_c}{1+e_0}$$

Mesri and Godlewski 1977 gave values of c alpha up on cc ratio for a variety of natural soils. According to these authors c alpha by cc has a median value of 0.05, the higher values of c alpha up on cc up to about 0.11 were exhibited by peats and certain organic

soils. While for inorganic soils the range if from 0.025 to 0.06. Ladd in 1976 suggested that for normally consolidated soils c alpha can be determined as C alpha in percentage equal to 4 to 6 times of cc by 1 plus e 0.

(Refer Slide Time: 07:54)

The secondary consolidation settlement is worked out from the following equation knowing H_0 and e_f for a particular stress increment.

$$S_s = \frac{\Delta e}{1 + e_f} H_0$$

The secondary consolidation settlement is worked out from the following equation knowing H 0 and ef, ef is the final void ratio after the consolidation process is over for a particular stress increment. So, ss will be given by delta e up on 1 plus ef in to H 0.

(Refer Slide Time: 08:15)

In the case of secondary consolidation the value of Δe is not related to the stress increment. Δe is a function of time and is equal to c_a times $\Delta \log t$. e_f is the void ratio at the end of primary consolidation (at U = 100%).

$$S_s = \frac{C_a \Delta \log t}{1 + e_f} H_0$$

In the case of secondary consolidation the value of delta e is not related to the stress increment, but it is a function of time and is equal to C alpha times delta log of t, so in

place of delta e we can write this as C alpha delta log of t and the. And the expression will be Ss equal to C alpha delta log of t by 1 plus ef in to H 0.
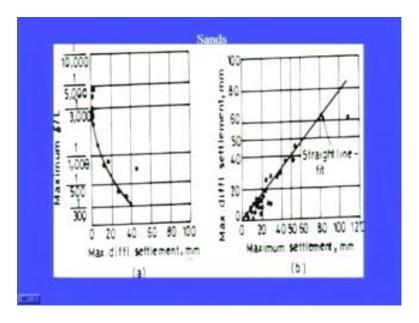
(Refer Slide Time: 08:43)

So, in this manner are we can find out the consolidation settlement also, so the total settlement will be equal to the initial or immediate settlement then plus the primary consolidation settlement and then the secondary consolidation settlement, then we will discuss the allowable settlement, what should be the allowable settlement for different structures. Now, relationship between the total and differential settlement, we know that settlement is of two types one is the total settlement another is the differential settlement.

We have already seen that the total settlement is not that important as the differential settlement, while it is relatively easy to estimate the total settlement, it is very difficult to estimate differential settlement by the means of a rotational analysis. For the for all practical purposes it may be enough to use an empirical relationship between total settlement and differential settlement, which is normally less than the total settlement. And specify the design criterion, merely in the terms of permissible total settlement.

This relationship is a function of the type of the structure, Jerome in 1963, observations on actual building resting on granular and clayey soil are presented in figures shown next, part eight is the relationship between the maximum angular distortion delta l, delta by l and maximum differential settlement. Part B show the relationship between maximum deferential settlement and maximum total settlement. Maximum distortion is determined by the differential settlement between adjacent columns.
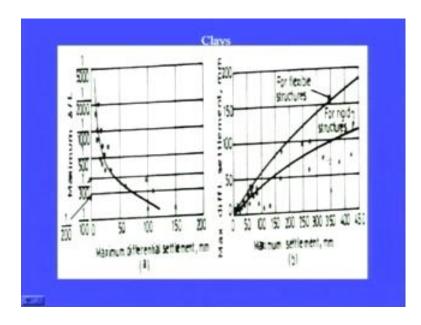
Where as the maximum differential settlement can well be between two columns which are far apart. So, this these are the curves presented by Jerome, for the case of sands

(Refer Slide Time: 10:41)

So, this the maximum delta by l and corresponding to this maximum delta by l this is the maximum differential settlement, so far that maximum delta by l we can find out the differential settlement and corresponding to this differential settlement, here from this maximum differential settlement, we can find out the maximum settlement. Now, this is a straight line fit for the case of sands.

(Refer Slide Time: 11:10)

Similarly, for the clays, this the relationship between maximum delta by l and differential settlement and here in the case of clays, this is the relationship for the case of reject footings and this the clays of for the case of flexible footings.

A study of these figures indicate that in granular soils, the minimum differential settlement in, some cases, be close to maximum total settlement whereas in clays, the differential settlement is much less than the total settlement. Maximum differential settlement, generally does not exceed 75% of the maximum total settlement in granular soils whereas in clays it seldom exceeds 50% of the maximum total settlement.

A study of these figures indicate that in granular soils, the minimum deferential settlement in some cases be close to the maximum total settlement, whereas in clays the differential settlement is much less than the total settlement, maximum differential settlement generally does not exceed 75 percent of maximum total settlement in granular size. Whereas in clays it seldom exceeds 50 percent of the maximum total settlement.

Limiting Angular distortions

Type of structural problem	Angular distortion δ/l
Difficulties with machinery sensitive to settlement	1/750
Danger for frames with diagonals	1/600
Limits for building where cracking is not permissible	1/500
Limit where first cracking in panel walls is to be expected or where difficulties with overhead cranes are to be expected	1/300
Limit where tilting of high, rigid buildings may become noticeable	1/250
Considerable cracking in panel and brick walls. Safe limit for flexible walls h/l<1/4, Limit where structural damage of general buildings any occur	1/150

Now, these are the limiting values of the angular distortion, that depends on the type of the structure problems, so this table gives for different types of the structures the value of the angular distortion. Which can be permitted in that particular structure, like in the case when there are difficulties with machineries sensitivity to settlement this delta by l angular distortion is kept 1 upon 750.

Similarly, for other cases also for example, limit where tilting of high rigid buildings may become noticeable we will have to keep this angular distortion as 1 upon 250, so the various values of the angular distortion which can be permitted can be seen from this particular table. And from this delta by l we can find out maximum, a differential settlement, and from the maximum differential settlement, we can find out the maximum total settlement by using the previous curves which I projected earlier.

The figures can be used in the following manner from the nature of the structure permissible by delta by l is selected, the plots are used to find first the maximum corresponding maximum differences settlement and than the maximum permissible total settlement. Actual estimated a settlement should be less than this permissible settlement. Then allowable limit of settlement, different building codes having specified permissible limits of settlement and angular distortions.

These are mainly related to the type of structure and intended use, the type of the type and sizes of foundation and the nature of soil IS 1904 gives limits of the total settlement, differential settlement and angular distortion for certain typical structures resting either on sand or hard clay or plastic clay. These are presented in the following tables...

(Refer Slide Time: 14:00)

Isolated Foundations on Sand and Hard Clay

Type of structure	Maximum settlement, mm	Differential settlement, mm	Angular distortion
Steel Structures	50	0.0033L	1/300
Reinforced Concrete Structures	50	0.0015L	1/666
Plain brick Walls in Multistoreyed Buildings			
(a) L/H < 3	60	0.00025L	1/4000
(b) L/H >3	60	0.000033L	1/3000
Water Towers and silos	50	0.0015L	1/666

Like, isolated foundations on sand and hard clay, so depending up on the type of the structure, this is the permissible maximum settlement, this the maximum permissible differential settlement and this is the permissible angular distortion. For example, if we have steel structures, so for steel structures having isolated foundations either on sand on hard clay maximum permissible settlement is 50 millimetre, whereas differential settlement is 0.0033l.

Where l is the distance between the columns or it is the distance between the foundations, and this is the shorter dimension, this the angular distortion 1 up on 300, for reinforce concrete structures, it is 50 millimetre. Differential settlement 0.0015L angular distortion 1 upon 666. Similarly, for plain brick walls in multi storied buildings if L up on H is less than 3, 60. Where H is the height of the wall this is 0.0025L and angular distortion permissible is 1 up on 4000.

Similarly, for the case L by H greater than 3, maximum settlement 60 millimeter, differential settlement 0.000033. And angular distortion is 1 up on 3000 for water towers and silos maximum settlement is 50 millimetre and differential settlement is 0.0015 and the angular distortion is 1 upon 666, now similar to this we have, tables for isolated foundations on plastic clay.

(Refer Slide Time: 16:00)

Raft Foundations on Sand and Hard Clay

Type of structure	Maximum settlement, mm	Differential settlement, mm	Angular distortion
Steel Structures	75	0.0033L	1/300
Reinforced Concrete Structures	75	0.002L	1/500
Plain brick Walls in Multistoreyed Buildings			
(a) L/H < 3	Not likely to be encountered		
(b) L/H > 3			
Water Towers and silos	100	0.0025L	1/400

You will find that these two values for all these structures are similar, only difference is in the maximum settlement, and that is for steel structures 50 millimetres for reinforced concrete structure is 75 millimetres and for plain brick walls having L up on H less than or equal to 3 it is 80 millimetre, greater than 3 it is 80 millimetre and water tower inside soil lose 75 millimetre. You can see from these two tables, that for plastic clay maximum settlement permissible is more than that on sand.

(Refer Slide Time: 16:34)

Raft Foundations on Sand and Hard Clay

Type of structure	Maximum settlement, mm	Differential settlement, mm	Angular distortion
Steel Structures	75	0.0033L	1/300
Reinforced Concrete Structures	75	0.002L	1/500
Plain brick Walls in Multistoreyed Buildings			
(a) L/H < 3	Not likely to be encountered		
(b) L/H > 3			
Water Towers and silos	100	0.0025L	1/400

Similarly, for the case of rock foundations, the maximum settlement values are given here and differential settlement values are given here. Angular distortion when the rock

foundations on sand and hard clay. Similar guidelines are available for the rock foundations on plastic clay, like this is for the is the maximum settlement, this is the differential settlement, this is the angular distortion for various structures, for example in the case of plastic clay, this for the reinforced concrete structure maximum permissible settlement is 100 millimetre.

Whereas for the isolated foundation, it is only 75 millimetre, if the isolated foundation are on sand it is only 50 millimetre. So, for the rock foundations, on plastic clay this value is higher and also different values of the differential settlement.

(Refer Slide Time: 17:30)

It can be seen from the tables that a higher total settlement is permissible in clays than in sands, this is explained in the terms of difference in the rate of settlement in sands and clays. The settlement in sands occurs almost immediately on the placement of the load. while in clays consolidation settlement occurs over a long period of time. Thus there is time for structures resting on clays to adjust to differential settlements. In sands differential settlement can occur as soon as the total settlement itself occurred.

Thus leaving the structure no time for gradual adjustment, then to determines allowing varying pressure, so far we have discussed how to determines the safe varying capacity and how to determines, the varying pressure based on permissible settlement, then how to find out the allowable varying pressure which is the minimum of these two. The allowable varying pressure is the maximum varying pressure that can be applied on the soil, such that the two fundamental requirements.

(Refer Slide Time: 18:39)

First is adequate factor of safety against shear failure by which we determines the same varying capacity and settlement with in permissible limits are satisfied, it is therefore, simple to arrive at the allowable varying pressure of the foundation as the smaller of the net safe varying capacity and net safe varying pressure.

(Refer Slide Time: 19:01)

Allowable bearing pressure for granular soils, the allowable varying pressure of a foundation on a granular soil is usually governed by the settlement criterion, unless the foundation is narrow and the soil is loose as for footings of usual sizes. Net safe varying capacity is quite high. The footings on granular soils are proportion commonly by the use

of N values the s p t test value, most of the methods propose, empirical equations or charts to determines allowable varying pressure for a specified maximum total settlement in terms of N values.

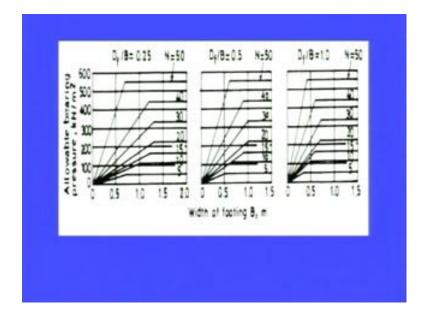One such method, which is very popularly used is the Peck Hanson and Thornburn procedure.

(Refer Slide Time: 19:52)

Peck, Hanson and Thornburn Procedure

Peck, Hanson and Thornburn (1974) have modified the original Terzaghi and Peck (1948) recommendations and presented charts to obtain allowable bearing pressure for a footing of known width, **B** with its base at a depth, D_f resting on a granular deposit in which N values are measured. The charts take in to account both the bearing capacity and settlement consideration.

Peck Hanson and Thornburn in 1974 have modified the original Terzaghi and Peck 1948 recommendations, and presented charts to obtain allowable varying pressure for a footing of a known width b. With it base at a depth Df resting on the granular deposit in which N values are measured. The charts taken to account both the bearing capacity and settlement consideration.

The these are the charts proposed by Peck Hanson and Thornburn, now you can see these are the charts for different values of Df by B ratio Like this is for Df by B 0.25, Df by B 0.5 and Df by B equal to 1.0. And this is allowable bearing pressure in kilo Newton per meter square, versus the width of the footing for different values of N, now from this for different permissible settlement these are for the permissible settlement of 25 millimetre.

So, these charts can be used to obtain allowable bearing pressure for different foundations having different Df by B ratio and for the soil strata having different values of N. The initial straight line portion, radiating from the origin gives the safe bearing capacity with a factor of safety two, while the later horizontal portion gives the safe bearing pressure for a permissible settlement of 25 millimetre. Thus, for the given width of footing and N value the charts that makes it possible to determines the criterion, governing the design before reading of the allowable varying pressure.

The allowable soil pressure for a footing on sand from settlement consideration is given by

$q_{a-net} = 0.44\, N\, S_a\, C_w$ kN/m²

where

N = average corrected 'N' value
S_a = permissible settlement in mm
C_w = water table correction factor
     = $0.5 + 0.5\,[D_w / (D_f + B_f)] \leq 1$
D_w = depth of water below ground level
D_f = depth of foundation
B_f = width of foundation

The allowable soil pressure for a footing on sand from settlement consideration is given by q a net equals to 0.44 and allowing the settlement as in to Cw, which is the water table correction factor. So, here N is the average corrected N value, Sa is the permissible settlement in millimetre, Cw is the water table correction factor that can be determined if we know the depth of the water table below the foundation level, so this is 0.5 plus 0.5 Dw up on Df plus Bf it should be less than 1.0.

Where, Dw is the depth of water table below ground level, Df is the depth of foundation and Bf is the width of foundation.

Teng's Correlation

Teng's (1962) equation is based on Terzaghi and Peck (1948) charts with an additional factor introduced to account for the influence of depth of foundation.

The safe bearing pressure q_{np} is given by the equation

$$q_{np} = 1.4(N-3)\left[\frac{B+0.3}{2B}\right]^2 R_w C_D S_a \ kN/m^2$$

Another correlation is given by Teng's, so Teng's equation is based on Terzaghi and Peck 1948 charts, with an additional factors introduced to account for the influence of depth of foundation. The safe bearing pressure q and rho is given by the this particular equation, which reads q n rho equal to 1.4 N minus 3 in bracket B plus 0.3 divided by 2 B is square R w dash C d S a in kilo Newton per meter square.

(Refer Slide Time: 23:22)

where

B = width of foundation, m

R'_w = water table correction factor

$= 0.5 + 0.5 [D'_w / B] \leq 1$

N = corrected N' value, N_{cor}

S_a = permissible settlement in mm

C_D = depth correction factor

$= 1 + [D / B] \leq 2$

D'_w = depth of water below base of footing

D = depth of foundation

Where, B is the width of the foundation in meters, Rw dash is the water table correction factor with given by this particular equation here Dw dash is the depth below the foundation level, N is the corrected N value that is the N corrected which we determine for a depth of 1.5 to 2 B below the foundation level, Sa is the permissible settlement in millimetre. So, for different structures for different type of soils, we know what that can be maximum permissible values that we can use here.

CD is the depth correction factor that is equal to 1 plus D up on B where D is the depth of the foundation an B is the width of the foundation, Dw dash is the depth of the water below the base of the footing and D is the depth of foundation.

$$N_{cor} = C_N \times N_{field} \quad \text{and}$$
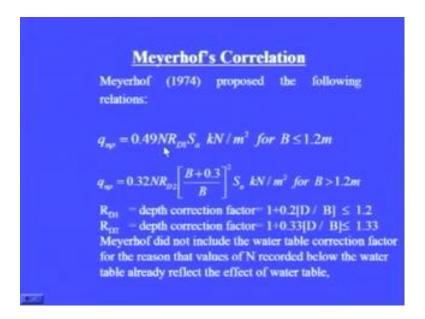
$$C_N = \left[\frac{1.75}{\sigma_0' + 0.7}\right]^2 \quad \text{for} \ 0 < \sigma_0' < 1.05$$

$$C_N = \left[\frac{3.5}{\sigma_0' + 0.7}\right]^2 \quad \text{for} \ 1.05 < \sigma_0' < 2.8$$

σ_0' = effective overburden pressure in kg/cm²

This N corrected is corrected for, the by this particular expression, that is N corrected equals to CN in to N field and CN is given by 1.75 by sigma 0 dash plus 0.75 square for sigma 0 dash less than 0 to 1.05, Cn is equal to 3.5 upon sigma zero dash plus 0.7 whole square. For sigma 0 dash between 1.05 to 2.8, where the sigma 0 dash is given in kg per centimetre square.

Meyerhof's Correlation

Meyerhof (1974) proposed the following relations:

$$q_{np} = 0.49 N R_{D1} S_a \ kN/m^2 \quad \text{for} \ B \leq 1.2m$$

$$q_{np} = 0.32 N R_{D2} \left[\frac{B + 0.3}{B}\right]^2 S_a \ kN/m^2 \quad \text{for} \ B > 1.2m$$

R_{D1} = depth correction factor = 1 + 0.2[D / B] ≤ 1.2
R_{D2} = depth correction factor = 1 + 0.33[D / B] ≤ 1.33
Meyerhof did not include the water table correction factor for the reason that values of N recorded below the water table already reflect the effect of water table.
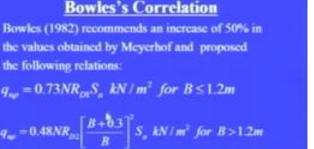
Similar correlation are given by Meyerhof and Meyerhof in 1974 proposed the following relationship q n rho is equal to 0.49 N R D 1 as kilo Newton per meter square if the width is less than 1.2 meter and if width is more than 1.2 meter, Then this q n rho is

equal to 0.32 N R D 2 B plus 0.3 up on B hole square into S a. Where RD 1 is the depth correction factor which can be determined like this, 1.0 plus 0.2 D up on B it should be less than 1.2 And RD 2 is depth correction factor 1 plus 0.33 D up on B, it should be less than or equal to 1.33.

Meyerhof did not include, the water table for the reason that values of N corrected below the water table already reflect the effect of water table.

(Refer Slide Time: 25:54)

Bowles's Correlation

Bowles (1982) recommends an increase of 50% in the values obtained by Meyerhof and proposed the following relations:

$$q_{np} = 0.73 N R_{D1} S_a \ kN/m^2 \ for \ B \le 1.2m$$

$$q_{np} = 0.48 N R_{D2} \left[\frac{B+0.3}{B} \right]^2 S_a \ kN/m^2 \ for \ B > 1.2m$$

Bowles recommends that the field N values must be corrected for overburden pressure but corrected N value must not exceed twice the field value.

Some correlation given by Bowels are shown below, he recommends an increase in 50 percent in the value obtained by Meyerhof and proposed the following equation, so this is nothing, but Meyerhof equation, the only thing is it is an increase there is an increase of 50 percent. So, this q n rho is given by this expression, for B less than 1.2 meter and q n rho is given by this expression for B greater than 1.2 meter.

Bowels's recommends that the field N value must be corrected for over burden, but not for must not exceed the twice the field value, in any case this should not be more than two times the field value.

(Refer Slide Time: 26:37)

where

B = width of foundation, m

R'_w = water table correction factor

= $0.5 + 0.5 [D'_w / B]$ ≤ 1

N = corrected 'N' value, N_{cor}

S_a = permissible settlement in mm

C_D = depth correction factor

= $1 + [D / B]$ ≤ 2

D'_w = depth of water below base of footing

D = depth of foundation

Now, different terms used in the Bowels equation are B is the width of the foundation, Rw dash is the water table correction factor given by this expression, N is the corrected N value S is the permissible settlement, CD is the depth correction factor which is given by this expression, Dw dash is the depth of the water below the footing and D is the depth of foundation.

(Refer Slide Time: 27:04)

$$N_{cor} = C_N \times N_{field} \text{ and}$$

$$C_N = \left[\frac{1.75}{\sigma'_0 + 0.7} \right]^2 \text{ for } 0 < \sigma'_0 < 1.05$$

$$C_N = \left[\frac{3.5}{\sigma'_0 + 0.7} \right]^2 \text{ for } 1.05 < \sigma'_0 < 2.8$$

σ'_0 = effective overburden pressure in kg/cm²

N corrected can be determined from the N field using these relationships, when sigma 0 dash is between 0 to 1.05 kg per centimetre square we can use this expression, and for

values between 1.05 to 2.8 we can use, this particular expression to find out the value of a correction factor.

(Refer Slide Time: 27:32)

Then, correlation for the Raft Foundation, A raft or a mat foundation covers the entire plan area of the building. It is used when individual spread footings cannot be used either because of heavy loads or poor soil conditions or both. Because of its rigidity, a raft tends to bridge over a local soft pockets or any other heterogeneity of the strata and the irregularities tend to get evenly distributed below the raft.

Owing to these reasons a raft foundation suffers much less differential settlement than when isolated foundation are provided, it is therefore natural that the safe bearing pressure for a raft foundation should be much higher than for an isolated foundation, for same limit of differential settlement. Putting it in another way, it is possible to have a permissible total settlement for raft almost twice that for an isolated foundation. But it still have the same differential settlement, resulting in both the cases.

A foundation with its width six meter or more can be classified as a raft foundation. The allowable varying pressure of a raft on a granular soil is always governed by the settlement considerations.

Teng's gives the following equation for allowable bearing pressure on the raft foundation q n rhois equal to 0.7 N minus 3 R w dash CD into Sa in kilo Newton per meter square, whereas Peck Hanson and Thonburn equation is q n net equal to 0.88 N S a into Cw kilo Newton per meter square. Above equation underestimates the allowable bearing pressure for N greater than 50 for N less than 5 that is, for a very loose deposit the soil requires densifying before a raft can be used or the raft may be supported on the piles.

Allowable bearing pressure for cohesive soil in the case of rock foundation, field test have limited relevance to the cohesive soils for the reason that these test being essentially

short duration test, cannot reflect in a realistic manner the consolidation settlement characteristic of a clay soil. Both the safe bearing capacity as well as the settlement are determined by using appropriate shear strength and consolidation parameters obtained from laboratory test on undisturbed, representative samples of clay.

It is not easy to say, whether the allowable bearing pressure will be governed by the net safe bearing capacity or the net safe varying pressure in a cohesive soil the usual procedure in proportioning a foundation on a clay soil is to select tentatively the dimensions of a foundation by using the net safe bearing capacity. It may be noted that, unlike in granular soils the bearing capacity is not affected by the size of foundation in clay soils.

(Refer Slide Time: 30:45)

The net safe bearing capacity it determined for the $\phi_u = 0$ condition which represents the most critical situation.

The net safe bearing capacity is determined for phi equal to 0, condition which presents represents the most critical situation.

The undrained shear strength, c_u is determined either from in-situ vane shear test or the unconsolidated undrained test or the unconfined compression test. The net safe bearing capacity is then assumed to be the load intensity acting on the soil at the base of the foundation and the magnitude of settlement is estimated, using the compressibility parameter of the soil. If the estimated settlement is within the permissible total settlement for the structure the dimensions of foundation assumed for analysis are adequate.

The undrained shear strength c u is determined either from in situ vane shear test or the unconsolidated undrained test or the unconfined compression test. The net safe bearing capacity is then assume to be the load intensity acting on the soil at the base of the foundation and and the magnitude of settlement is estimated using the compressibility parameters of the soil. If the estimated settlement is within the permissible total settlement for the structure the dimensions of the foundation assumed for the analysis are adequate.

The dimensions of the foundation may be increased if the settlement criterion is not satisfied, Raft foundation used below one or more basement floors offers a very practical foundation tribes when the loads are heavy and the clay soil is soft. When basement floors are constructed, the excavated soil is not back field, the net loading intensity transfer to the soil can be equated to the net allowing bearing pressure. We call it as the floating foundations.

(Refer Slide Time: 32:06)

$$\frac{Q}{A} - \gamma D_f = q_{a-net}$$

where

Q = total superimposed load

A = area of raft

γ = unit weight of soil

D_f = depth of the base of raft

Then this Q up on a minus gamma Df will be equal to q a net, where q is the total super imposed load A is the area of the raft, gamma is the unit weight of soil and Df is depth of the base of the raft.

(Refer Slide Time: 32:20)

The base of the raft can be located at such a depth (D_f) that the allowable bearing pressure of the raft is sufficient to take care of the net loading intensity at that depth. Such a foundation is known as a partially compensated or partially floating raft. The net loading intensity would reduce to zero if

$$\frac{Q}{A} = \gamma D_f$$

The base of raft can be located at such a depth Df, that the allowable bearing pressure of the raft is sufficient to take care of the net load intensity at that particular depth. Such a foundation is known as the partially compensated or partially floating raft. The net load intensity would reduce to zero if Q up on Q is equal to gamma Df

In such a situation, the soil is not called upon to resist any load. The superimposed load due to construction is fully compensated by the weight of the soil which is excavated but not backfilled. Such a raft is called a fully compensated or a floating raft. Theoretically, such a raft should not settle at all. However, in practice, after the soil is excavated, there is some heaving of soil because of load release and this shall manifest itself as settlement after the structure is raised.

In such a situation the soil is not called upon the to resist any load, the superimposed load due to construction is fully compensated by the weight of the soil itself which is excavated, but not back filled. Such a raft is called a fully compensated or floating raft. Theoretically such a raft should not settle at all; however, in practice after the soil is excavated, there is some heaving of the soil because of the load release and this shall manifest as settlement after the structure is raised.

The principle of load compensation can be used with advantage even in the case of loose granular soil deposits.
The net safe bearing capacity of a raft foundation on a cohesive soil is usually determined by the Skempton's equation. Settlement of a raft on a normally consolidated clay can be very high even for a low loading intensity.

The principle of the load compensation can be used with advantage even in the case of loose granular soil deposits. The net safe varying capacity of a raft foundation on a

cohesive soil is usually determined by the Skempton equation. Settlement of a raft on a normally consolidated clay can be very high even for a low load intensity.

(Refer Slide Time: 33:58)

SOME GUIDELINES FOR LOCATION AND DEPTH OF FOUNDATIONS

Look, there are some guidelines given in the IS code for the location and the depth of the foundation, which have to be observed when we select the depth of the foundation and also the distance between two footings. Foundations must be properly located and placed at such a depth that its performance, it is not adversely affected by the factors such as lateral expulsion of soil from beneath the foundation, seasonal volume changes caused by freezing and time and a presence of adjoining structures.

(Refer Slide Time: 34:33)

As a general rule, any foundation must be placed at a depth where the soil stratum is adequate from the point of view of bearing capacity and settlement criteria.

As a general rule, any foundation must be placed at a depth where the soil stratum is adequate from point of view of bearing capacity and settlement criteria. Further foundation must be placed below the zone of swelling and shrinkage which is crucial in the case of black cotton soils. And below the zone of fast heave in case of sands and cells in the areas where extremely low temperatures are lightly to materialise.
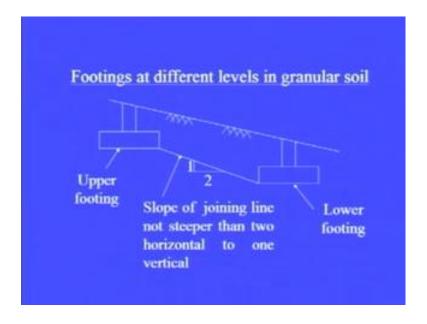
(Refer Slide Time: 35:08)

IS 1904-1978 have given recommendations for the location and depth of the foundations. As per the recommendations, the minimum depth of the foundations should be 50 centimetre, when the ground surface slopes downwards adjacent to a footing the sloping surface should not encroach upon a frustum of bearing material under the footing having sides which make an angel of 60 degree with the horizontal for rock and 30 degree with the horizontal for soil.

The horizontal distance, that is the edge distance from the lower edge of the footing to the sloping surface must be at least 60 centimetre for the case of rock and 90 centimetre for the case of soil.
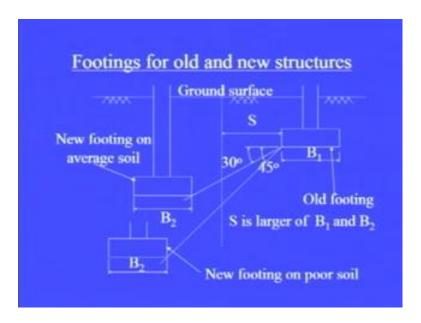
Now, if the footings are at different levels, then we will have to observe these recommendations. Let us say you have got a sloping surface and there are two footings this is the upper footings and this is the lower footings, then the slope of the line joining the bottommost edge or bottommost corner of the upper footings and the bottommost

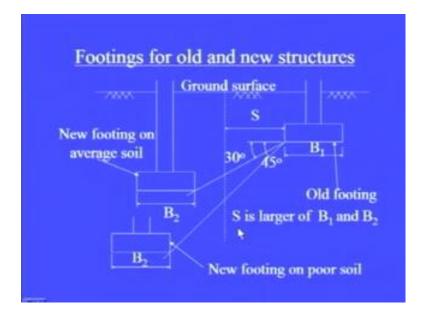corner or the edge of the lower footing. It should not be steeper than two horizontal to one vertical.

Now, if the footings are at different levels in the clay soil. It is recommended that for the this bottom most edge of the upper footings and the line joining the uppermost edge of the lower footings the slope should not be steeper than 2 is to 1, 2 horizontal to 1 vertical.

(Refer Slide Time: 36:58)

Now, suppose it is proposed to place a foundation, adjacent to a old structure, like we have a an old footings, which is here and now we want to place another new footings then we will have to observe these two criteria. Number one this is the spacing or the distance of the footings from the centre line, so S should be larger of B 1 or B 2, where B 1 is the width of the old footing and B 2 is the width of the new footing on the average side. And when we join the bottom most edges of the old structure or old footing and the new footing. This slope with the horizontal 35 degrees, when the new footings is on the average side. And this should be equal to 45 degrees, when the footing is on the poor soil.

(Refer Slide Time: 38:02)

Footings for old and new structures

The steps involved in the proportioning of the footings can be summarised, now as below.

(Refer Slide Time: 38:10)

- It is essential to estimate the dead load, live load and other loads such as wind load or seismic load that act on the foundation. It is a common practice to assume that the wind load and the seismic load do not act simultaneously. Moreover, since some live loads are temporary and transient, it is necessary to include only that part of the live load that may actually induce settlement in the foundation soil.

It is essential to estimate the dead load, live load and other loads such as wind load or seismic load that act on the foundation. It is a common practice to assume that the wind load and the seismic load do not act simultaneously. Moreover, since some live loads are temporary and transient, it is necessary to include only that part of the live load that may actually induce settlement in the foundation soil.

For this reason, in the case of clay soils, where the settlement is time-dependent, it is enough if the design is carried out for a reduced value of live load, this being defined as the live load acting for the major part of the life of the structure. Generally, one half of the design live load is taken as 'permanent'. However, for foundations resting on coarse-grained soils, the maximum live load that can ever occur is considered, irrespective of the duration of such a load, since settlement can occur in a very short period in such soils. The bending moment at the base of the column or wall, if any, should also be determined.

For this reason, in the case of clay soils, where the settlement is time-dependent, it is enough if the design is carried out for a reduced value of live load, this being defined as the live load acting for the major part of the life of the structure. Generally, one half of the design live load is taken as permanent. However, for foundations resting on coarse-grained soils the maximum live load that can ever occur is considered, irrespective of the duration of such a load, since settlement can occur in a very short period in such soils. The bending moment at the base of the column or wall if any, should also be determined.

- A square footing is preferred where only axial loads act. If the footing is to be designed for bending moment also in addition to the axial load, a rectangular footing is more suitable.

- The depth of foundation is fixed suitably, by considering the nature of the structure and subsoil conditions as revealed by the borehole log data.

A square footing is preferred where only axial loads act, if the footing is to be designed for bending moment also in addition to the axial load, a rectangular footing is more suitable. The depth of the foundation is fixed suitably, by considering the nature of the structure and sub-soil conditions as revealed by the borehole log data.

(Refer Slide Time: 39:51)

The proportioning of a footing is a problem which is interactive in nature. The typical structural engineer's approach of obtaining the plan dimensions by dividing the column load or wall load by the so-called bearing capacity cannot be used here. Suitable plan dimensions, namely the width of a square or a strip footing and the width and length, if the choice is rectangular footing have to be assumed to begin with.

For this assumed plan area, the net loading intensity, q_n is equal to the design load divided by the plan area. q_n should not be allowed to exceed the net allowable bearing pressure, $q_{a\text{-net}}$, i.e. $q_n < q_{a\text{-net}}$. If it does, the design is unsafe. On the other hand, if it is very much less, i.e. if $q_n \ll q_{a\text{-net}}$, the design is oversafe and hence uneconomical. The proportioning of a footing is, thus, a process of optimization, ensuring both safety and economy. Since $q_{a\text{-net}}$ is the smaller of net safe bearing capacity (shear) and the safe bearing pressure (settlement), both of these considerations will have to be factored in the design.

For this assumed planned area, the net loading intensity qn is equal to the design load equal to the plan area q n should not be allowed to exceed the net allowable bearing pressure, qa net that is qn should always be less then qa net. If it does, the design is unsafe. On the other hand if it is very much less, that is qn is far, far less than q a net the design is over safe and hence uneconomical. The proportioning of a footing is, thus, a process of optimization ensuring both safety and economy.

Since q a net is the smaller of net safe bearing capacity from shear consideration and the safe bearing pressure from settlement consideration both the considerations have to be factored in the design.

(Refer Slide Time: 41:16)

For the assumed dimensions and depth of footing, the net safe bearing capacity, q_{ns} can be calculated on the basis of soil data obtained from tests and using the methods discussed all through the lectures on shallow foundations.

In granular soils, the standard penetration test or the plate load test gives the data required for this purpose. The N value to be used is the average of the corrected N values from the base of the footing up to a depth equal to the assumed width of the footing.

For the assumed dimensions and depth of footing, the net safe bearing capacity qns can be calculated on the basis of soil data obtained from tests and using the methods discussed all through the lectures on shallow foundations. In granular soils, the standard penetration test or plate load test gives the data required for this purpose. The N value to be used is the average of the corrected N values from the base of the footing up to a depth equal to the assumed width of the footing.

(Refer Slide Time: 41:51)

For clay soils, the average undrained shear strength c_u- required as input in the bearing capacity equation is obtained from laboratory shear tests on undisturbed samples.

Check if $q_n < q_{ns}$. If not, the dimensions of the footing need to be revised upward and step is repeated till the check is O.K.

For clay soils, the average undrained shear strength cu required as input in bearing capacity equation is obtained from laboratory shear test or undisturbed samples. Check if

qn is less than qns if not, the dimension of the footing need to be revised and step is repeated till the check is.

(Refer Slide Time: 42:15)

- The total or maximum settlement of the footing in a granular soils is determined using the methods with N values or the plate load test data as basis. The safe bearing capacity is used as the pressure inducing settlement in the settlement computation. The settlement in a clay soil is determined with the safe bearing capacity assumed to act at the base of the footing, using consolidation test data.

The total or the maximum settlement of footing in a granular soils is determined using this methods with N values or the plate load test data as basis. The safe bearing capacity is used as the pressure inducing settlement in the settlement computation. The settlement in a clay soil is determined with safe bearing capacity assumed to act at the base of the footing using consolidation test data.

(Refer Slide Time: 42:42)

- An estimate is also made, where required, of the differential settlement between various footings and the angular distortion between various parts of the structure. The maximum settlement, the differential settlement and the angular distortion obtained are compared with the permissible values or any accepted code of practice. If they are not within the permissible limits, the safe bearing capacity is revised and the settlement computation is repeated to ensure that all the requirements are met.

An estimate is also made, where required, of the differential settlement between various footings and the angular distortion between various parts of the structure. The maximum settlement the differential settlement and the angular distortion obtained are compared with the permissible values or any accept by any accepted code of practice. If they are not within the permissible limits the safe bearing capacity is revised and the settlement computation is repeated to ensure that all the requirements are met.

(Refer Slide Time: 43:20)

The stability of the footing is checked to ensure safety against sliding and over turning. The factor of safety should not be less than 1.75 against sliding and not less than 2.0 against overturning. When only dead load, live loads and earth pressure are considered. It must now be clear why it was said earlier that the proportioning of footing is an interactive procedure. Unless the depth of footing and tentative dimensions of footage are assumed. The average n corrected N values cannot be worked out.

Also, the soil strata forming the seat of settlement and their thickness are also dependent on the assumed values of depth of footing and the width of the footing.

(Refer Slide Time: 44:08)

> Some investigators believe that while the bearing capacity can be calculated by considering the soil stratum/strata included within a depth of B below the base of the footing, the settlement computation should be done by considering the seat of settlement to be at-least 1.5B to 2.0B below the footing base.
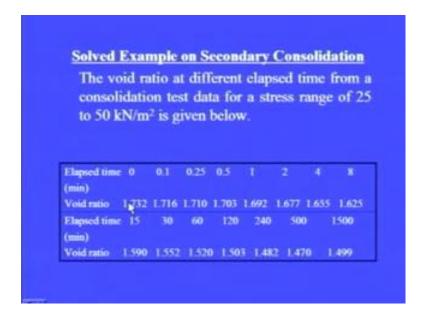
Some investigators believe that while the bearing capacity can be calculated by considering the soil stratum or strata included within a depth of B below the base of the footing, the settlement computation should be done by considering the seat of settlement to be at least 1.5 to b to 2.0 B below the footing base.

So, following these steps discussed earlier, we arrive at the dimensions of the foundation.

(Refer Slide Time: 44:40)

SOLVED EXAMPLES

Now, Ii am going to discussed few solved examples, by which it will become more clear how to obtain the allowable bearing pressure of foundations.

(Refer Slide Time: 44:46)

Solved Example on Secondary Consolidation
The void ratio at different elapsed time from a consolidation test data for a stress range of 25 to 50 kN/m² is given below.

Elapsed time (min)	0	0.1	0.25	0.5	1	2	4	8
Void ratio	1.732	1.716	1.710	1.703	1.692	1.677	1.655	1.625
Elapsed time (min)	15	30	60	120	240	500	1500	
Void ratio	1.590	1.552	1.520	1.503	1.482	1.470	1.499	

Now, this is the solved example on secondary consolidation first. The void ratio at different elapsed time from a consolidation test data for a stress range of 25 to 50 kilo Newton per meter square is given below. Now, here these are the void ratio verses elapsed time. Actually in the consolidation test we get reading corresponding to different time interval, now by using either height of solids method or changing void ratio methods we can determines the void ratio corresponding to that reading.
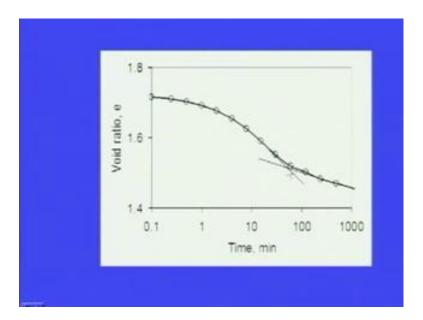
So, these are the direct void ratio to obtain from those readings, so for example here at the start, the void ratio is 1.732 after 0.25 minutes it is 1.710, after 1 minute it is 1.692 similarly, after 8 minutes it is 1.625, after 60 minutes it is 1.52, and after 500 minutes it is 1.470, so we will be using this data to determines secondary consolidation settlement.

The load increment in the test represents the anticipated stress increment in the field. The primary consolidation may be assumed to be completed after 25 years, as we know secondary consolidation settlement is dependent on the time, not on the stress range, but it occurs at the constant effective stress. The thickness of compressible layers at the site is 8 metres. Compute the amount of secondary consolidation settlement for a period from 25 to 100 years after the construction.

So, first of all, what we do we plot this data on e verses log of time curve, and then by these two portions we draw a tangent here and then the tangent here, we get a point

corresponded to 100 percent degree of consolidation. Now, the particular portion from here to here it represents the primary consolidation whereas, this represents the secondary consolidation. Now, in order to find out c alpha, we find out the slope of this line slope of this line corresponding to 1 log cycle.

And corresponding to 1 log cycle, whatever is the delta e which we can read from here, that is the c alpha coefficient of secondary consolidation.

(Refer Slide Time: 47:32)

Figure shows the void ratio plotted against logarithm of time. c_a is equal to Δe when $\Delta \log t$ corresponds to one full logarithm cycle.
From Fig., $c_a = 0.033$, the void ratio, e_f, at the end of primary consolidation=1.495
Then secondary consolidation settlement is given by

$$S_s = \frac{C_a \Delta \log t}{1+e_f} H_0$$

$$= \frac{0.033 \times \log(100/25)}{1+1.495} \times 8 = 0.064m$$

So, this figure shows the void ratio plotted against log of time t, c alpha is equal to delta c when delta log corresponds to one full log cycle, from figure c alpha has been found to be 0.033, the void ratio ef is the final void ratio after the consolidation process is over, at the end of primary consolidation, we can read it from that figure it comes out to be 1.495. Then secondary consolidation settlement is obtained by this equation, c alpha delta log of t upon 1 plus ef into H 0.

When we substitute all these values, we will get this as the secondary consolidation comes out to be 0.064 metres.

Solved Example on Peck, Hanson and Thornburn procedure

A 3m wide footing is resting at a depth of 1.5m below ground level on coarse sand stratum (γ =1.8 t/m³). The ground water table is encountered at 3.0m depth. The submerged unit weight (γ') of the coarse sand below water table is 1.8 t/m³. A layer of fine sand ($\gamma' = 0.8$ t/m³) lies beyond 4.5m depth. The observed N values are given as follows:

Depth GL, m	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00
N values,	8	10	15	15	18	20	16	25

Allowable settlement = 40 mm.

This is solved example on the Peck Hanson and Thornburn procedure. A 3 metre wide footing is resting at the depth of 1.5 metres belowground level on course and stratum, having unit weight as 1.8 ton per meter cube. The ground water table is encountered at a depth of 3 meter. The submerged unit weight gamma dash of the course sand below water table is 1.8 ton per meter cube. A layer of fine sand gamma dash, equal to 0.8 ton meter cube lies beyond 4.5 meter depth. The observed N values are given as below

This is the depth below ground level and these are the observed N values. 0.75 metre it is 8, 1.5 meter it is 10, 2.25 meter it is 15, likewise 5 .25 meter it is 16 and 6 meter it is 25. The allowable settlement for this foundation is 40 millimetre.

(Refer Slide Time: 49:06)

Solution:
The average value of N_{cor} between El -1.50m and El -6.00 m will be considered for the calculations. This would mean a depth equal to 1.5 B or 4.5 m below the base.

The field value of N, are corrected first for overburden pressure (N') and then for dilatancy (N") using the Peck, Hanson and Thornburn procedure. It may be noted that dilatancy correction is not required in the coarse sand layer.

The average N value of N corrected between elevation 1.5 meter and 6.0 metre, below the ground surface will be considered for the calculations, this would mean a depth equal to 1.5 B or 4.5 meter below the base. The value of N are corrected first for overburden pressure and then for dilatancy using Peck Hanson and Thornburn procedure. It may be noted that dilatancy correction is not required in the coarse sand layer. So, these calculations are done in the tabular form as shown here.

(Refer Slide Time: 49:42)

El. m	σ', kg/cm²	C_N	N	N'~ C_NxN	N"
-1.50	0.270	1.44	10	14.4	14.4
-2.25	0.405	1.30	15	19.5	19.5
-3.00	0.540	1.21	15	18.1	18.1
-3.75	0.615	1.16	18	20.9	20.9
-4.50	0.690	1.13	20	22.6	18.8
-5.25	0.750	1.10	16	17.6	16.3
-6.00	0.810	1.07	25	26.8	20.9

Average N_{cor} = N"
= (14.4 +19.5 + 18.1 +20.9 +18.8 +16.3+ 20.9)/ 7 = 18

First of all, in order to find out the overburden correction, we will have to find out overburden pressure at that particular level. So, knowing the submerged unit weight of

the soil or the effective unit weight of the all the soils are of the point of consideration, we can find out this overburden pressure as gamma into D.

So, from that we can find out the CN as per the Peck Hanson and Thornburn procedure, so corrected the N values in the field are given here, and N corrected for over burden will be equal to CN into N, so we apply this correction factor. Then after getting this we apply the correction for dilatancy, now when we apply correction for dilatancy, we know that the dilatancy correction in which the rate of dissipation of pore water pressure is very high in the case of course end. So, we do not apply dilatancy correction for course end.

After getting this we get the average n value of the N corrected and that is equal to N double dash we use for the, so this will be equal to summation of all these values divided by number of values, so it comes out to be equal to 18. So, this N corrected value is used for the calculation of allowable bearing pressure.

(Refer Slide Time: 51:05)

For $D_f/B = 0.5$, B= 3 m, the allowable bearing pressure corresponds to the horizontal portion of the plot. Hence settlement criterion governs the design.

$q_{a\text{-}net} = 0.044 C_w N S_a \quad t/m^2$

(N =18, S_a=40mm)

$C_w = 0.5 + 0.5 D_w/(D_f+B) = 0.83$

$q_{a\text{-}net} = 0.044*0.83*18*40 = 26 \quad t/m^2$

For Df by B equal to 0.5 and B equal to 3 metre, the allowable bearing pressure corresponds to the horizontal potion of the plot of Peck Hanson Thornburn, hence settlement criterion governs the design. In order to determine q a net we use this particular relationship 0.044 Cw N into S a ton per meter square. Now, here N corrected we have found it comes out to be 18 and permissible settlement is 40 millimeter.

There is water table correction factor can be determined by this equation and it comes out to be 0.83, when we substitute all these values, we get the q a net as 26 ton per meter

square, It means the foundation can be designed for an allowable bearing pressure of 26 ton per metre square, It should b remembered that Peck Hanson and Thornburn use a factor of safety of 2 on ultimate varying capacity.

If the factor decided is the usual value of 3, q ultimate can be computed by obtaining the bearing capacity factors Nq and N gamma from N and use in the bearing capacity equation to obtain the ultimate bearing capacity and then, obtain this safe bearing capacity.

(Refer Slide Time: 52:19)
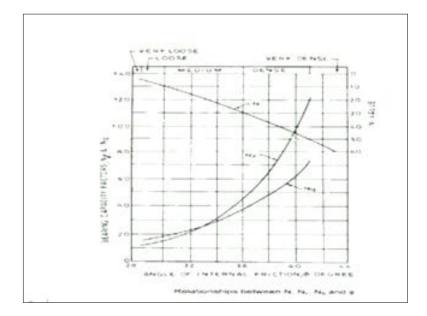

Solved Example on IS Recommendations
Determine the net allowable bearing pressure of the footing in in the previous example, using the IS code recommendations.

Solution:
$N_{cor} = 18$
After Peck, Hanson and Thornburn, 1974
For N= 18, Φ= 33°
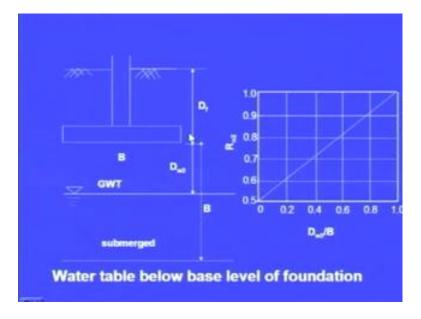and for Φ= 33°, $N_q = 22$, $N_\gamma = 28$

This is another solved example, that uses the IS recommendation, now in order to determine net allowable bearing pressure of the footing, in the previous example using the IS code recommendations, we use this N corrected as 18. So, after Peck Hanson and Thornburn 1974 for N equal to 18 phi is 33 degrees. Now, for phi equal to 33 degrees we can obtain parameters Nq and N gamma, using this particular chart.

So, here this is the N value and from this N value, we can find out angle of shearing resistance and from angle of sharing resistance, we can find out values of Nq and N gamma.

Water table below base level of foundation

Then, we also apply the water table correction when the water table is below the base level of the foundation using this particular plot, so we find out this Rw 2 this we have already discussed, so here in this case this is the depth of the footing, this the depth of the water table below the ground level and we consider this width equal this depth equal to

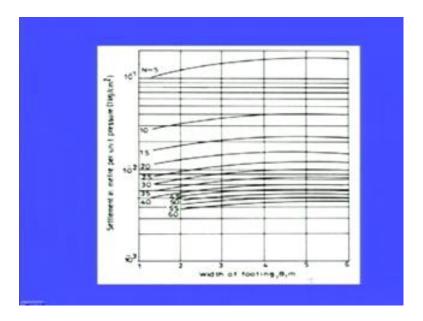equivalent to the width of the footing. And depending upon Dw 2 by B we can find out Rw 2.

(Refer Slide Time: 53:31)

For c'=0

$$q_{nu} = q(N_q-1)s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma R_{w2}$$

$q = 1.8*1.5 = 2.7 \text{ t/m}^2$
$R_{w2} = 0.75$
$s_q = 1+0.2B/L = 1+0.2*3/5 = 1.12$
$s_\gamma = 1-0.4B/L = 1-0.4*3/5 = 0.76$
$d_q = d_\gamma = 1 + 0.1(D_f/B) \tan(45° + \Phi/2)$
$d_q = d_\gamma = 1 + 0.1(1.5/3.0) \tan(45° + 33/2) = 1.09$

So, for c (( )) soil c dash equal to 0, q net ultimate is given by q nq minus 1 sq dq plus 0.5 gamma BN gamma s gamma d gamma into Rw 2, where q is the over burden pressure at the foundation level that comes out to be 1.8 into 1.5 equal to 2.7 ton per meter square. Rw 2 from the previous figure it is 0.75 and safe factors are determined by using the procedure given in IS code and the expressions are like this. Sq equal to 1 plus 0.2 B upon L, S gamma equal 1 minus 0.4 B upon Ll.

When, we substitute different dimensions, we will get this as 1.12 and S gamma as 0.76. dq equal to d gamma is given as 1 plus 0.1 df upon b tan 45 plus phi by 2 and we can obtain it as 1.09.

(Refer Slide Time: 54:25)

When we substitute, these values and using the methods suggested by IS code to determine, the allowable bearing pressure from the settlement criteria, we use this particular chart for the given width and the value of N we can find out, what is the settlement in meter per unit, pressure kg per centimetre square.

(Refer Slide Time: 54:53)

So, first of all we find out q mu that comes out to be 116 ton per metre square, qns is qnu divide by factor of safety that is equal to 38.7 ton per metre square. So, for N equal to 18 and B equal 3, using the previous plot we can find out the settlement for a pressure of 1 kg per centimetre square or 10 ton per metre square, that comes out to be 15 millimetre.

Taking into account the position of water table, actual settlement will be equal to 15 millimetre divided by correction factor that is equal to 20 millimetre. For Sa equal to 40 millimetre qn rho will be equal to 10 upon 20 into 40 that is 20 ton per metre square. Hence qa net using IS recommendation will be equal to 20 ton per metre square. In this manner we can determine the allowable bearing pressure of the foundations.

So, in the last few lectures, we have discussed about the theories related to the shallow foundations and especially the Terzaghi varying capacity equation and the modifications suggested by Meyerhof, Hanson, bridge Hanson, Thornburn. The IS code procedure to determine, the bearing capacity of foundations. Then, we have also discussed how to determine the settlement of the foundations, and what would be the permissible settlement and then we have also discussed the step by step procedure to find find out the dimensions of the foundation.

So, it cover the entire component of the syllabus of the shallow foundations which is start in the under graduate course.

Thank you very much.