Foundation Engineering

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Module - 01
Lecture - 06
Shallow Foundation

I again welcome you on the lecture series of Shallow Foundations. Now, in the last lecture, we discussed about the settlements of shallow foundations and we have seen that the total settlement of the foundation comprises of immediate elastic settlement, consolidation settlement and secondary consolidation settlement. In order to determine immediate or elastic settlement, we can use elastic theory.

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$$S = S_i + S_c + S_s$$

$$S_i = q_n B \frac{(1-\mu^2)}{E_s} I_f$$

$$S_i = q_n B \frac{\mu_0}{E_s} \mu_1$$

And when we use elastic theory by knowing the parameters of the soil mass, Es and mu and depending up on the type of foundation, we can find out influence factor and from these for a net load intensity of q n, which is placed on a width of it fourteen B we can determine the immediate settlement by elastic theory. Jeunotel has also proposed an equation for determining elastic settlement and by this, if we know the parameters mu 0 and mu 1 we can substitute these values here and can get elastic settlement. They have also given, guidance for determining mu 0 and mu 1, mu 0 is a function of the d f by b where d f is the depth of foundation and b is the width of the foundation.

Whereas mu 1 is a function of the shape of the footing and it is also a function of the ratio of H upon b, where H is the thickness of the soil stratum below the foundation level. So, we can use, we can use those charts and from those charts we can obtain mu 0 and mu 1 and substitute in this equation and get immediate elastic settlement.

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$$S_c = \frac{\Delta e}{1+e_0} H_0$$

$$S_c = \frac{a_v}{1+e_0} H_0 \Delta\sigma'$$

$$S_c = m_v H_0 \Delta\sigma'$$

$$S_c = C_c \frac{H_0}{1+e_0} \log \frac{\sigma_o' + \Delta\sigma'}{\sigma_0'}$$

$$S_c = C_r \frac{H_0}{1+e_0} \log \frac{\sigma_o' + \Delta\sigma'}{\sigma_0'}$$

Now, in order to determine consolidation settlement, consolidation settlement can be determined, if we have our relationship between void ratio and the effective stress. And this relationship can be obtained by conducting consolidation test or odometer stress in laboratory on a soil specimen of undisturbed soil, which we extract from the field from a particular depth of the compressible layer. And we can determine this consolidation settlement by this equation, that is Sc equals to delta e up on 1 plus e 0 into H 0.

Where, is delta e is the change in void ratio from load intensity of sigma 0 bar or sigma 0 dash institused stress plus delta sigma bar. So, this delta e is representing change in the void ratio, that is change in volume of void, that is the change in volume of soil, due to the application of delta sigma, that is increase in a stress. And if we know the initial void ratio e 0 and the thickness of the clay stratum, we can obtain consolidation settlement.

Now, this delta e can be written in different form also, like we can write this delta e in terms of coefficient of compressibility a v, this a v is nothing but, the slope of the relationship between e and sigma 0 bar for a particular stress range. So, for that stress range we found this a v and then, we can use in place of delta e we can use this and find out the consolidation settlement, and when we use this, the major of the equation is a v upon 1 plus e 0 into H 0 delta sigma dash.

Now, this a v upon 1 plus e 0 is also known as the coefficient of volume compressbility or coefficient of volume change, so in place of this, we can use this m v and then we can determine this consolidation settlement by equation m v equal into H 0 into delta sigma dash. Now, when we plot this relationship between e and sigma bar, on e log sigma bar curve we will find that the initial portion of the curve is a curved one and as we as the stress increases that relationship assumes straight line relationship.

So, wherever the straight line portion of the curve is there, that is the part where the soil will behave as the normally consolidated clay and the slope of that curve is nothing, but the compression index. So, that compression index can determine, from their end, in place of delta e we can use the relationship in terms of compression index. And the consolidation settlement will be Cc in to H 0 up on 1 plus e 0, log to the base 10 sigma 0 bar plus delta sigma bar divided by sigma 0 bar.

Now, in the portion, where the soil will behave as the preconsolidate clay or the over consolidated clay, which we can determine by obtaining the value of preconsolidation pressure using the Casagrande method. And if the stress range is in the preconsolidated range, in place of compression index we use a term which is known as recompression index and this Cr can be replaced by Cc. So, C this will be written as Cc H 0 point 1 plus e 0 log sigma 0 dash plus delta sigma dash upon sigma 0 dash.

Now, if the total stress range from sigma 0 dash to sigma final dash, that is the sigma 0 dash plus delta sigma dash is in the preconsolidated as well as normally consolidated range, then the settlement can be determined in two parts.

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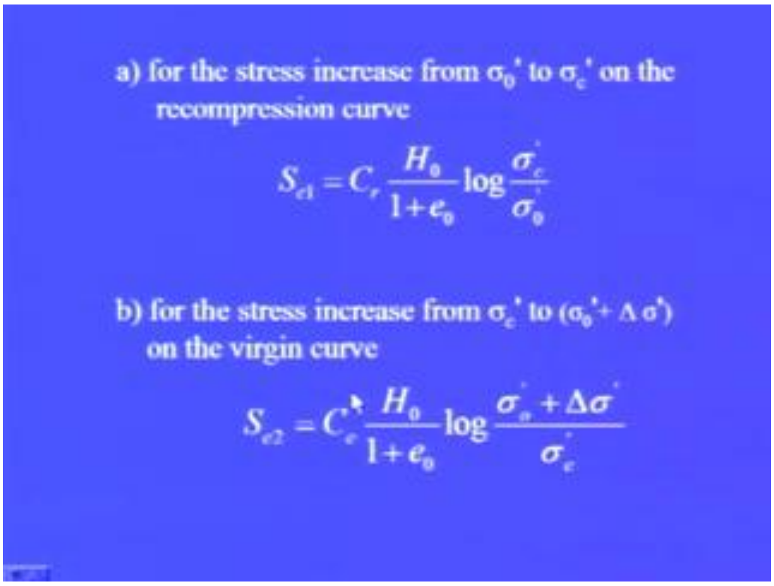

From sigma 0 dash to sigma C dash, we use the recompression index and from sigma C dash to sigma 0 dash plus delta sigma dash, we use the compression index, and the summation of these two will give us the consolidation settlement. Now, whatever settlement we have determined, so far these are to be corrected for the 3 dimensional consolidation as well as the depth effect or the embedment effect.

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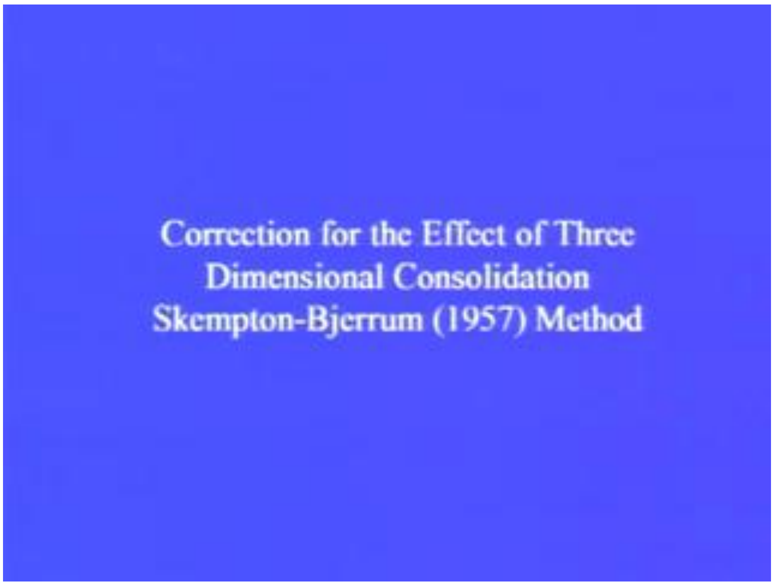

So, Skempton-Bjerrum has given a method by which we can apply a correction for the 3 dimensional consolidation. Now, here we determine consolidation settlement on the basis of 1 dimensional consolidation and then we correct it for 3 dimensional case.

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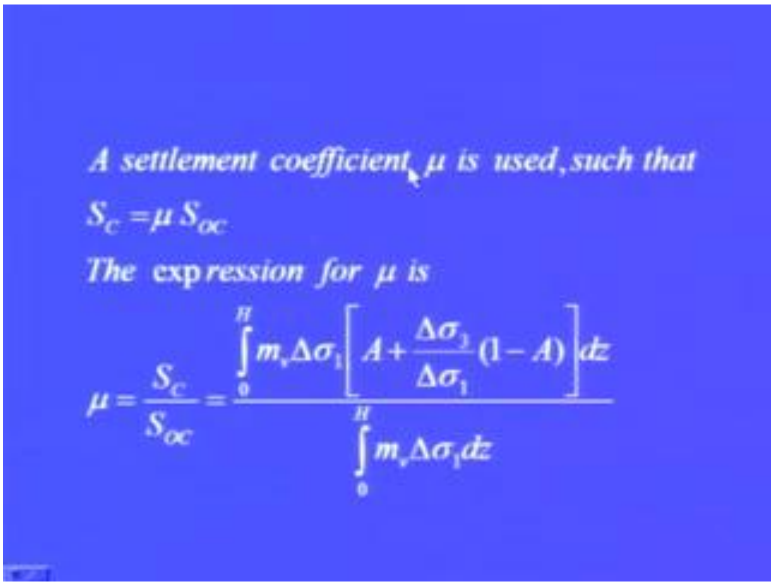

Now, they have suggested our settlement coefficient mu and this mu can be determine by this particular equation, so Sc equal to mu into Soc, where Soc is the settlement determined not the basic of odometer or the consolidation test.

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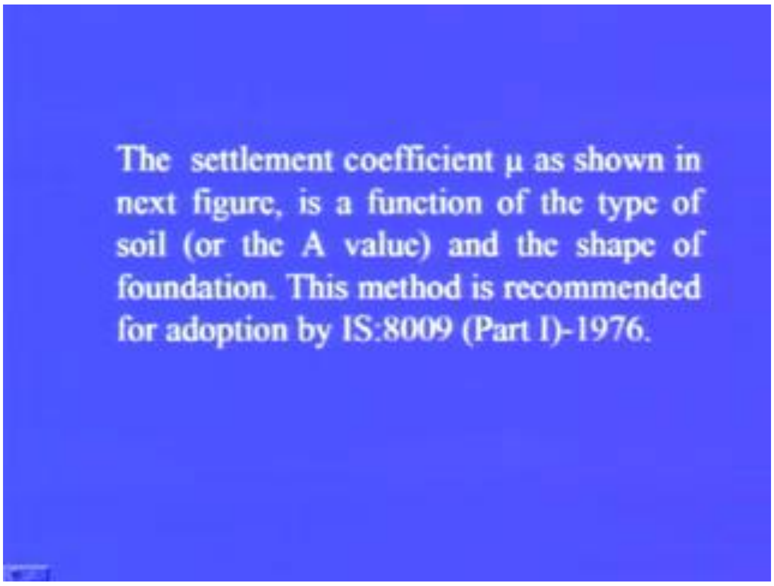

The settlement coefficient mu as shown in next figure is function of type of soil or the A value and the shape of foundation, this method is also recommended by the IS code is 8009 part I 1976. So, this is the plot between the settlement coefficient versus the core pressure coefficient for different values of H by b ratio.

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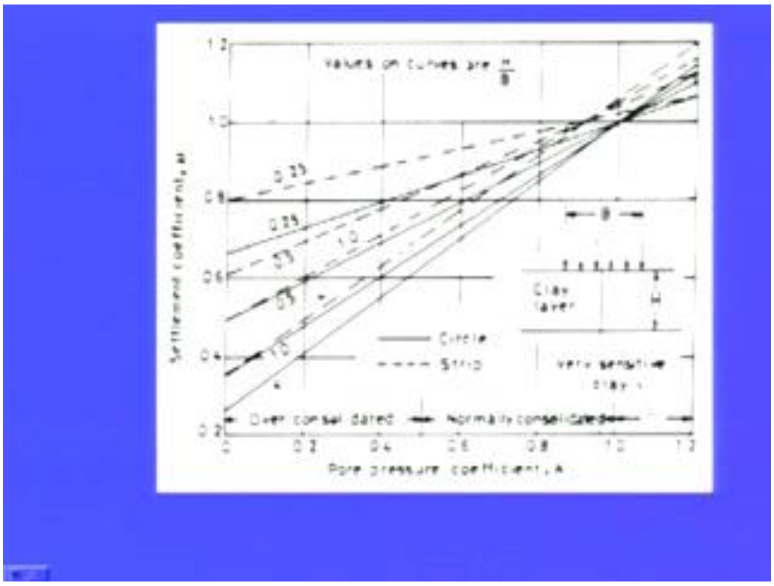

Now, for the circular footing as well as for the strip footing, so this coefficient mu can be obtained using this chart for the given value of the pore pressure coefficient a and depending up on the H the thickness of the clay layer and H by b ratio.

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Values of Settlement coefficient μ

Type of clay	μ
Very sensitive clays (soft alluvial and marine clays)	1.0 to 1.2
Normally consolidated clays	0.7 to 1.0
Over-consolidated clays	0.5 to 0.7
Heavily over-consolidated clays	0.2 to 0.5

The recommendations are also given in the tabular form, which can be used as guidelines for the values of settlement coefficient, like if we have a very sensitive clay, like soft alluvial and marine clay, then in that case this mu value can be taken as 1 to 1.2 For the case of normally consolidated clays, it is taken as point 0.7 to 1.0. For the case of over consolidated clays it is taken as 0.5 to 0.7, and for heavily over consolidated clays it is

0.2 to 0.5, so using this settlement coefficient mu, we can correct the settlement determine on the basis of consolidation settlement.

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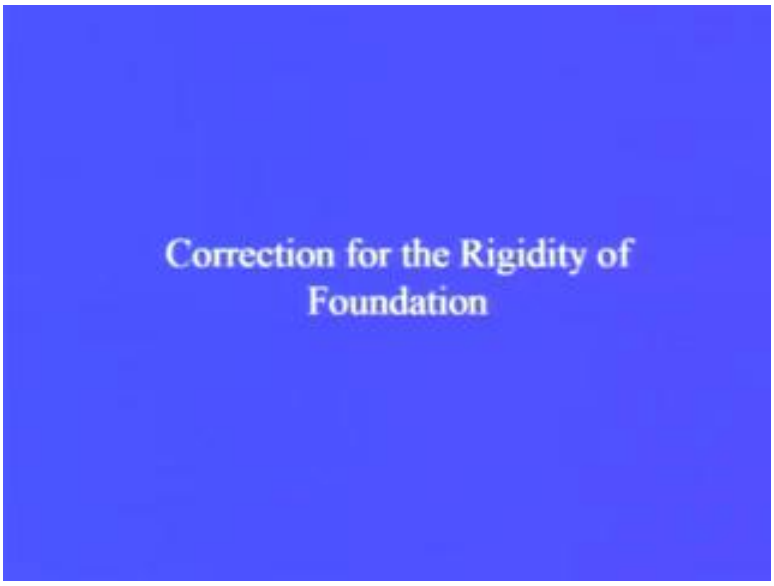

Then we also apply the correction for the rigidity of foundation.

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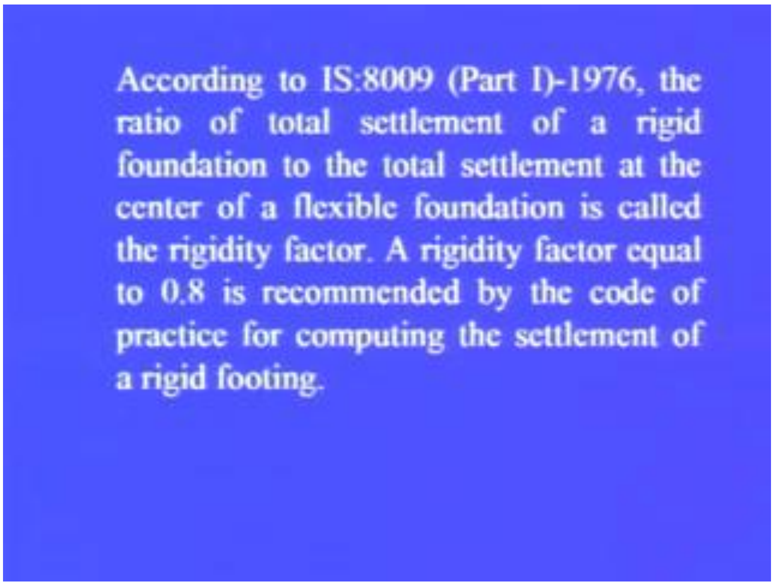

Now, here in the analysis we have determined the settlement for the flexible foundation and we have seen that the settlement behaviour as well as the pressure below the foundation for the rigid footing, as well as the flexible footing are different for sands and clays. So, we will have to apply a correction factor to account for the rigidity of foundation, now according to IS 8009 part I 1976. The ratio of the total settlement of a rigid foundation to the total settlement at the centre of a flexible foundation is called the rigidity factor.

And a rigidity factor equal to 0.8 is recommended by the code of practice for computing the settlement of a rigid footing, we also correct the settlement for the depth of embedment of foundation, whatever settlement we have determined we assume that the footing is placed on the surface.

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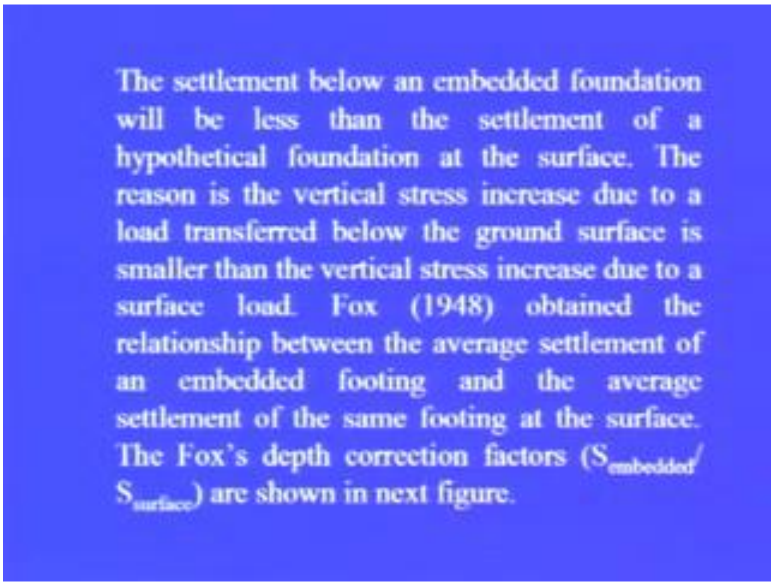

The settlement below an embedded foundation will be less than the settlement of a hypothetical foundation at the surface, the reason is the vertical stress increase due to a load transferred below the ground surface is smaller than the vertical stress increase due to a surface load. Fox in 1948 obtained the relationship between the average settlement of an embedded footing and the average settlement of the same footing at the surface. And he has suggested the depth correction factors as S embedded divided by S surface and these are shown in the next figure.

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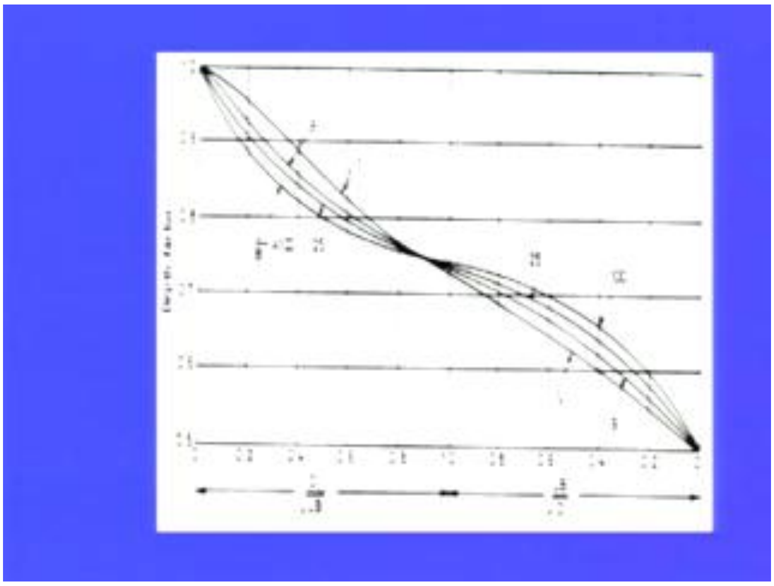

As you can see from this figure, that the depth factor is given here and that is the function of this ratio d upon under root lb and d is the depth of foundation, l is the length ratio is starting from l y b equal to 1 to l y b equal to 100 from these plots. We can obtain the depth factor and can use in the determination of the actual settlement of an embedded footing.

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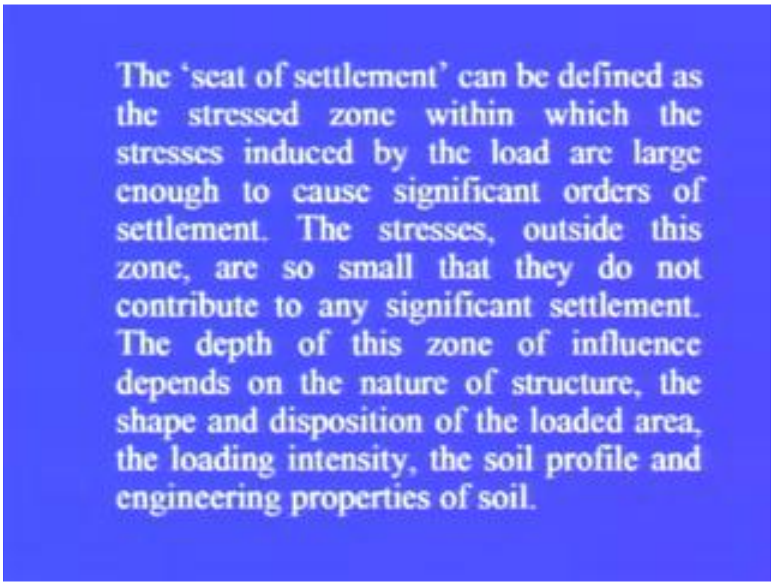

Then comes the seat of settlement, now the seat of settlement is can be defined as the stressed zone within which the stress induced by load are larger enough to cause significant orders of set settlement. The stresses outside the zone are so small that they do not contribute to any significant settlement, the depth of the zone of influence, depends on the nature of the structure the shape and disposition of loaded area, the loading intensity, the soil profile and the engineering properties of soil.

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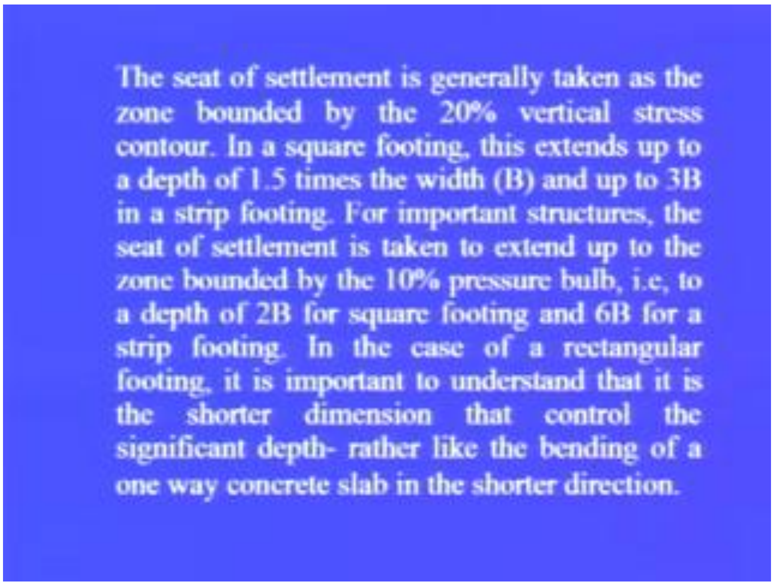

The seat of settlement is generally taken as the zone bounded by 20 percent vertical stress contour, in a square footing this extends up to a depth of about 1.5 times the width of the footing and up to 3 times B in the case of strip footing. For important structures, the seat of settlement is taken to extent up to the zone bounded by the 10 percent pressure bulb, that is to a depth of 2B for square footing and 6B for a strip footing. In the case of a rectangular footing, it is important to understand that it is the shorter dimension that control the significant depth, rather like the bending of one way concrete slab in the shorter direction.

It is more logical to relate, the seat of settlement to ratio delta sigma upon sigma dash, the ratio of increase in stress due to the in situ stress are the over burden pressure, one commonly used guideline is to take the seat of settlement as extending up to a depth, where the increase in stress due to structure load is 10 percent. And 5 percent for the important structures of the in situ stress before the application of the load. Next figure illustrates the use of pressure valve concept.

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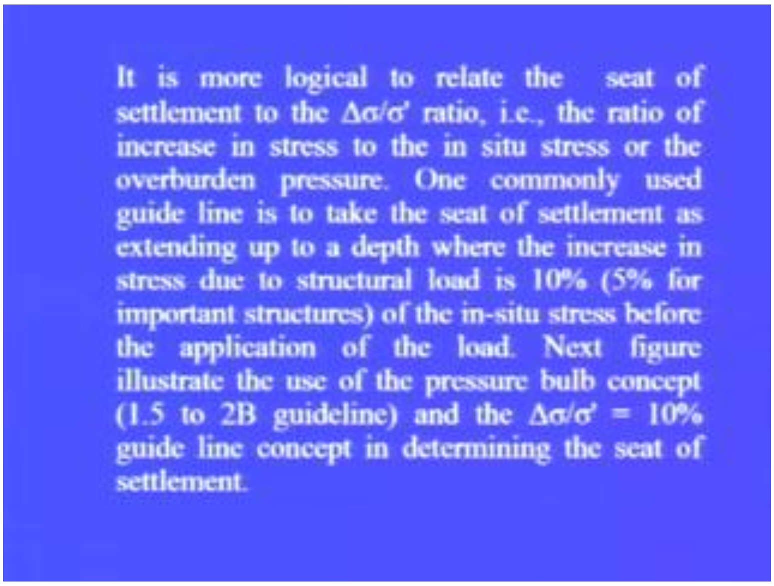

1.5 to 2B guideline and the ratio delta sigma upon sigma dash equal to 10 percent guideline concept in determining the seat of settlement.

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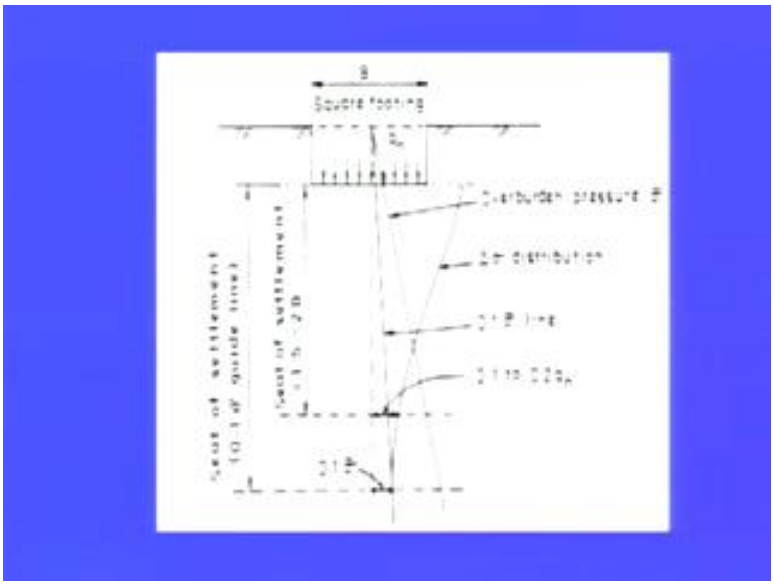

Now, as you can see from, this particular figure that this is the case of the square footing of the ((Refer Time: 14:14)) b. Now, this is the ground surface, then the variation of the overburden pressure, which can be determined as gamma into z is shown by this particular figure.

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Shown, by this particular figure by this straight line, and the sigma bar distribution is shown by this particular line, now if we consider the 10 percent of the pressure bulb then the 10 percent of the pressure bulb will give this 10 percent to 20 percent of q n here. But that will be the increase in pressure, increase in pressure due to the loading, which is placed at a foundation this particular level, so the seat of settlement will be of the order of 1.5 to 2B.

Now, if we consider the ratio of delta sigma bar divided by delta sigma bar divided by sigma dash, then this line it shows the relationship for 0.1, 10 percent of sigma bar line, so this is 10 percent of sigma bar line and the 10 percent of sigma bar is here. So, this is the seat of settlement and this will be about 0.1 percent of sigma bar can be considered as guideline to determine the seat of settlement.

So, we have seen that the n values etc we correct, corrected n values are used for a depth from here to here or here to here, whatever depth we decide depending upon the guidelines given. So, that, the most of the settlement is of immediate nature and the settlement which we determined for the case of the fine grain soils the settlements is consolidation settlement, but as the rate of consolidation or compression is very high in the case of the granular soils. So, we use the field methods to determine the settlement of foundations resting on such soils.

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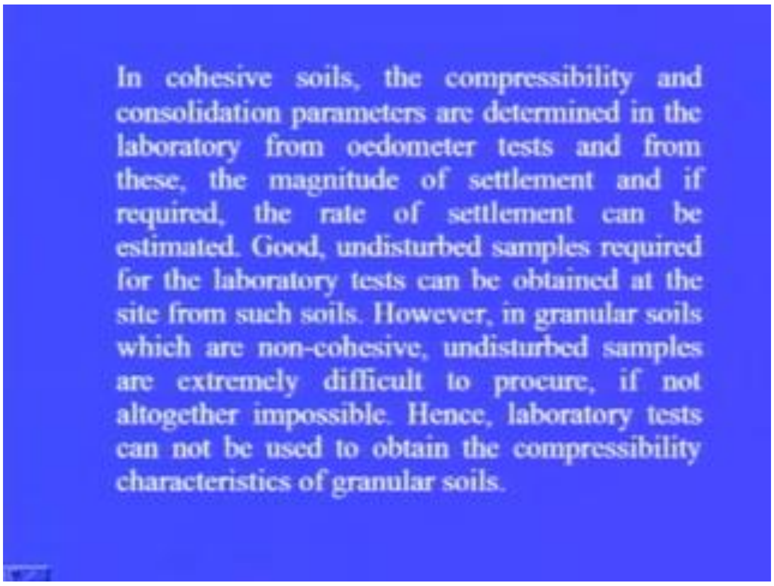

In cohesive soils the compressibility and consolidation parameters are determined in laboratory from odometer tests and from these the magnitude of settlement if required and also if required, the rate of settlement can be estimated. Good undisturbed samples required for laboratory tests can be obtained at the site, from for such soils. However in granular soils, which are non cohesive undisturbed samples are extremely difficult to procure, if not altogether impossible. Hence, laboratory test cannot be used to obtain the compressibility characteristics of the granular soil.

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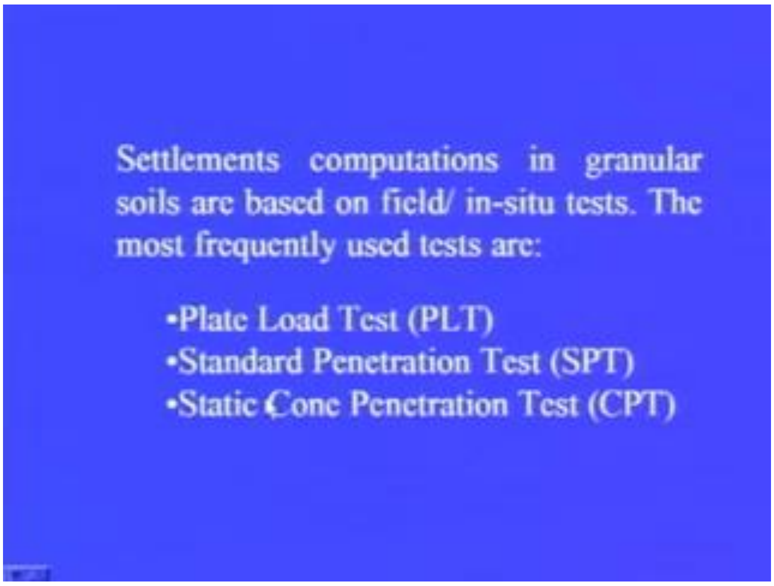

So, we go for the field test settlement computations in granular soils are based on fields or in situ test, the most frequently used test are. Plate load test, standard penetration test and the static cone penetration test. These test we have already discussed when we discuss the bearing capacity of granular soils.

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Now, we can determine the settlement of footing based on the plate load test method. Now, this is the set of which we have already discussed, but briefly I will discussed here also.

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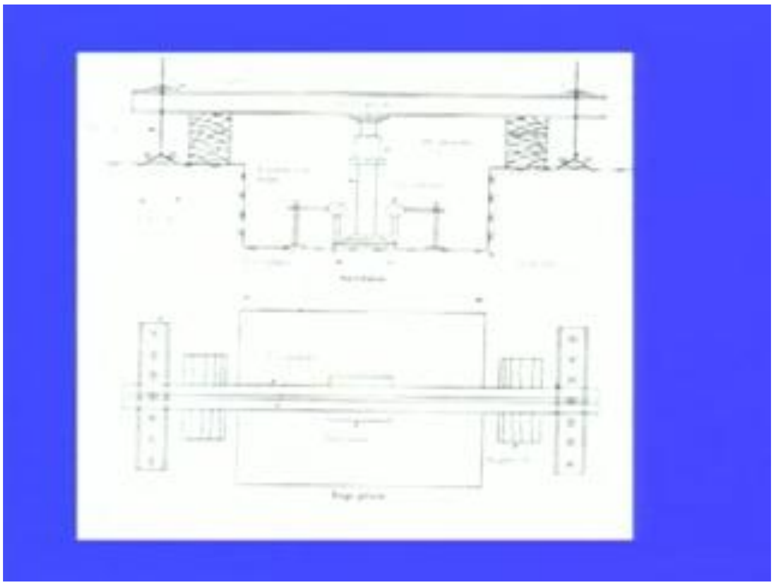

That, our plate is loaded and the settlement behaviour of this plate corresponding to different load intensity is observed and then a relationship is developed between the pressure and the settlement of the plate.

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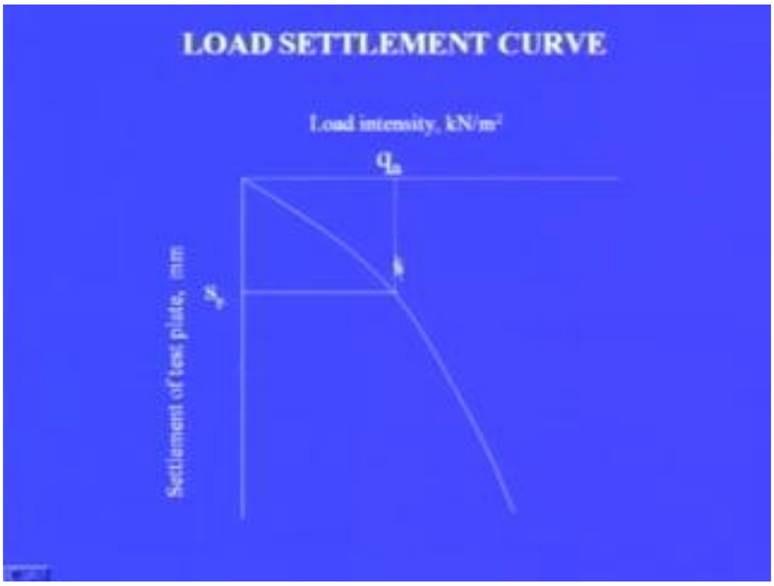

Now, a typical load settlement curve is shown here, here this is the load intensity kilo Newton per meter square and this is the settlement of the plate. So, corresponding to any load intensity, we find out the settlement of the plate and we get different points and we join them smoothly, we get the load settlement behaviour.

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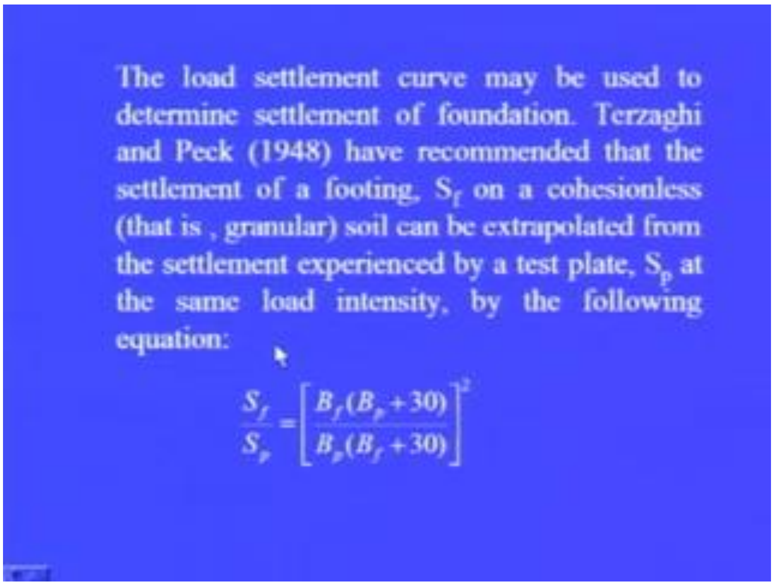

The load settlement curve may be used to determine settlement of foundation. Terzaghi and Peck in 1948 have recommended that the settlement of a footing, that is Sf on a cohesion less soil, that is the granular soil can be extrapolated from the settlement experienced by a test plate, that is the settlement of the plate Sp at the same load intensity. And the following equation can be used, equation is Sf upon Sp is equal to Bf plus Bp plus 30 divided by Bp Bf plus 30 whole square.

It may be shown from this equation, that in the case of granular soils, the settlement of the foundation cannot exceed above 4 times the settlement of a plate of 30 centimetre width, how so ever large it may be. However Jerome and Agustine have shown on the basis of several case records, that this equation is really valid for medium and densile, use of it for loose ends may lead to an under estimation of the settlement. So, we will have to be very cautious when we determined, settlement of loose end using this particular equation.

It may again be recalled, that the plate load test being short duration test, the settlement measured is only the intermediate settlement, in granular soils immediate settlements can be taken as total settlement, while in cohesive soils consolidate settlement which constitutes most part of the total settlement cannot be predicted through this test. Hence, the plate load test is not of much reverence in clayey soils, for is the settlement criterion very important in the determination of allowable varying pressure of a foundation.

Now; however, we can use the data obtained from plate load test, in the case of cohesive soils also or the fine grain soils also.

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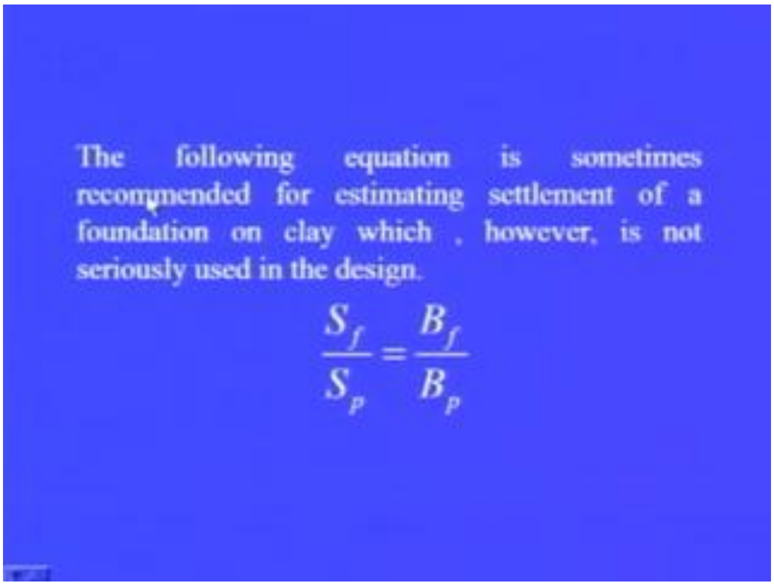

And following equation is sometimes recommended for estimating, the settlement of a foundation on clay and that is, not actually seriously used in the design. That is Sf upon Sp equal to Bf upon Bp, where Bf is the width of the foundation, Bp is the width of the test plate, Sf is the settlement of the foundation and Sp is the settlement of the test plate.

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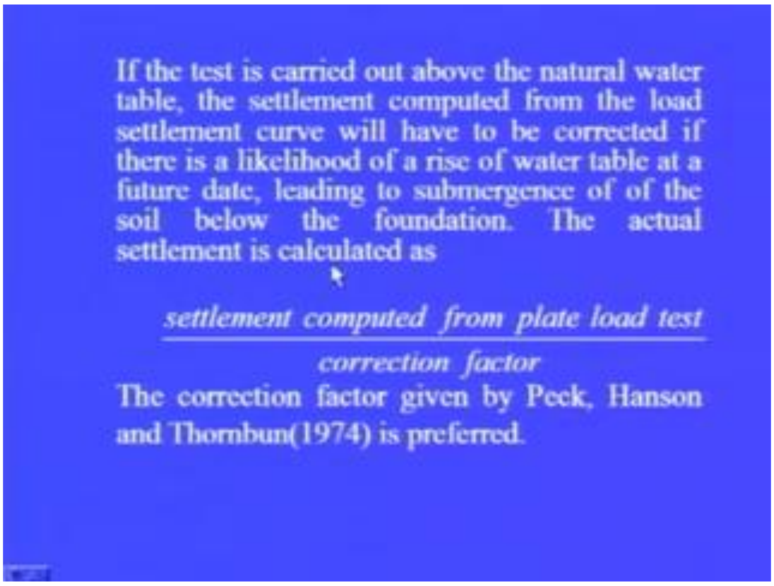

If, the test is carried out above the natural water table, the settlement computed from the load settlement curve will have to be corrected if there is a likelihood of a rise of water table at a future date. Leading to submergence of the soil below the foundation, the actual settlement is then calculated, as settlement computed from the plate load test divided by the correction factor. And this correction factor is given by Peck Hanson and Thornbun, which we have already discussed is preferred.

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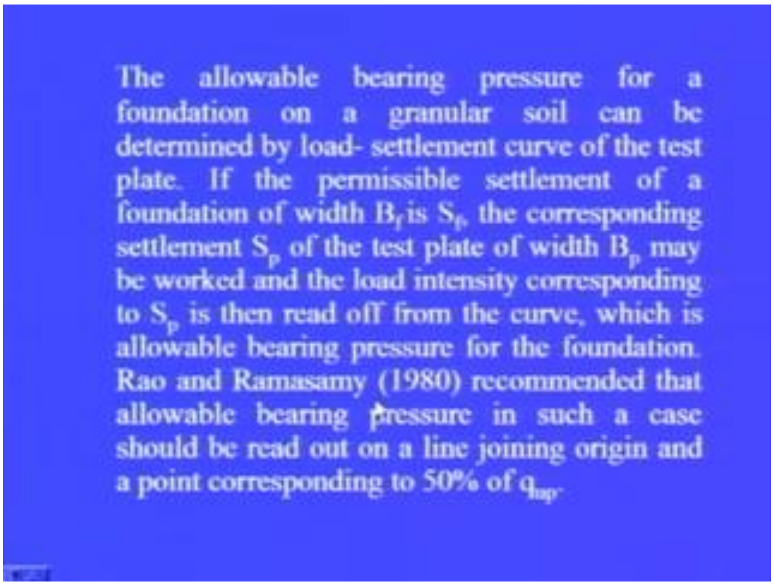

The allowable pressure for a foundation on a granular soil can be determined by load settlement curve of the test plate, if the permissible settlement of the foundation of the width Bf is Sf, the corresponding settlement Sp of the test plate of width Bp can be worked out from the equation. And the load intensity corresponding to Sp is then read off from the curve, which is allowable bearing pressure for the foundation.

Rao and Ramasamy in 1980 recommended that allowable bearing pressure in such a case should be read out on a line joining origin and a point corresponding to 50 percent of Qup, that is the ultimate varying capacity of the test plate. Now, it will become more clear by this solved example.

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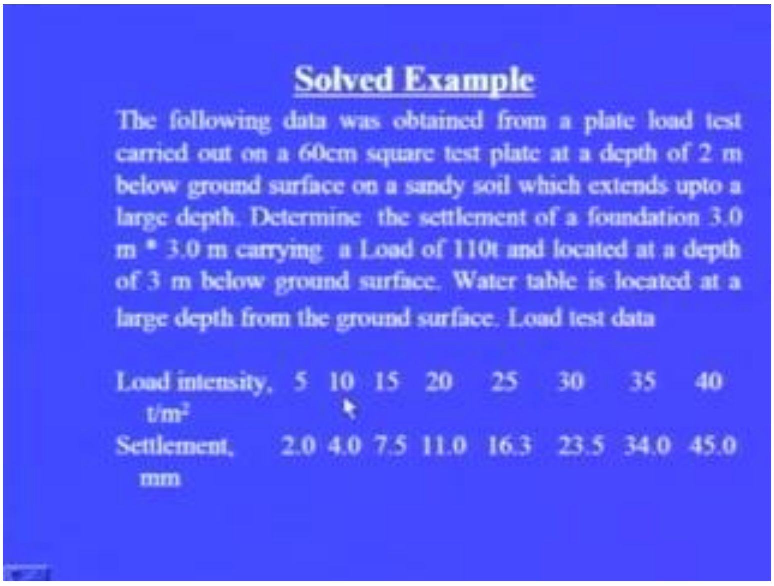

How to use this plate load test, for determination of the settlement of the footing resting on granular soil? The following data was obtained from a plate load test carried out on a 60 centimetre square test at a depth of 2 meter below ground surface on a sandy soil which extents up to a large depth. Determine the settlement of the foundation 3 meter by 3 meter carrying a load of 110 tonnes and located at depth of 3 meter below ground surface.

Water table is located at a large depth from the ground surface, then the load test data is given in the form of load intensity and settlement like for 5 ton per meter square settlement of the plate is 2 millimetre. For 15 it is 7.5, for 30 it is 23.5 and 40 it is 45 millimetre. So, likewise this data is given, we will use this data to plot load intensity verses settlement curve and then obtain the settlement of the foundation. The load settlement curve is shown in this figure.

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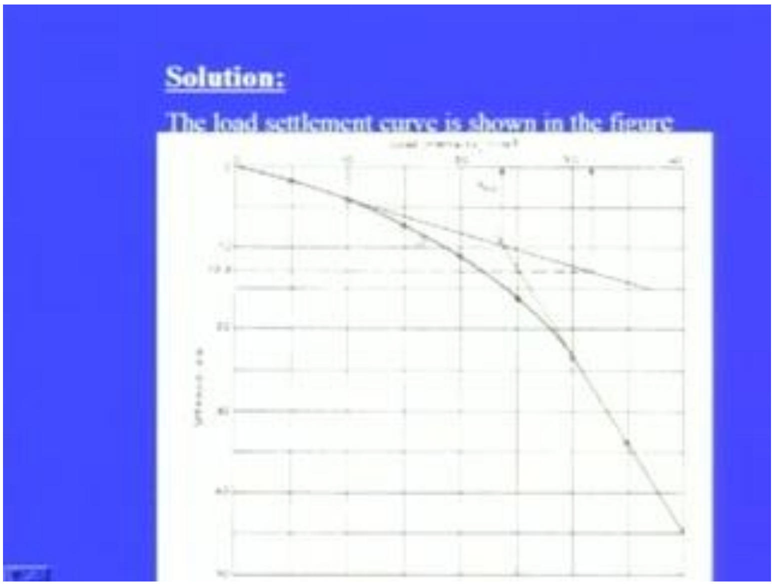

So, from this data we obtain a relation between the load intensity, which is given in tonnes per metre square and the settlement in millimetre, we will get different points, we join these points smoothly, we will get the load settlement behaviour of this. In order to determiner ultimate varying capacity, we go for the double tangent method, so we take straight line portion of this part of the curve and this part of the curve, wherever these two tangents meet, that is the ultimate varying capacity of the plate.

And we make use of this ultimate varying capacity of the plate to find out factor n gamma and then we determine phi from there and then nq and n gamma we use for actual foundation to determine ultimate varying capacity of the foundation. Now, Ramasamy and Rao have recommended, that this allowable bearing pressure should be determined should be read off for from a straight line joining this with 50 percent of ultimate varying capacity of the plate. So, we will be using this for determining the allowable bearing pressure. Right now, we want to determine the settlement of the foundation.

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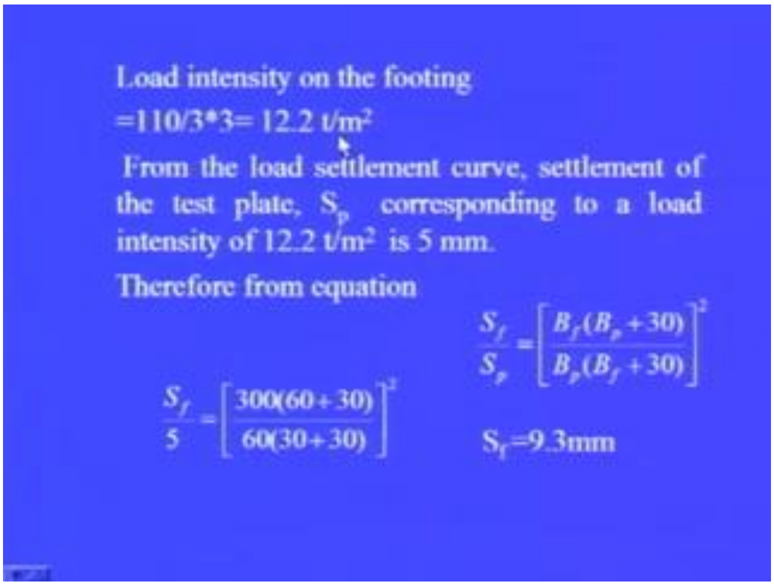

So, first of all we determine, what is load intensity on the footing, load intensity on the footing is the total load that is 110 tonnes divided by the area of the footing that is 3 meter by 3 metres square footing it comes out to be 12.2 ton per meter square. So, from the load settlement curve corresponding to 12.2 ton meter square, we find out what is the settlement of the plate and the settlement of the plate, from this figure comes out to be 5 millilitres.

Now, we use this equations Sf upon Sp that is equal to Bf plus Bp plus 30 Bp divided by Bf plus 30 whole square, now here width of the plate is 60 centimetre and the width of the actual foundation is 300 centimetre. So, we use this data here and the settlement of the foundation which we have already determined for 12.2 ton per meter square from the load settlement curve that is 5 millimetre, when we substitute all these values here, we find that Sf the settlement of the foundation comes out to be 9.3 millimetre.

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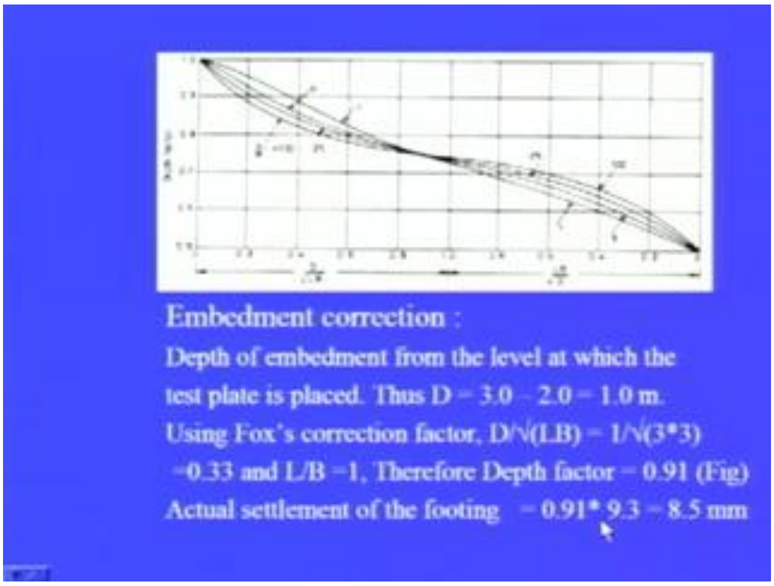

Now, then we apply an embedment correction in order to use this embedment correction, we use the chart which is given by fox and is recommended by IS 8009 part I 1976. So, from this chart, first of all we find out what is the depth of embedment and the depth of embedment is equal to 3 minus 2 that is equal to 1 meter. By using fox correction factor, first we find out D upon under root LB, so then D upon under root LB, it is equal to 1 divided by under root 3 into 3 that is equal to 0.33.

And this is the case of a square footing L by B ratio is equal to 1, so 0.33 be see from here and for a square footing this ratio is 1, so this particular line or the graph is used, and from this we can see, that it comes out to be about 0.91. So, depth factor from this figure we can observe and that is 0.91, so actual settlement of the footing when we apply this embedment correction, that will be equal to 0.91 into 9.3 equal to 8.5 millimetre.

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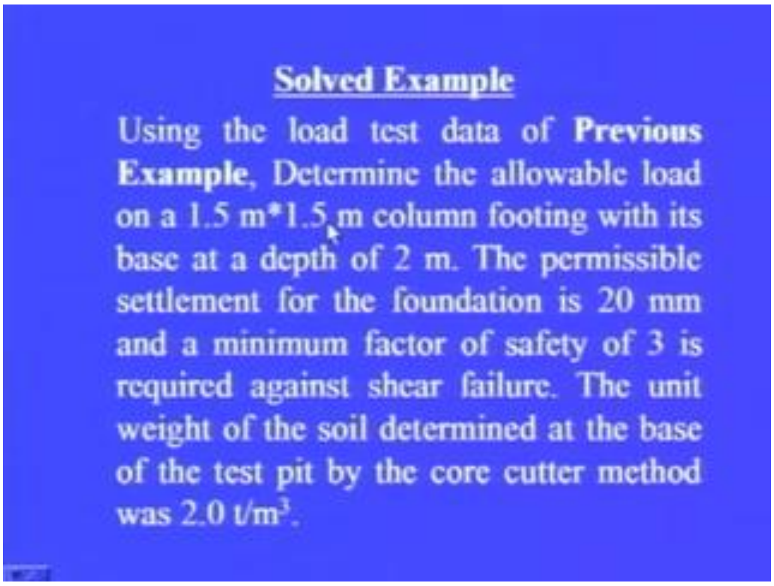

Now, using the load test data of previous example, we can also determine the allowable load on a 1.5 meter by 1.5 metre column footing, with its base at depth of 2 meter. The permissible settlement for the foundation is 20 millimetre and minimum factor of safety of 3 is required against shear failure. The unit weight of the soil determined at the base of the test pit by core cutter method was found to be 2 ton per meter cube. Now, we know that, the allowable bearing pressure of a foundation is the minimum or the least value out of two criteria.

One is the soil should not fail in shear, that is the bearing capacity criteria and the soil should not experience a permissible settlement more than the permissible settlement, so we will have to apply these two here and the we will have to find out, what is the allowable bearing pressure and then the load, the column can carry.

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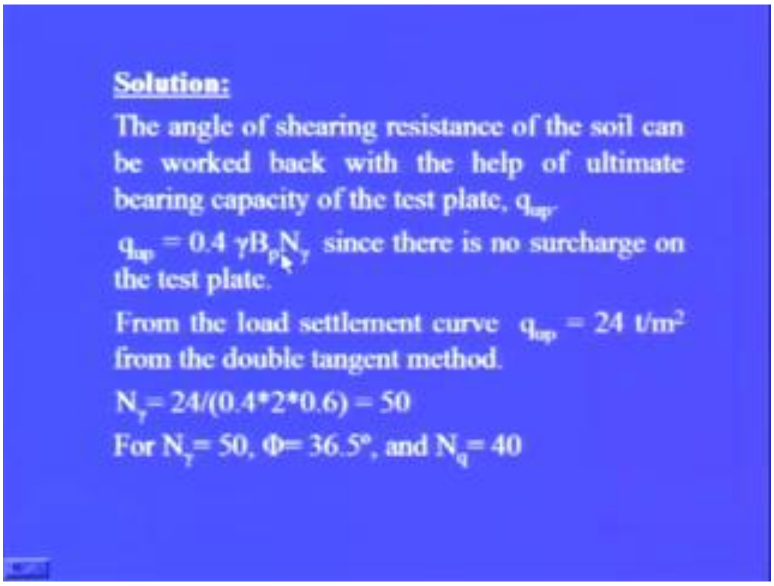

The angle of shearing resistance of the soil can be worked back with the help of ultimate bearing capacity of the test plate Qup, this Qup is determined as I said is determined by the double tangent method and in this particular case, it comes out to be Qup is equal to 24 ton per meter square. And for a case of a square footing, we can use this relationship for the ultimate varying capacity of the plate as 0.4 gamma B N gamma. Since there is no such charge on the test plate and C equal to 0.

So, we equate 0.4 gamma B N gamma, B is the width of the plate Bp with this 24 ton per meter square and only unknown is N gamma, so N gamma can be worked out, it comes out to be 50. Now, from the Peck Hanson Tohrnbun plots between the N gamma N Q and phi valve we can find out for this particular value of N gamma that is 50, we can work out phi and that comes out to be equal to 36.5 degrees and corresponding to this N q is equal to 40.

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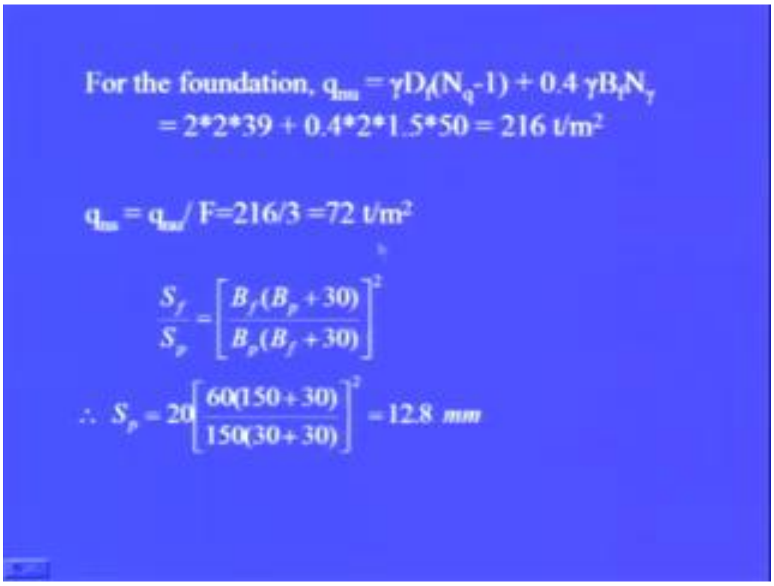

So, for the foundation net ultimate varying capacity of the foundation is given by this relationship, gamma Df Nq minus 1 plus 0.4 gamma Bf N gamma because it is I square foundation, so when we substitute, all the known values in this equation, we will get net ultimate varying capacity of the foundation that comes out to be 216 ton per meter square. Now, the net F can be found out by dividing it with a factor of safety and that is given as 3.

So, when we divided it by 3, it comes to be 72 ton per meter square, now in order to find first of all we find out, what is the settlement of the plate for 72 ton per meter square. Now, that comes out to be equal to 12.8 millimetre.

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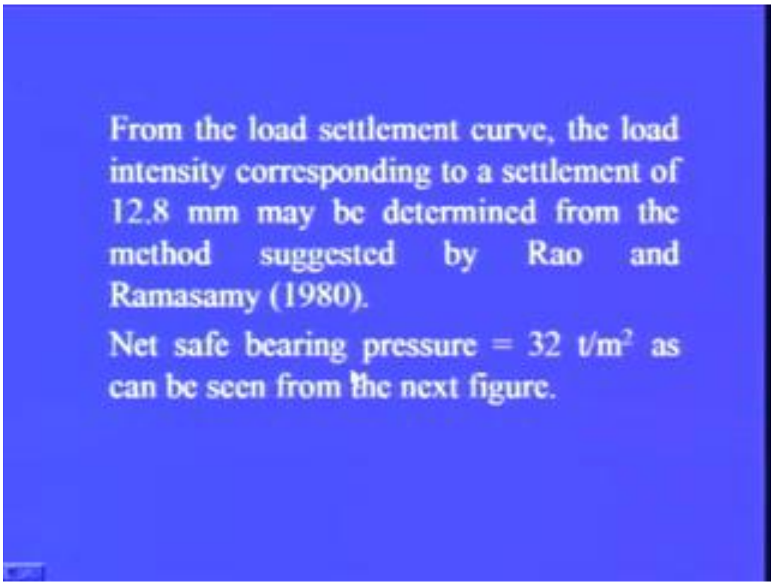

Now, from the load settlement curve, the load intensity corresponding to settlement of 12.8 millimetre may be determined from the method suggested by Rao and Ramasamy. The net safe bearing pressure that is equal to 32 ton per meter square as can be seen from the next figure.

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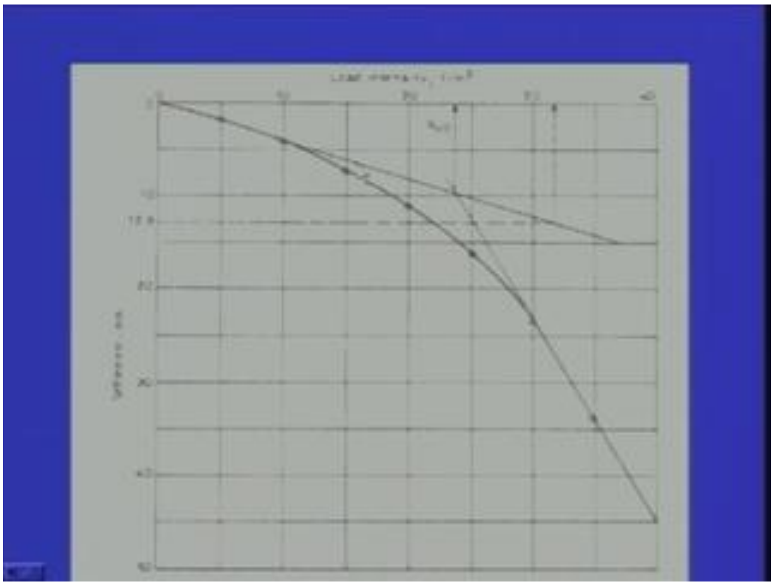

So, corresponding to 12.8 millimetre of settlement, we use the approach used by Rao and Ramasamy, we, simply draw a straight line from the origin and we join a point corresponding to 50 percent of the ultimate varying capacity of the plate, and when we extend that, we get this straight line. And we read the value of the bearing pressure corresponding to 12.8 millimetre of settlement and that comes out to be about 32 ton per meter square.

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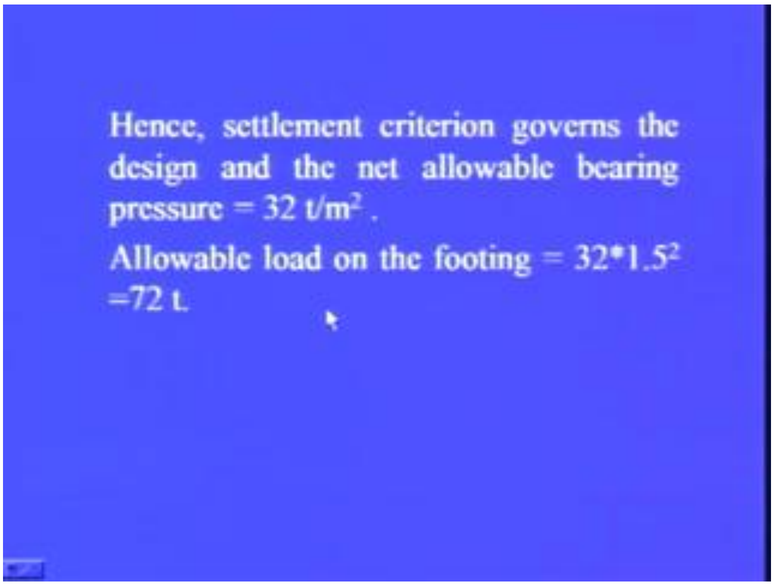

Hence the settlement criterion governs, the design and the net allowable bearing pressure is equal to 32 ton per meter square, so allowable load on the footing that will be equal to 32 multiplied by the area of the footing comes out to be 72 tons. Now, there is another popular test which we use in the case of the foundations resting on granular soils and we use this for determination of settlement that is the standard penetration test.

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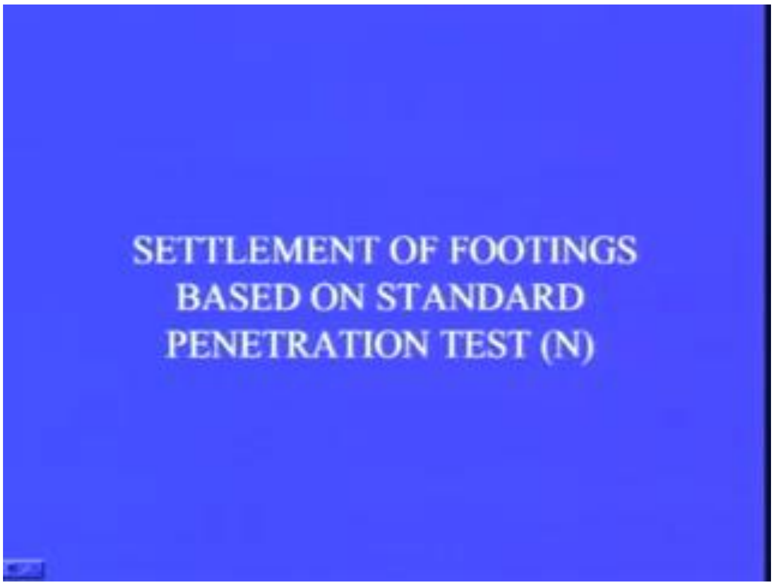

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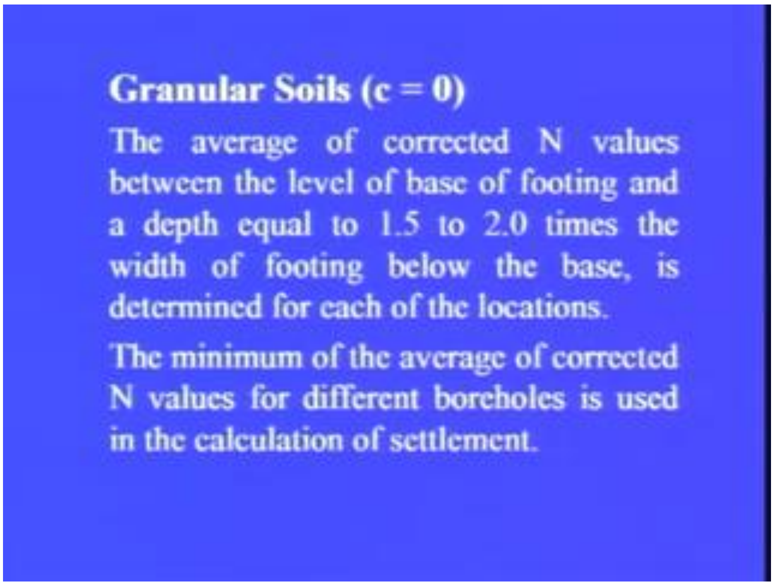

Now, for the case of granular soils, the average of the corrected N values between the level of the base of footing, and the depth equal to 1.5 to 2 times that is the seat of the settlement, the width of the footing below the base is determined for each of the locations. The minimum of the average of the corrected N value for different boreholes is used in the calculation of settlement. IS 8009 part I 1976 gave a chart for the calculation of settlement per unit pressure.

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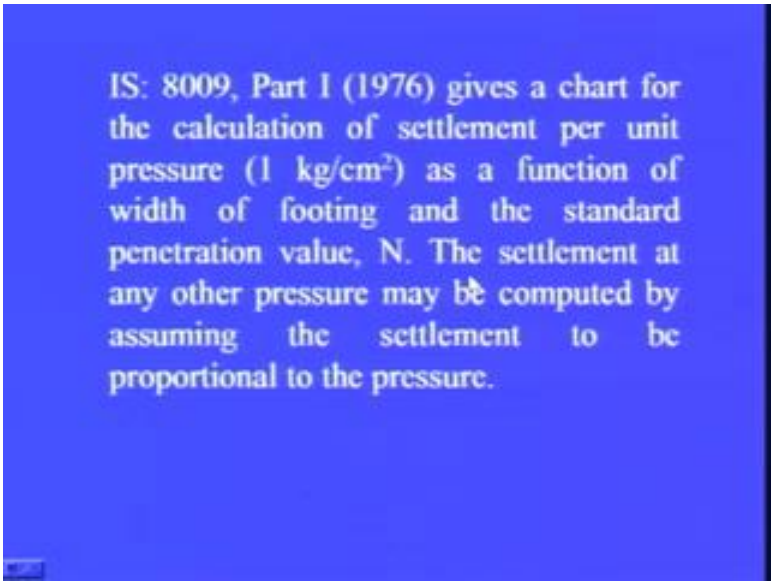

And that unit pressure is 1 kg per centimetre square, as a function of the width of the footing and the standard penetration test value N. The settlement at any other pressure may be computed by assuming the settlement to be proportional to the pressure.

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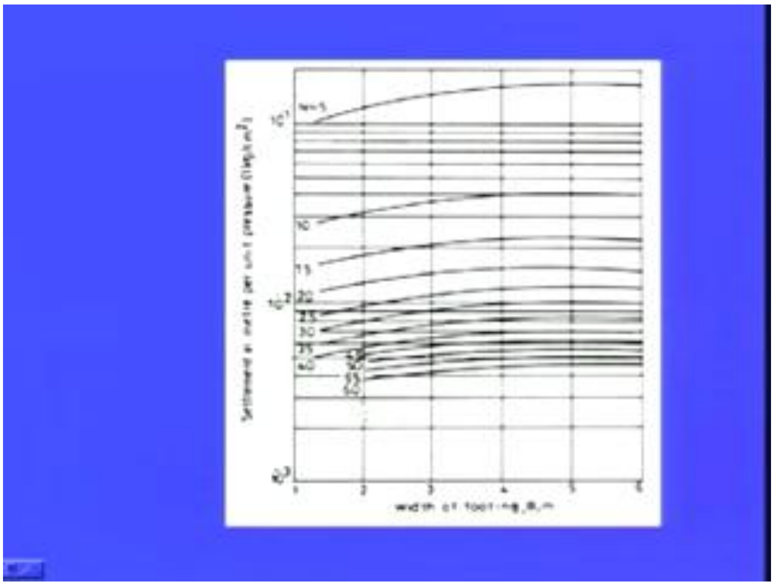

Now, this is the relationship suggested in IS code, that is the settlement per meter per unit pressure kg per centimetre square versus the width of footing for different values of N. So, here this is from N equal to 5 to N equal to 60. Now, we can use this particular chart for a given dimension of the footing and for the object and the corrected value of N. We can find out, what will be the settlement corresponding to unit pressure and we multiply it by the actual load intensity and then we calculate the actual settlement of the footing foundation. Now, for the values between like here...

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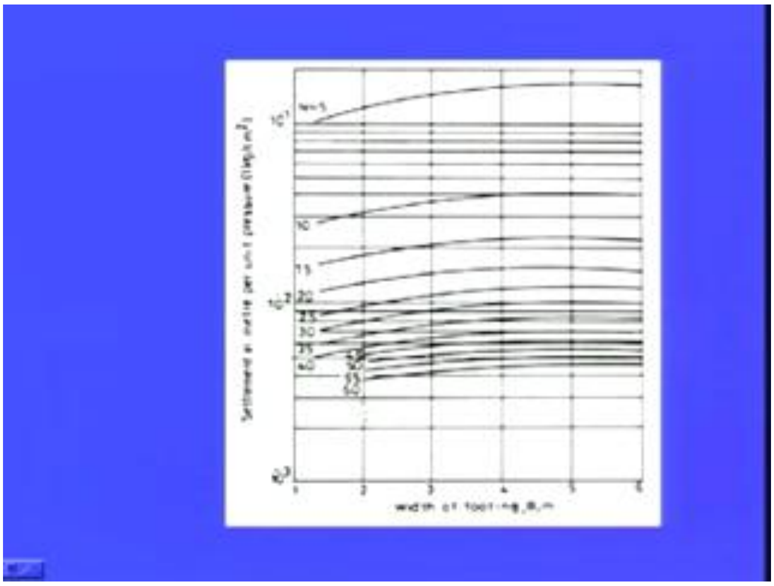

The relationship is given for N at a interval of 5, so for intermediate values can interpolate the value of the settlement, one more thing is obvious from this particular figure that if the width of the foundation is more than 5 meter for all values all N values you will find that the settlement is almost equal, whatever may be the width of the footing. If the natural water table is close to the base of foundation or correction factor in the form suggested as below as per IS 8009 is applied.

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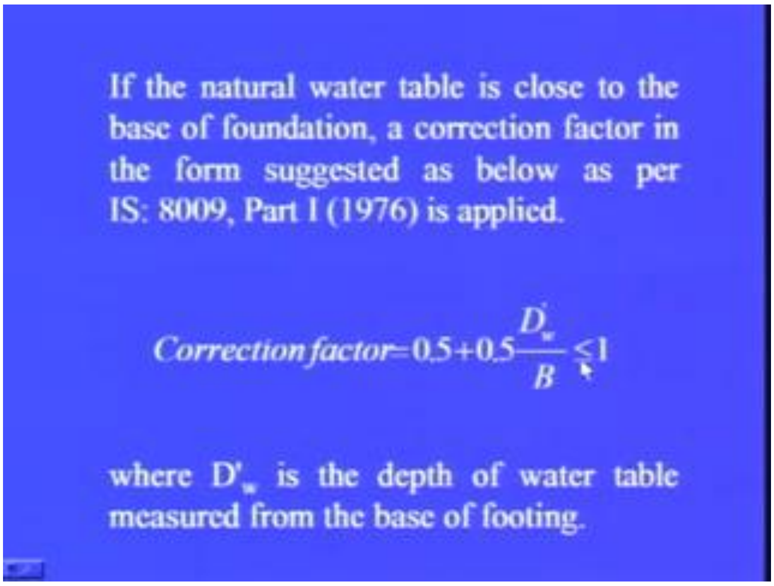

That correction factor is 0.5 plus 0.5 D w dash by B, where D w dash is the depth of the water table measured from the base of footing and it will always be less than or equal to

1. We can also obtain settlement of footings based on the static cone penetration test that is the CPT values or Qc values.

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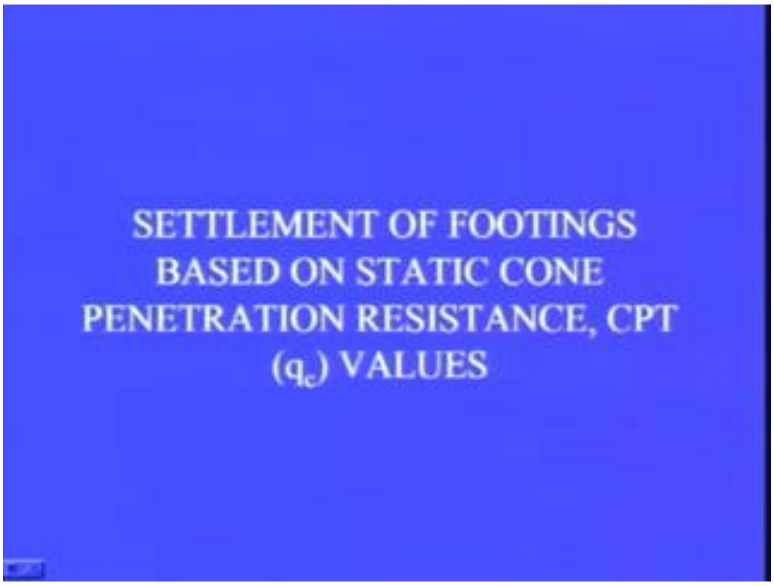

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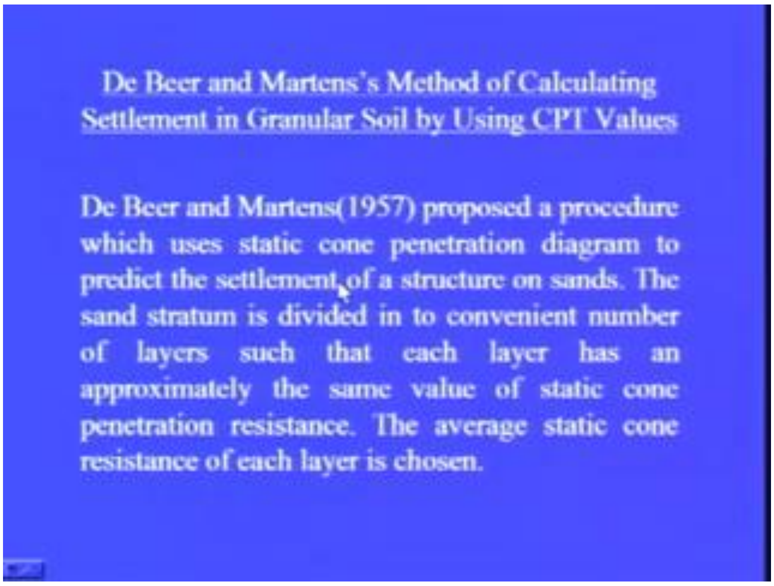

There are two approaches one is given by De Beer and Marten and another is given by Sparkman, the De Beer and Marten 1957 proposed a procedure, which uses static cone penetration diagram to predict the settlement of a structure on sands. The sand stratum is divided in to convenient number of layers such that each layer has an approximately the same value of static cone penetration resistance. The average static cone penetration resistance of each layer is chosen for the calculation of settlement.

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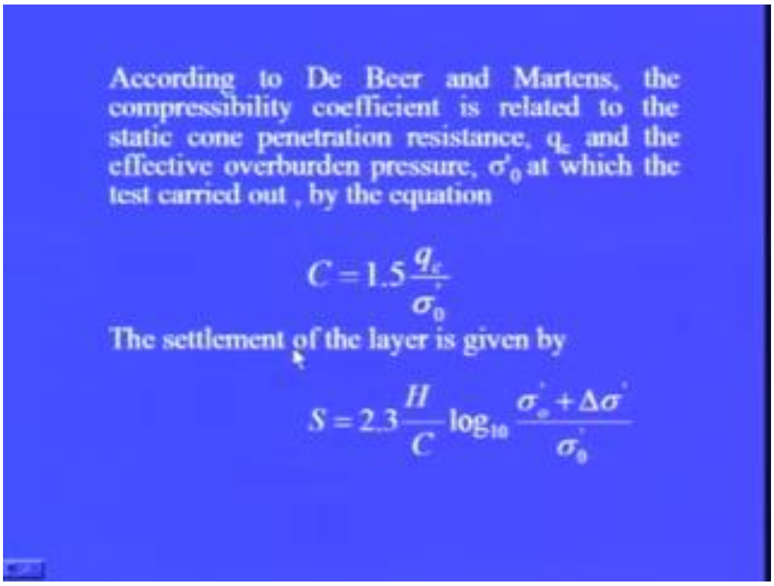

According to De Beer and Martens, the compressibility coefficient is related to the static cone penetration resistance value that is Qc and the effective over burden pressure sigma 0 dash at which the test is carried out and the equation is C equals to 1.5 Qc up on sigma 0 dash. Then the settlement of the layer can be calculated by this equation S equals to 2.3 H up on C, where H is the thickness of that stratum log to the base 10 sigma 0 dash plus delta sigma dash upon sigma 0 dash where sigma 0 dash is the initial over burden pressure. And delta sigma dash is the increase in pressure due to the loading at that particular level and that level is normally taken as the middle of the layer.

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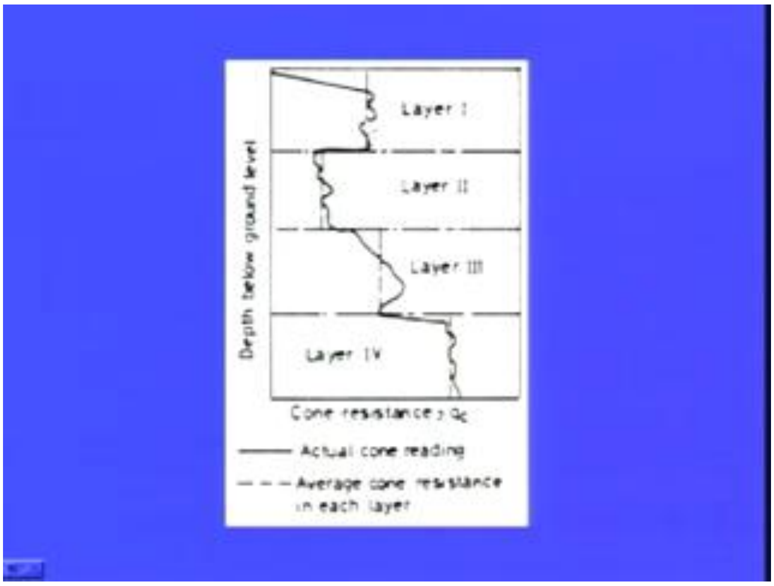

Now, here we divide this into number of layers having almost same value of Qc, so this is one such variation of Qc with depth in different layers. So, depending up on the variation, we can divide this in to number of layers and the average value is taken as the Qc for that particular layer.

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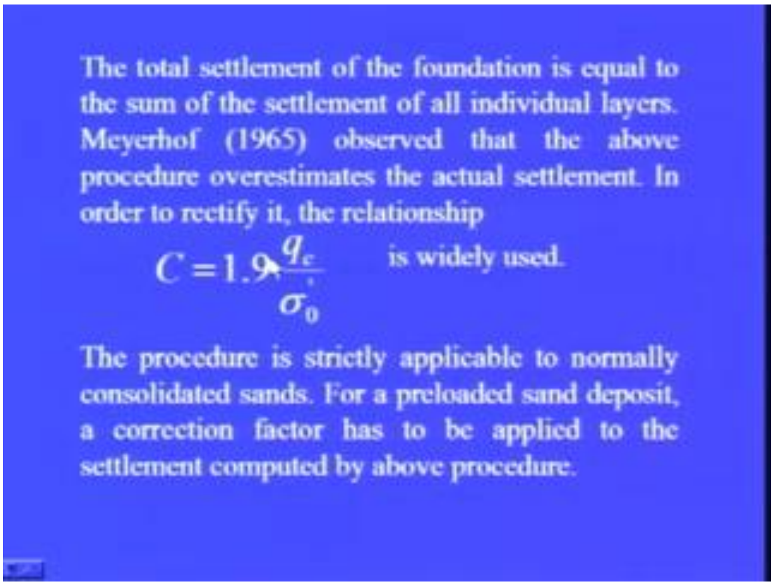

The total settlement of foundation is equal to the some of the settlement of all individual layers, Meyerhof observed that the our procedure over estimates the actual settlement, in order to rectify it the relationship in place of 1.5 is 1.9 is widely used. The procedure is strictly applicable to normally consolidated sands, for a preloaded sand deposit, a correction factor has to be applied to the settlement computed by the our procedure.

The Schmertmann method of calculating settlement in granular soil by using CPT values, Schmertmann pointed out that the distribution of vertical strain below the centre of a square or circular footing, that is the case of Asymmetric on a sand can be simplified in the characteristic manner shown in the next figure. The seat of settlement is taken as equal to 2 b and 4 b below the base of foundation. The soil layer is divided into number of convenient layers of thickness delta z and the average static cone penetration resistance of each layer is determined.

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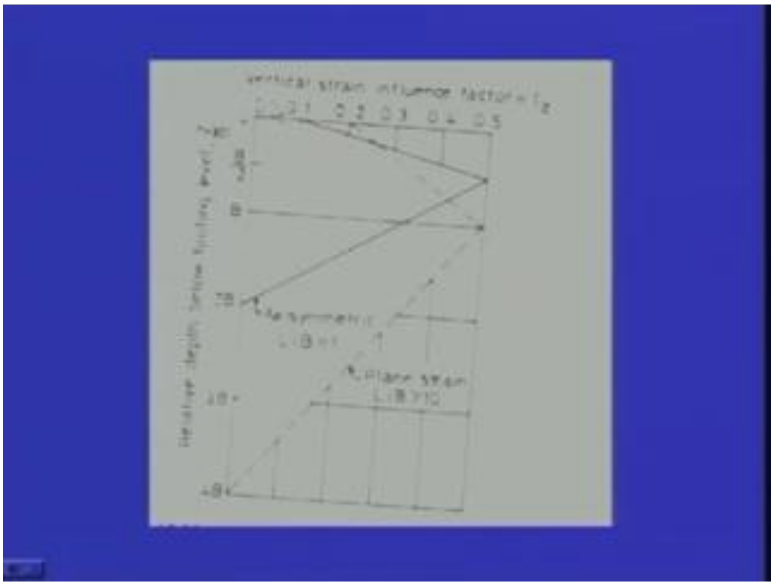

So, in this figure you can see the vertical strain influence factor I z, and the related tap below the footing that is ratio z up on d, so this is given in the non dimensional form. Now for the case for the circular footing, Asymmetric case, this is the relationship and for the square or the strip footing for the plain strain case for L by B greater than 10 this is the relationship Now, here the seat of the settlement is taken 2 B or 4 B. The maximum value for this particular case, in the case of asymmetric case maximum value of a strain value is 0.5 and it occurs at a depth of d by 2

Whereas, for the case of the plain strain, that is the case of the a stiff footing, this min this value is 0.2 at the surface and it is maximum 0.5 at a depth of d and again 0 at a depth of 4 B. Schmertmann gave following equation for calculating the settlement.

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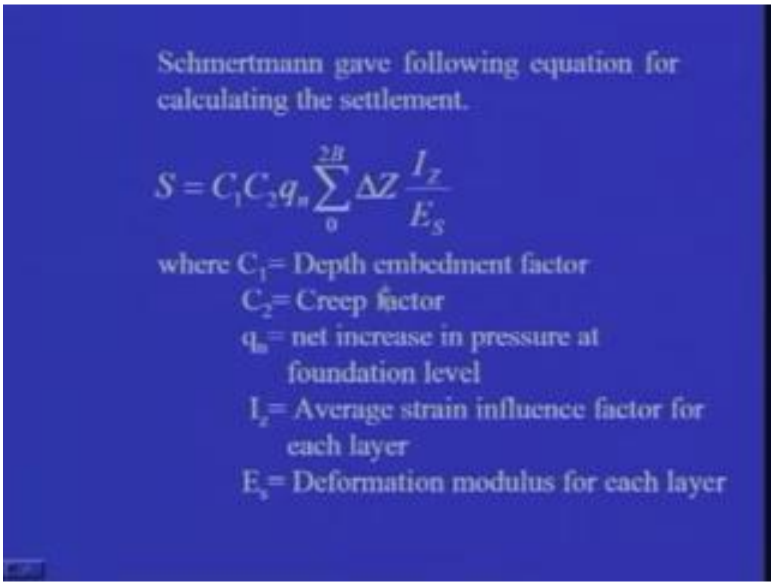

S equal to C 1 C 2 Qn and for all these layers from summation from 0 to 2 B that is up to the seat of settlement, delta Z into Iz up on Es where, C 1 is the depth embedment factor, C 2 is the creep factor and qn is the net increase in pressure at the foundation level, Iz is the average strain influence factor for each layer that can be obtained from the previous figure. And Es is the deformation modulus for each layer.

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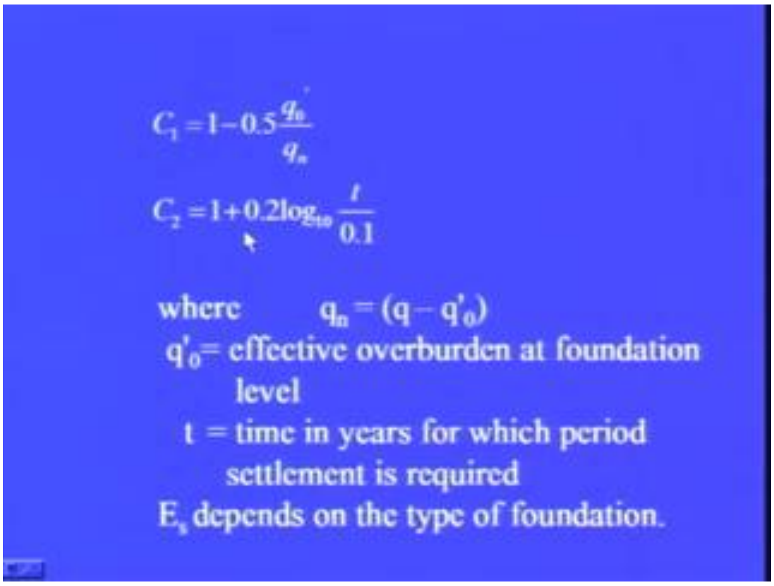

Now, this C 1 is equal to 1 minus 0.5 q 0 dash upon q n, whereas C 2 is equal to 1 plus point 2 log to the base 10 t up on 0.1, where q n is the grass pressure intensity minus the initial over burden pressure at that particular level, so q minus q 0 dash, q 0 dash is the effective over burden at the foundation level. Now, this t is the time in years for which period settlement is required and Es depends on the type of foundation.

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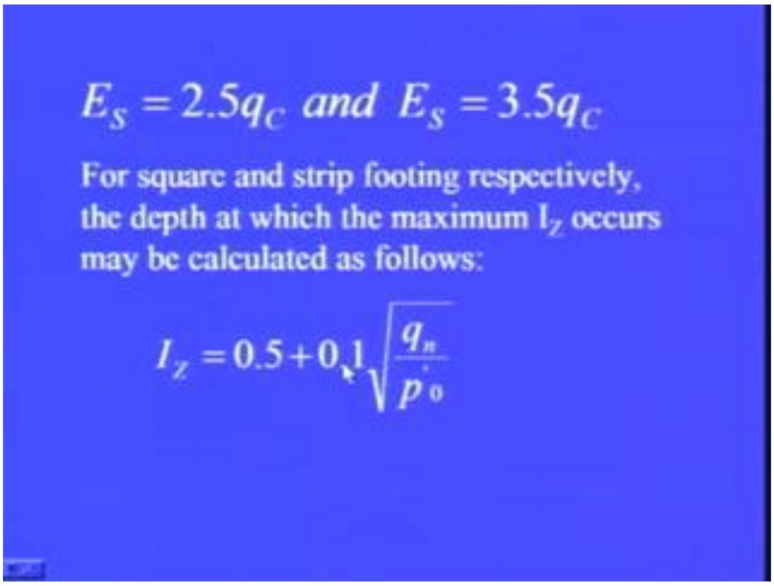

The correlation for Es are Es equals to 2.5 q C and Es equal to 3.5 qc for a square in a square footing respectively, so this is for square footing and this is for the strip footing.

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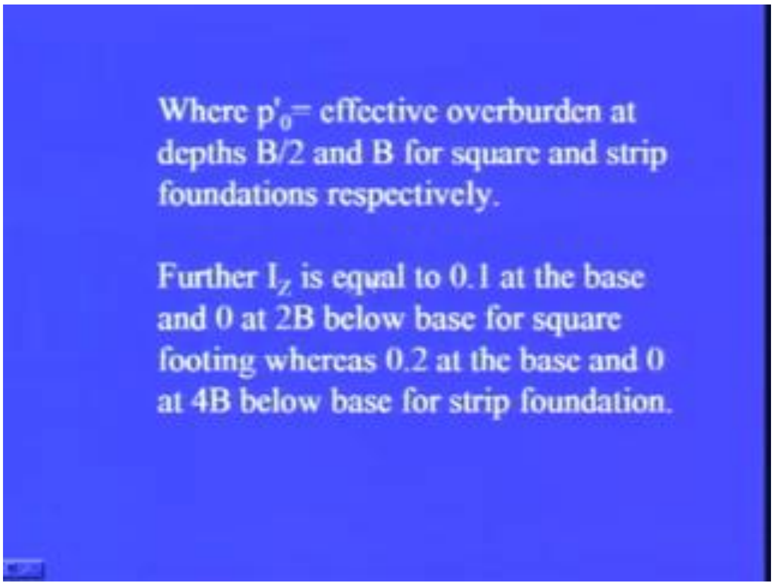

The depth at which the maximum I z occurs may be calculated as follows, Iz equals to 0.5 plus 0.1 under root of qn up on p 0 dash where p 0 dash is the effective over burden at depths B by 2 and B for square and strip foundations respectively, that we have seen in the last figure. Further Iz is equal to 0.1 at the base and 0 at 2 B below base for the square footing whereas, it is 0.2 at the base and 0 at 4 B below the base for the case of strip foundation.

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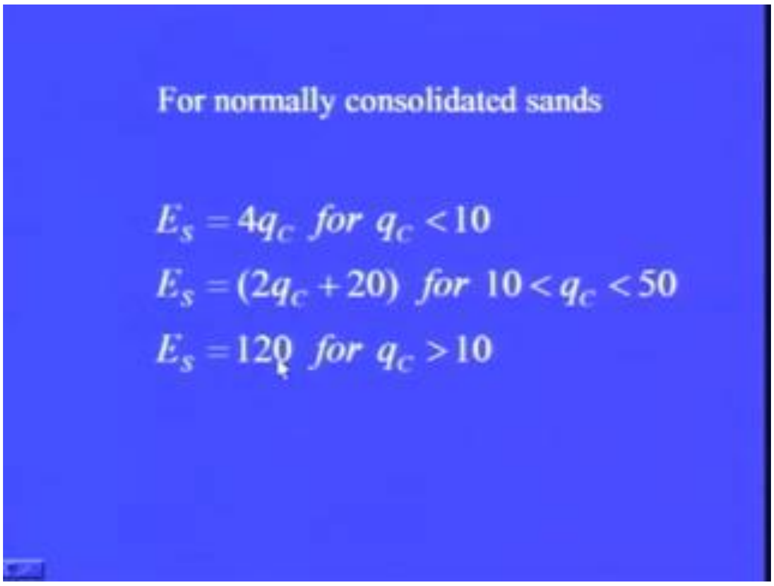

For normally consolidated sands, Es can be taken as 4 qc for values of qc less than 10 and it is 2 qc plus 20 for qc values between 10 and 50 and it is 120 for qc greater than 10.

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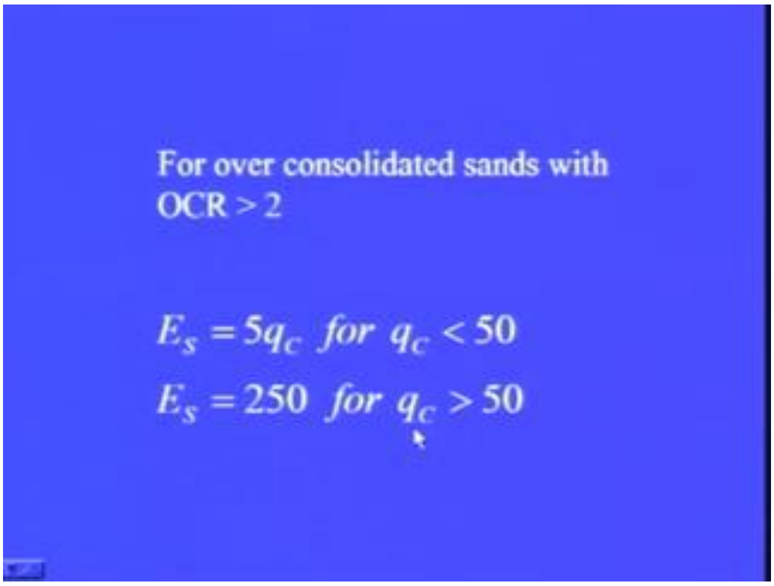

For over consolidated sense with OCR greater than 2 is taken as 5 qc for qc less than 50 and 250 for qc greater than 50. Now, this will be more clear by a solved problem.

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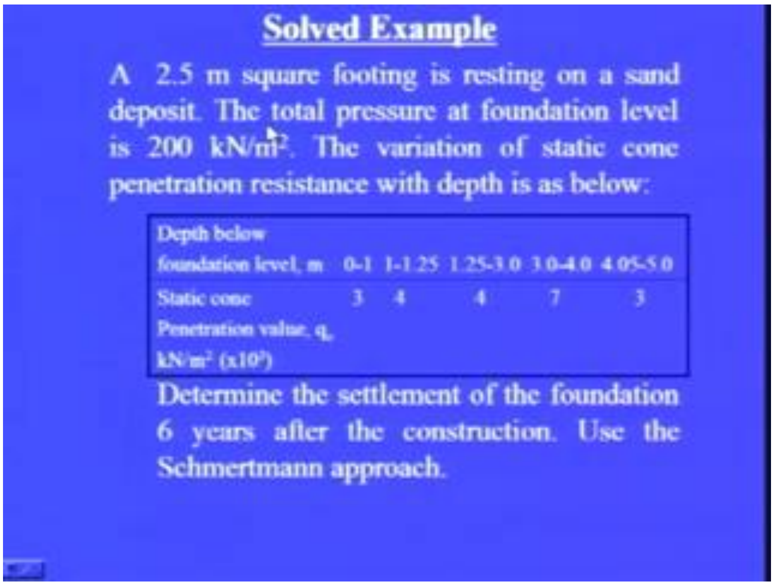

At 2.5 meter square footing is resting on a sand deposit, the total pressure at the foundation level is 200 kilo Newton per meter square, the variation of a static cone penetration resistance with depth is given below. So, this is the depth below the foundation level and this is the static cone penetration test value qc in kilo Newton per meter square into ten to the power three. Now, from 0 to 1 meter depth it is 3, 1 to 1.25 it is 4, 1.25 to 3 it is 4, 3 to 4 meter it is 7 and 4.05 to 5 it is 3.

We will have to determine the settlement of the foundation 6 years after the construction use Schmertmann approach

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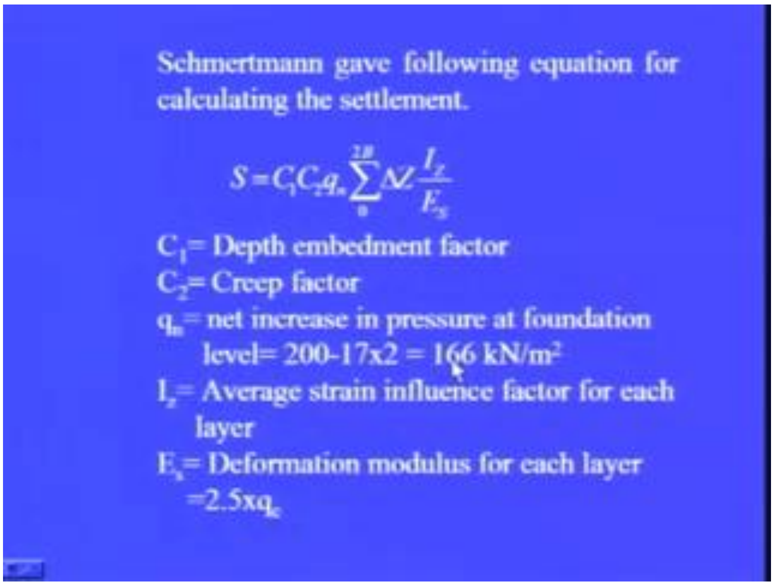

So, we divide this into number of layers and we use this Schmertmann equation to find out the settlement of the foundation, now in this case, even as said earlier C 1 is the depth embedment factor and C 2 is the creep factor, that we will determine q n is the net increase in pressure at the foundation level and that is equal to 200 minus 17 into 2, where 17 is the unit weight of soil and 2 is the depth of the foundation.

So, it comes out to be 160 kilo Newton per meter square, Iz is the average strip strain influence factor for each layer and Es is taken as 2.5 times of qc.

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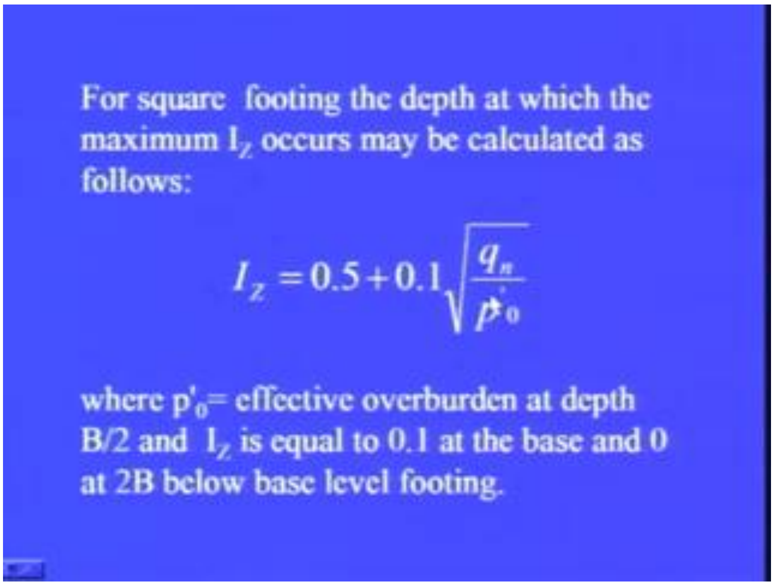

For a square footing the depth at which the maximum Iz occurs may be calculated from this particular relationship or can be read from the figure presented earlier where p 0 is the effective overburden pressure at B by 2 and Iz equal to 0.1 at the base and 0 at 2 B below the base level of the footing.

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$$C_1 = 1 - 0.5\frac{q_0}{q_n} = 1 - 0.5\frac{17 \times 2}{166} = 0.898$$

$$C_2 = 1 + 0.2\log_{10}\frac{t}{0.1} = 1 + 0.2\log_{10}\frac{6}{0.1} = 1.356$$

$$S = 0.898 \times 1.356 \times 166 \sum_0^{2B} \Delta Z \frac{I_z}{E_s}$$

$$S = 202.14 \sum_0^{2B} \Delta Z \frac{I_z}{E_s} = 202.14 \times 0.1393$$

$$= 26.7\ mm$$

The value of $\Sigma(I_z/E)\,\Delta z$ is determined as shown in the Table, next.

Now, C 1 can be calculated if we know q 0 dash and qn, so qn is already known, we know q 0 dash also at the depth of the foundation, so it comes out to be 0.898. Similarly, if the number of years are known, then we can calculate C 2, that is the key factor we substituted here, so it comes out to be 1.356. Now, when we substitute these values in this Schmertmann equation we can obtain, the value of the settlement, so settlement comes out to be equal to 202.14 to 2 B delta Z Iz Es.

So, this particular calculation is shown in the tabular form in the next slide and that comes out to be 0.193 and so the total settlement comes out to be 26.7 millimetre.

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Layer	Δz (mm)	q_c (kN/m²)	E_s (kN/m²)	I_z	$(I_z/E_s)\,\Delta z$
1	1000	3000	7500	½ (0.1+0.42) =0.260	0.0347
2	250	4000	10000	0.380	0.0095
3	1750	4000	10000	0.385	0.0674
4	1000	7000	17500	0.200	0.0114
5	1000	3000	7500	0.067	0.0089
Sum					0.1319

The calculation is shown here, what we have done we have divided this total depth from 0 to 2 B in 5 layers, and each layer having different thicknesses. Here this is 1000 millimetre, 250 millimetres, 1750 millimetre 1000 millimetre and again 1000 millimetre. And for each layer we know the values of qc which is given or obtained form which is given in kilo Newton per metre square. And from this qc we can obtain using relationship 2.5 qc the value of qs and here 2.5 times 3000 it is equal to 7500.

And we also obtain Iz vertical string influence factor, either by the correlation or we can use that graph to obtain the value of Iz. And once when all these are known we can determine Iz upon Ez into delta z. Let us say for this particular case, it comes out to be 0.0347 in such a manner, we determine this for all the layers and then when we sum up we will get this summations as 0.1319.

And we substitute this in the Schmertmann equation to determine the settlement of the foundation, for the condition shown in previous example, determine the settlement of foundation using the De beer and Martens approach.....

(Refer Slide Time: 46:16)

Layer	Δz (mm)	q_c (kN/m²)	E_s (kN/m²)	I_z	$(I_z/E_s)\Delta z$
1	1000	3000	7500	½ (0.1+0.42) =0.260	0.0347
2	250	4000	10000	0.380	0.0095
3	1750	4000	10000	0.385	0.0674
4	1000	7000	17500	0.200	0.0114
5	1000	3000	7500	0.067	0.0089
Sum					0.1319

So, De beer and Martens has given this particular equation for determining the settlement of the foundation.

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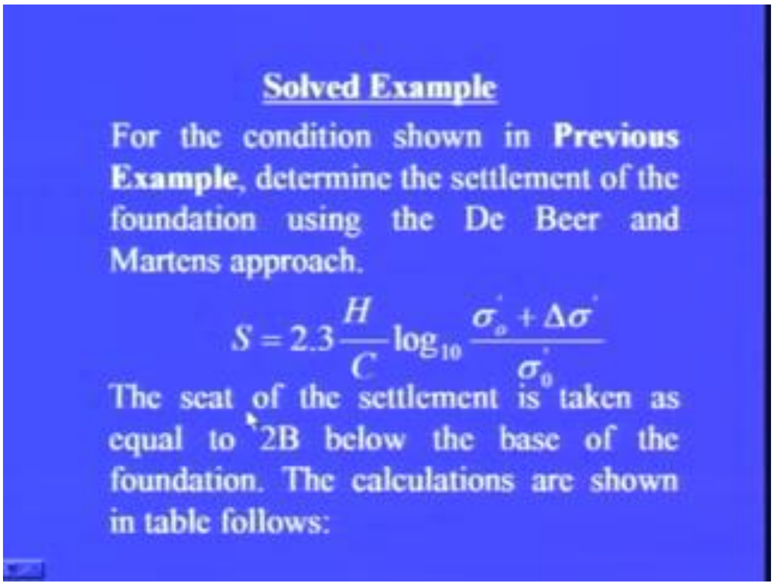

The seat of settlement is taken equal to 2 B below the base of foundation, the calculation are shown again in the tabular form in the next slide...

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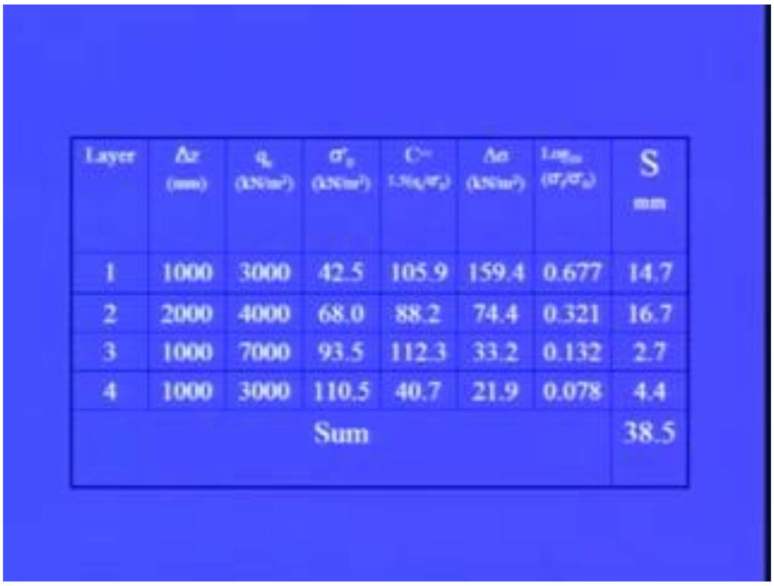

Layer	Δz (mm)	q_c (kN/m^2)	σ_o' (kN/m^2)	$C = 1.5 q_c/\sigma_o'$	$\Delta\sigma$ (kN/m^2)	$\log_{10}(\sigma'/\sigma_o')$	S mm
1	1000	3000	42.5	105.9	159.4	0.677	14.7
2	2000	4000	68.0	88.2	74.4	0.321	16.7
3	1000	7000	93.5	112.3	33.2	0.132	2.7
4	1000	3000	110.5	40.7	21.9	0.078	4.4
			Sum				38.5

Now, here the total layer is divided into four layers depending upon the value of qc, so these are the thicknesses of the clay layer 1000 mm, 2000 mm, 1000 mm again 1000 mm for all the layers. And the average value of qc is given for all these layers like, for first layer it is 3000 qc, then sigma 0 dash at the middle of the layer can be determined if we know the depth of the middle of the layer and the unit weight, it comes out to be 42.5. Then C value can be calculated as 1.5 qc upon sigma 0 dash, so this value is calculated.

Then, delta sigma, that the increase in stress at the middle of the layer is determined by 2 is to 1 method or we can use Bussyneck or the Westergaard formulations and we find out delta sigma. Then we calculate log to the base 10 sigma fr sigma 0 r with sigma 0 bar, a is the overburden pressure initial overburden pressure and sigma fr is the summation of these two. And finally, using De beer and Martens equation, we find out the settlement of the first layer, then second layer, third layer and fourth layer. And when we sum up we get the settlement of the total compressible layer between 0 to 2 B.

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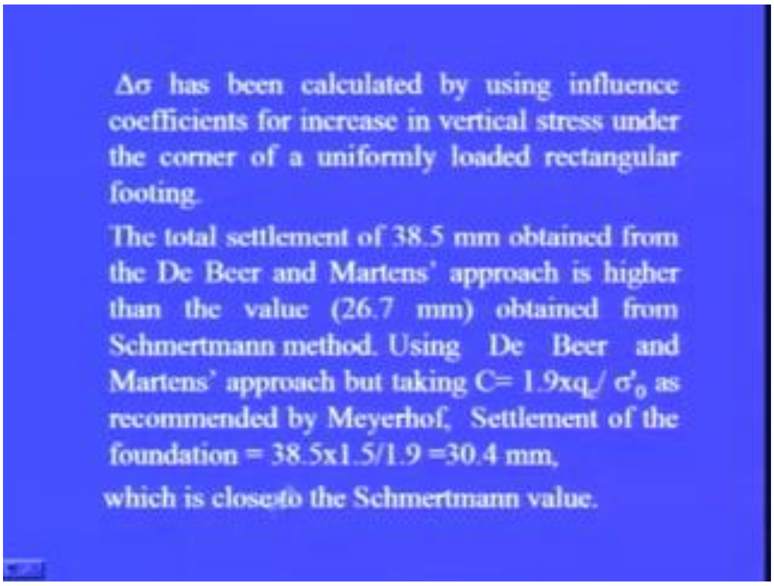

Delta sigma has been calculated by using influence coefficient for increase in vertical stress under the corner of the uniformly loaded rectangular footing. The total settlement of 38.5 millimetre obtain from De Beer and Martens approach is higher than the value of 26.7 millimetre. Which was obtained from Schmertmann method using De Beer and Martens approach, but taking C equal to 1.9, qc upon sigma 0 dash as recommended by Meyerhof settlement of the foundation will be equal to 30.4 millimetre and which is very close to the Schmertmann value.

What will be the settlement in the previous example if the effect of rigidity of footing and the depth of embedment is taken into the account, so the last two examples we have not taken into account the effect of embedment as well as the rigidity of the footing.

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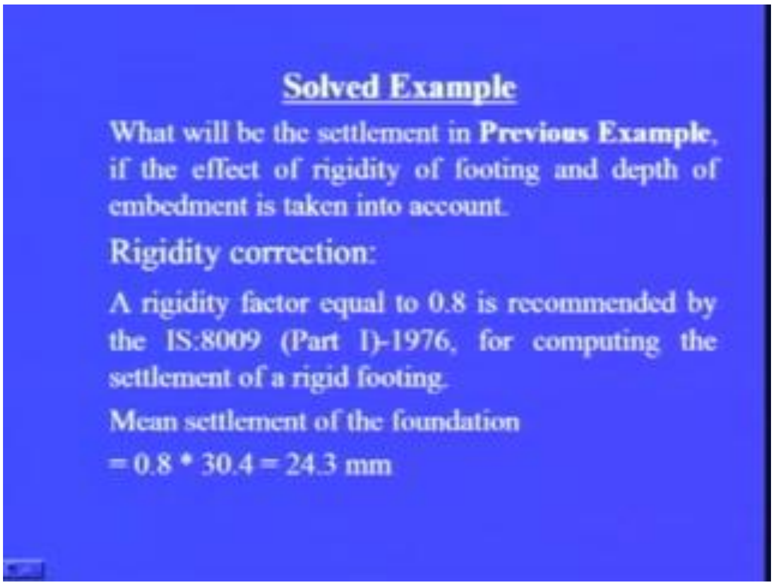

So, rigidity correction factor suggested by is IS 8009 part I 1976, it is 0.8, so mean value of settlement of the foundation will be equal to 0.8 into 30.4 that is the settlement we obtain from the from the De Beer and Martens approach using 1 q Es equal to 1.9 into qc upon sigma 0 dash. That, is comes out to be equal to 24.3 millimetre.

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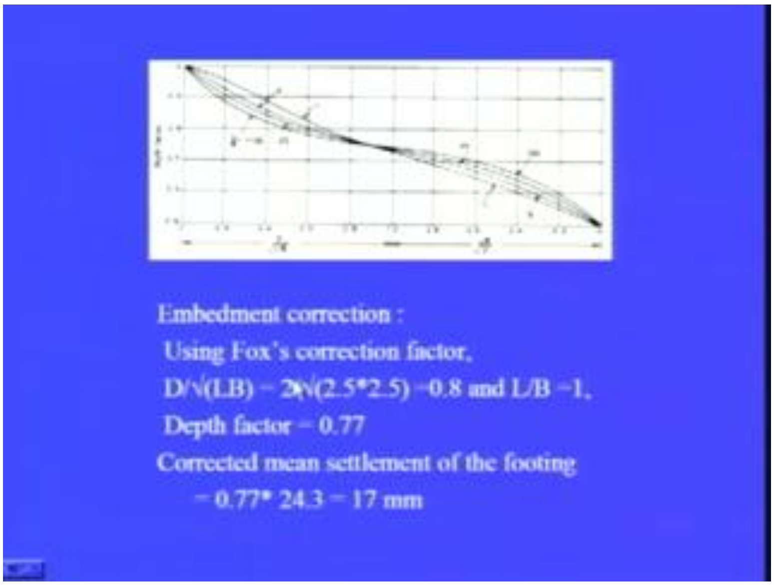

In order to apply the embedment correction factor, again we use the curve given in IS 8009 as suggested by fox and we find out first D upon root LB, so for this particular case it is 0.8 and L upon B equal to 1.The corresponding to this D root upon LB or 0.8 l upon B, using the appropriate curve, we find out the depth correction factor, we read from here hat come out to be point 0.77. And we multiply, the settlement as computed earlier by 0.77 it comes out to be 17 millimetre.

So, in this lecture, we have discussed the methods to determine the settlement of foundations resting on granular soils, especially by using the fielded test data like plate load and then the standard penetration test and static cone penetration test by various approaches suggested by various researches. And we have also discussed the correction factors, which are we use for determination of the actual settlement of the footing, like depth factor and embedment factor, as suggested in the IS 8009.

So, in the next lecture, I will discuss the third and important part of settlement, that is the consolidation settlement and then how to determine allowable varying pressure using various approaches.

Thank you.