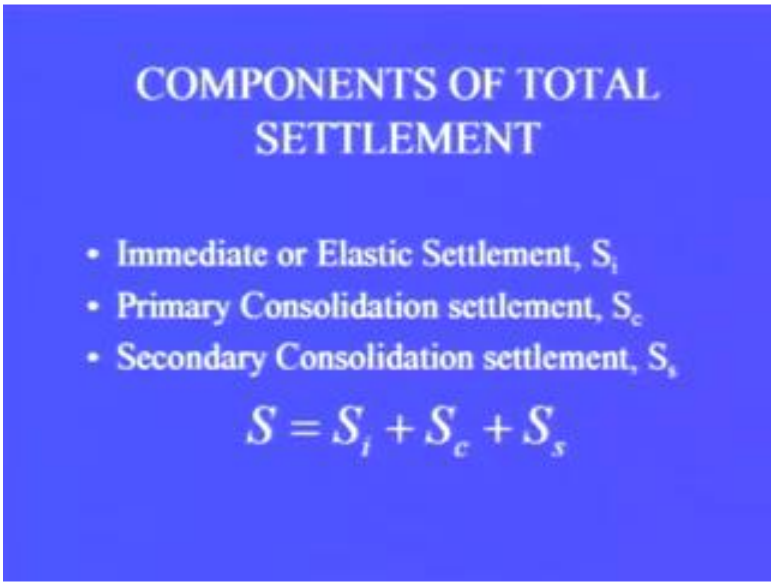
Foundation Engineering
Prof. Dr N.K. Samadhiya
Department of Civil Engineering
Indian Institute of Technology Roorkee

Module – 01
Lecture - 05
Shallow Foundation

Welcome to the lectures on shallow foundation, in the last lecture we were discussing about the settlement of shallow foundations. We discussed the, a detrimental effects due to the settlements of foundations like, there if there is differential settlement. Then there may be cracking there may be angular distortion, and there may be tilting. And we have seen that leaning tower Pisa is the classical example of tilting. Now, we have also discussed about the type of settlement and in that, we discuss that the total settlement comprises of 3 settlements.

(Refer Slide Time 01:06)

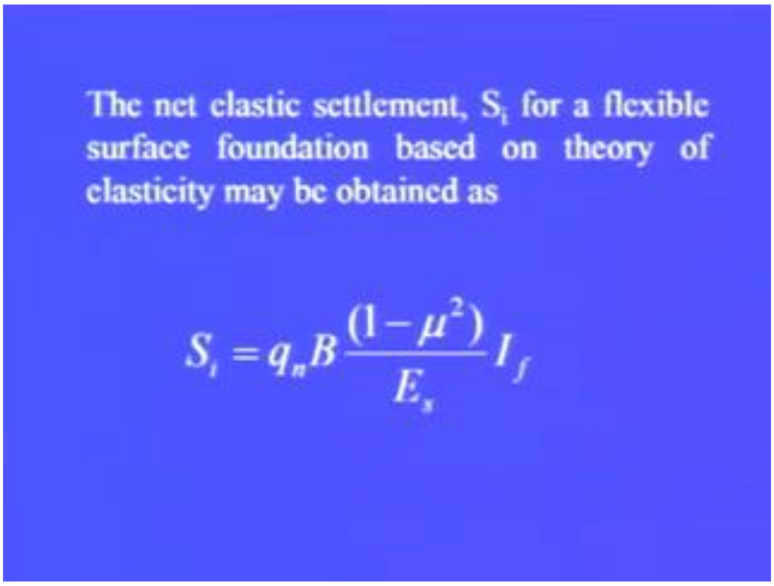


One is known as, immediate or elastic settlement, we designated by Si. Primary consolidation settlement, that is Sc and secondary consolidation settlement that is Ss. So, this total settlement of the structure can be written as Si plus Sc plus Ss.

(Refer Slide Time 01:28)

Now, in order to compute elastic settlement, which is of the immediate nature and due to the elastic compression of the soil mass we have methods available based on theory of elasticity and the method that is suggested by Janbu et al.

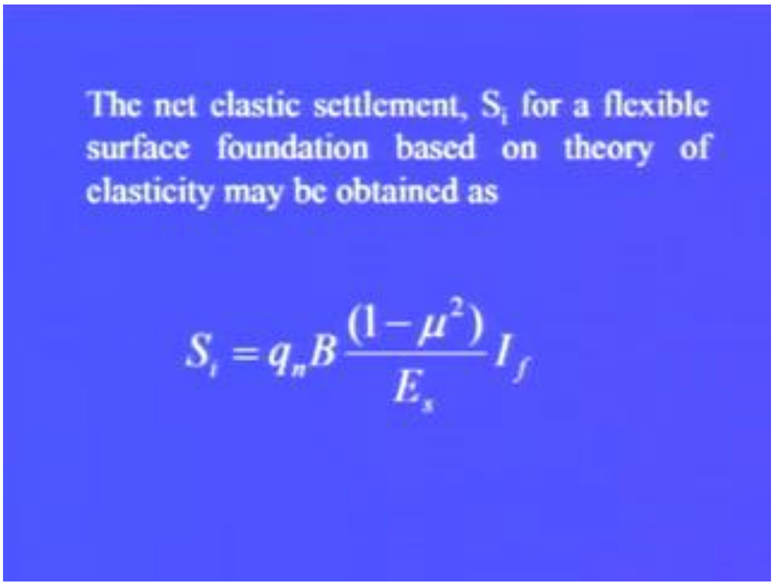
(Refer Slide Time 01:50)



Now, the net elastic settlement Si for a flexible surface foundation, based on theory of elasticity can be determined by this equation Si equal to qn B 1 minus mu square upon Es into influence factor If. Now, here this qn is the net load intensity at the level of the foundation. B is the width of the foundation, that is flexible foundation then mu is the

Poisson ratio of the soil. And for different type of soils in different conditions, different values have been suggested or otherwise we can determine this value by triaxial test in the undrained condition.

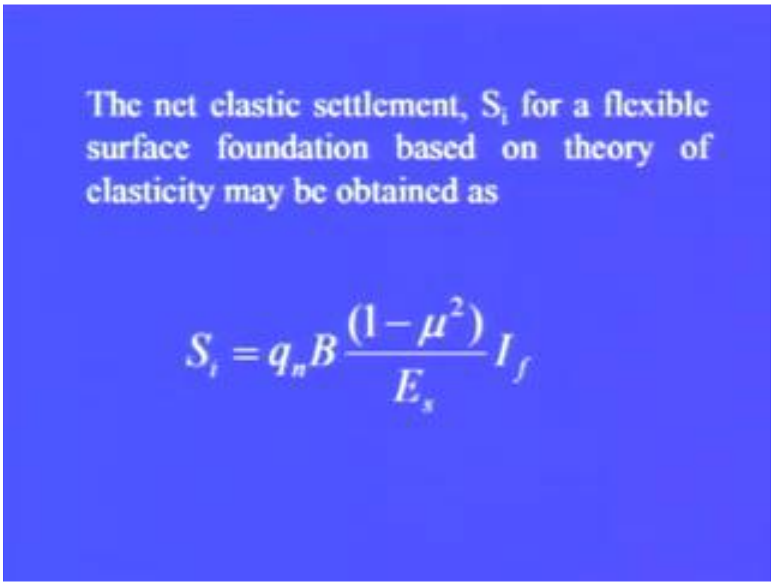
(Refer Slide Time 02:40)



Then we have Es as the modulus of elasticity of the soil, and again for this we can go for the laboratory method. Or the methods which are based on the field test like Standard penetration test or the plate load test or the pressure meter test or the static cone penetration test. Various researchers have given, values for Es and those tables can be used which we discussed in the last lecture for estimating the value of Es. Then the influence factor, we know that due to the pressure distribution in the case of rigid foundation resting on clay. Or the flexible foundation resting on clay or on the sand the pressure distribution varies.

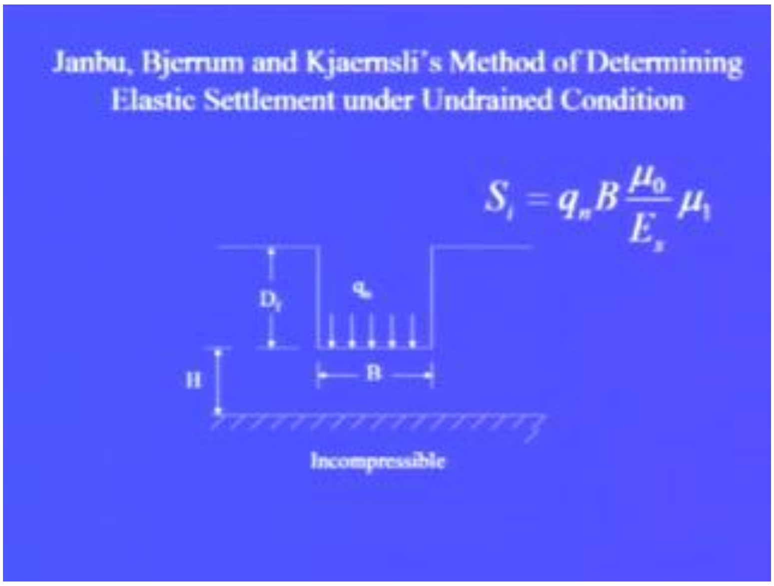
And hence the values of the settlement at the center of the footing, at the corner of the footing are different for the flexible foundation and for the rigid foundation and also for different shapes of the foundations. So, these values have been suggested by various researchers and especially by Vawills and those tables we have discuss in the last class. And we can use those tables for obtaining the value of I f and when we substitute all these we get settlement known as the immediate or elastic settlement.

(Refer Slide Time 04:03)



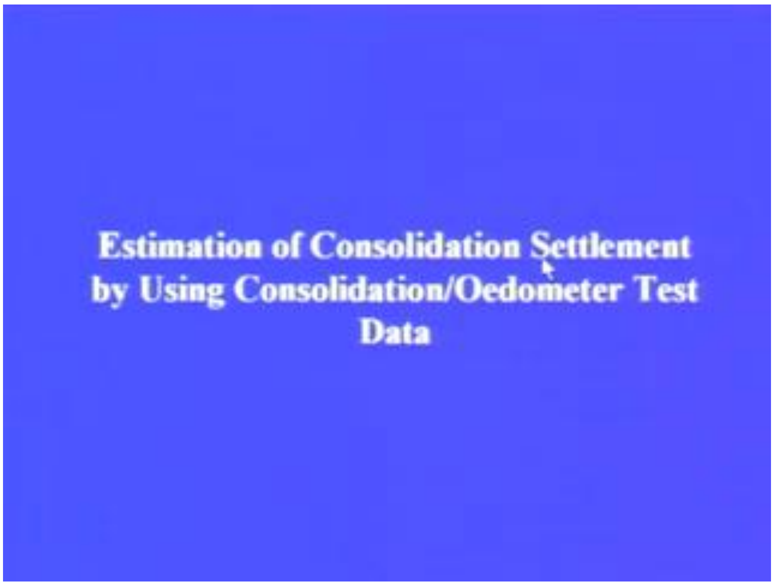
As we discussed that the in the case of sands, mostly the settlement is of the immediate nature. Because the permeability of the sand is far far greater than the permeability of clay and hence whatever settlement takes place, that takes place, immediately or during the loading itself. So, in that case this method is very useful and we can determine this immediate elastic settlement.

(Refer Slide Time 04:45)



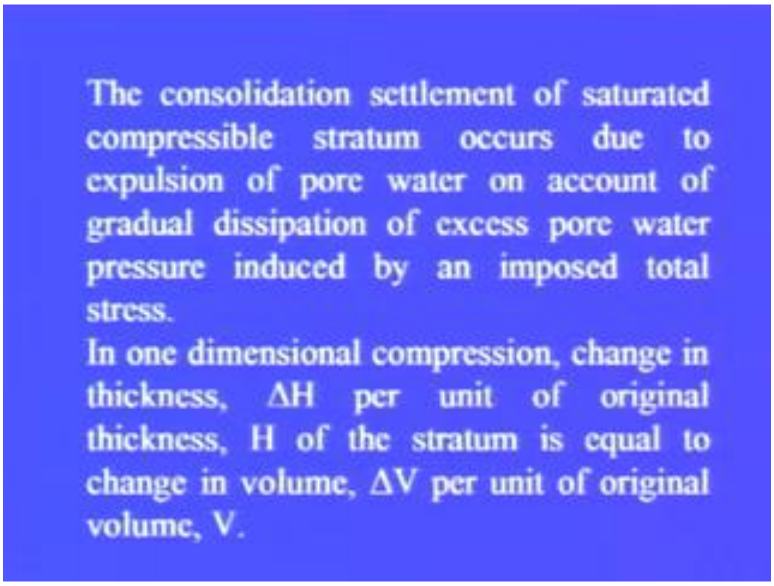
This immediate elastic settlement can also, be determined by method given by Janbu et al. Let us consider a foundation as shown in this figure; it is placed at a depth of Df below the ground surface. And the foundation is loading this soil by a load intensity of qn, on the area are the strip of footing that is of the dimension B. And there is a compressible layer, that is of depth z and below that there is an incompressible layer. Then the elastic settlement can be given as qn into B mu 0 mu 1 upon Es whereas, mu 0 and mu 1 mu 1 are the parameters, which can be obtained by the charts provided by Janbu et al. Now, this mu 0 is a function of Df by B whereas, mu 1 is the function of H upon B for different shapes of footing like square footing, strip footing, circular footing and rectangular footing. So, using that chart, we can obtain value mu 1 and then when we substitute in this equation we get elastic settlement.

(Refer Slide Time 06:10)



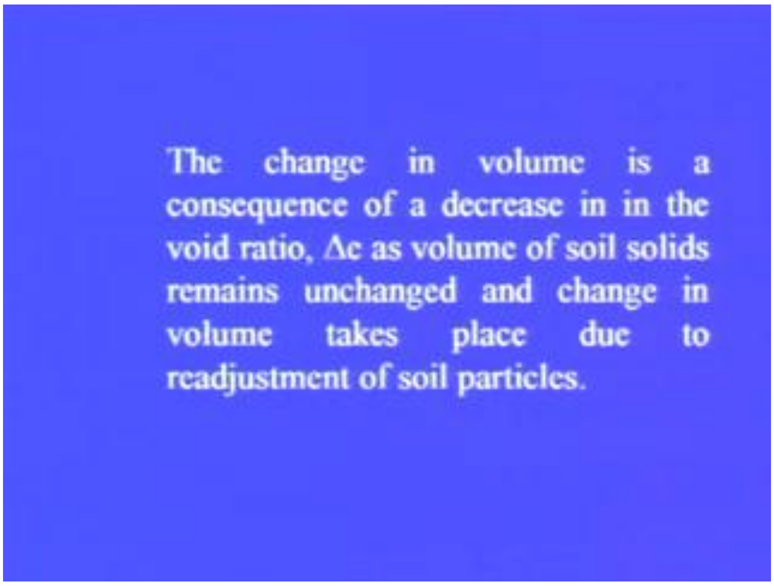
Now, in order to determined consolidation settlement, we use the data obtain by the consolidation test or Oedometer test.

(Refer Slide Time 06:24)



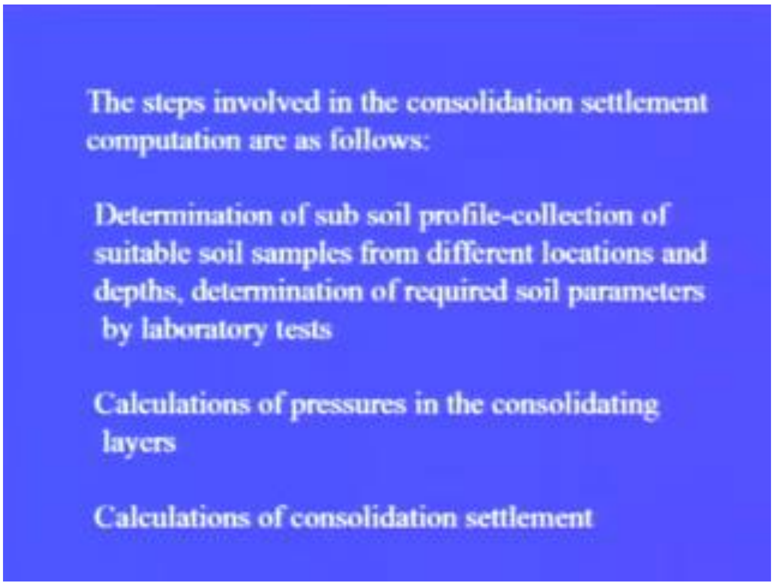
Now, as you must have learn in the soil mechanics course, that the consolidation settlement of saturated compressible stratum, occurs due to expulsion of pore water on account of, gradual dissipation of excess pore water pressure, induced by an imposed total stress. In one dimensional compression change in thickness delta H per unit of original volume H of the stratum is equal to the change in volume, delta V per unit of original volume. So, using this data what we can do? We can obtain the parameters and then we can calculate consolidation settlement.

(Refer Slide Time 07:11)



The change in volume is a consequence of, decrease in the volume of voids that is void ratio delta e. As volume of soil solids remain unchanged and the change in volume takes place due to readjustment of the particles. As water escapes out from the voids, these soil solids are readjusted and we get a compression.

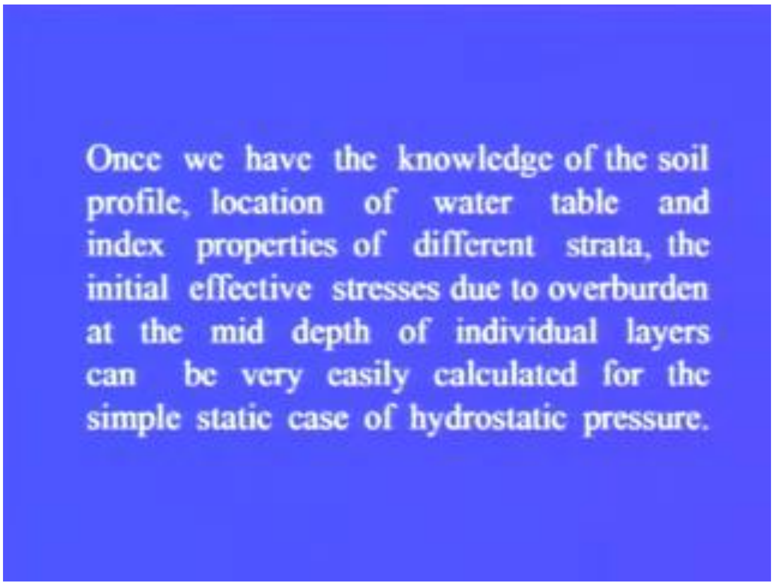
(Refer Slide Time 07:41)



Now, in order to determine the consolidation settlement the steps, involved in the consolidation settlement are as follows. Determination of sub soil profile collection of suitable soil samples from different locations and depths, and determination of required soil parameters by laboratory tests. So, what we do? If we want to determine consolidation settlement of a compressible layer we will have to first get the profile sub soil profile of the area.

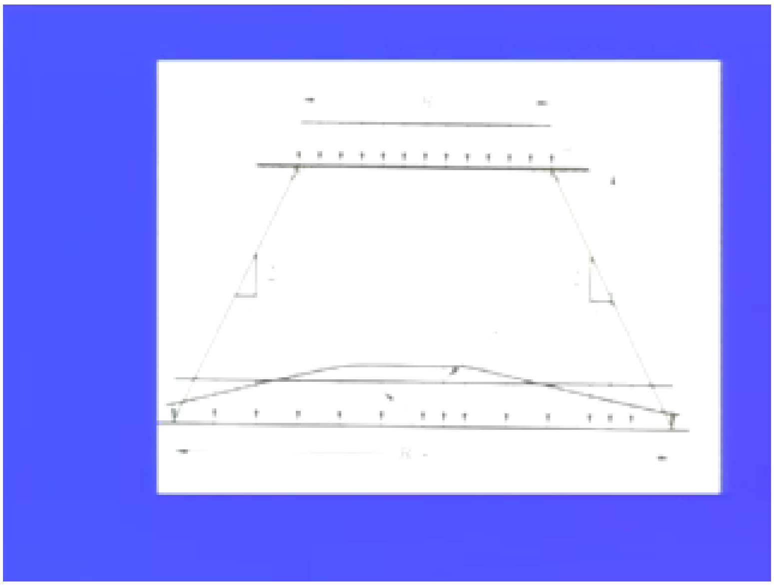
Now, what for that what we do? We go for boreholes we advance boreholes up to the required depth, and then we collect undistorted and distorted samples. And on undistorted samples, we conduct consolidation test and obtained the consolidation parameters, which are for the calculation of settlement. Then one once that strata is known and if we know the load which is applied by the footing, on to the sub soil we can also determine the pressures in the consolidating layers. And then finally, we calculate the consolidation settlement.

(Refer Slide Time 08:59)



Once we have the knowledge of soil profile, location of water table and index properties of different strata. The initial effective stresses due to overburden at the mid depth of individual layers, can be very easily calculated for the simple static case of hydrostatic pressure.

(Refer Slide Time 09:15)

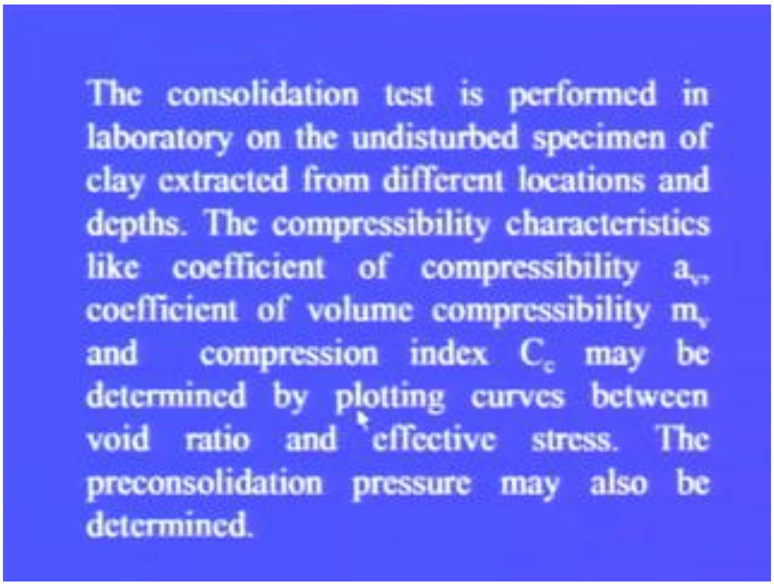


In the last class, we also discuss the methods to determine the stresses, at different levels, due to the foundation loading. Now, we have methods given by Bussyneck, we have methods given by a Westergaard. So, by using Bussyneck method or by using

Westergaard, which is normally preferred in the sedimentary soil deposits. We can determine, what is the increase in stress at the particular level? Now, this particular normally, we consider at the mid depth of the consolidating layer. Now, there is an method approximate method, given that is known as 2 vertical is to 1 horizontal distribution of pressure, the which we discussed in the last lecture. Now, here there is a loaded area of width B, and load intensity Q is applied to the soil, then if you want to determine the stress increase. Due to this stress at a level z depth below the ground surface, then what we will have, to we will have to consider this to vertical is to 1 horizontal load distribution. And then the loaded area at that particular depth will, become equal to the width of B plus Z and hence what we can do?

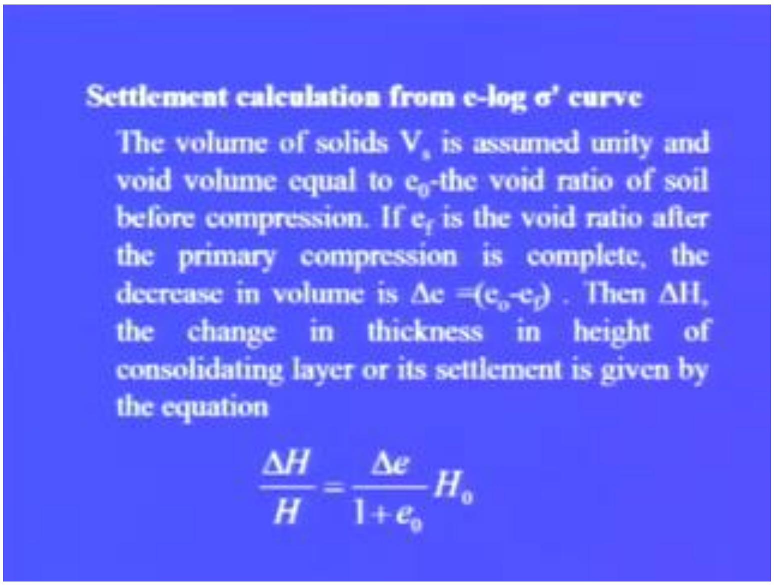
We can determine this in is increase in stress, as B into Q into the dimension in the other direction. In the case of strip footing, it is 1 in the case of rectangular footing it is length L and in the case of square footing this is width B. So, from that, we can distribute this load total load B into Q into whatever is the dimension in this direction, divided by B plus z into 1 or the dimension in the other direction depending upon the shape of footing. And hence, we calculate this value of the mean stress, increase in stress, due to the foundation loading. Now, this is the graph or the curve, which we obtain either by the Bussyneck method or by the Westergaard method whereas, this uniform stress intensity we get by 2 is to 1 method.

(Refer Slide Time 11:53)



The consolidation test is performed, in laboratory on the undisturbed specimen of clay from different locations and depths. The compressibility characteristics like coefficient of compressibility av coefficient of volume compressibility mv and compression index Cc may be obtained by plotting curves, between void ratio and effective stress. The preconsolidation pressure may also be determined from this data.

(Refer Slide Time 12:23)



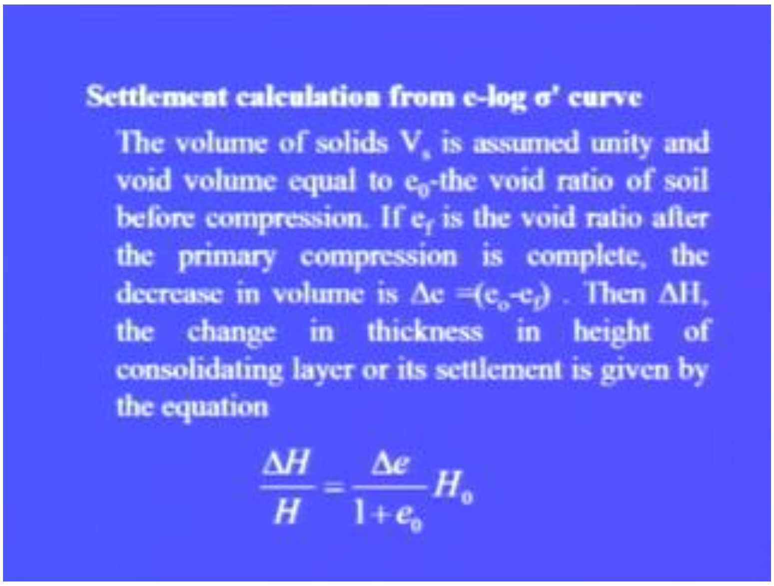
Now, as for as the consolidation, has it concern you must have been studied it in the soil mechanics course, but for the benefit, we I simply give you salient features of the consolidation test. What we actually do? We take out and undistorted sample from the field, and then that undistorted sample from the field is transferred to the consolidating ring consolidation test in the consolidation test operators. We have a consolidation ring of the standard size normally it is of the diameter 75 millimeter and thickness 20 millimeter. After transferring that, in the consolidation ring we keep this consolidation ring in the consolidation setup and then we saturate the soil sample. We know that the consolidation theory given by Terzaghi is valid only for the saturated clays.

So, we saturate the soil sample and then after saturation is achieved, we apply a load increment and in that load increment, we get the compression of that consolidating layer at different time intervals. Like starting from time t equal to 0, to time t equal to 0.25 minutes, then time t equal to 0 point 36 minutes. And likewise up to 24 hours week keep on measuring the compression. And after that, we after that compression is achieve we increase the load intensity to another loading increment, like. We go as 50 kilo Newton per square, then 100 kilo Newton per meter square 200 kilo Newton per meter square 400 kilo Newton per meter square 800 kilo Newton per square.

And if required 1600 kilo Newton per meter square, it has been observed from the test that this consolidation test is valid only for the large increment ratio and that increment ratio should be at least 1. That is why we preferred 0 25 50 100 200, so, that the load increment ratio is at least 1. Now, at all stress increment we find out the compression of the soil, of that 20 millimeter thick soil layer which is which we have already placed in the consolidation ring. And from this data, we can obtain coefficient of consolidation or coefficient of compressibility compression index or we develop our relationship, between e versus effective stress.

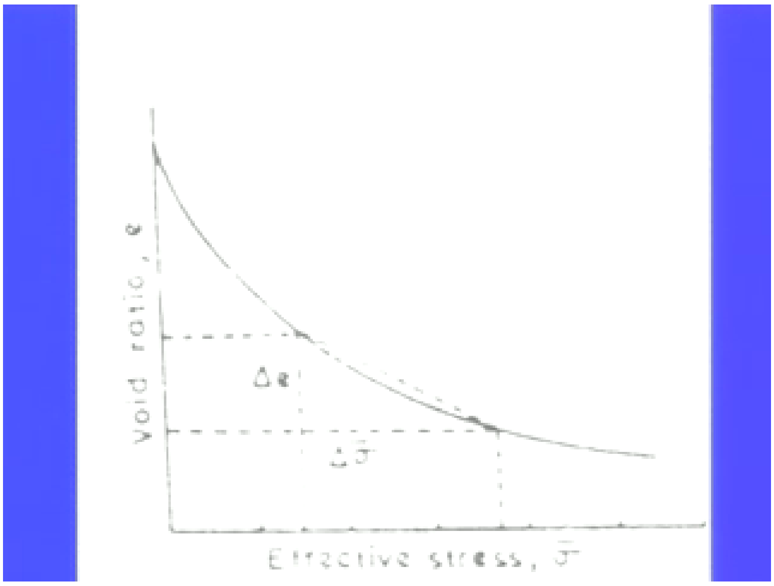
Now, in order to determine, value of e there are 2 methods available. 1 is known as height of solids method, another is known as a change in voids ratio method. So, from that we develop our relationship between e versus sigma bar, similarly, in order to get coefficient of consolidation. We have either Taylors method square root of a time fitting or a secondary method by which square root of log of the then using those methods we can determine coefficient of consolidation. But here in the calculation of ultimate settlement or total settlement we are interested in av, that is coefficient of compressibility. My; that is coefficient of volume change, and the compression index and using that, we can determine the settlement. So, we develop our relationship between, wide ratio and effective stress. Once that relationship is known using that relationship we can find out the consolidation settlement, of the actual soil layer using those parameters.

(Refer Slide Time 16:43)



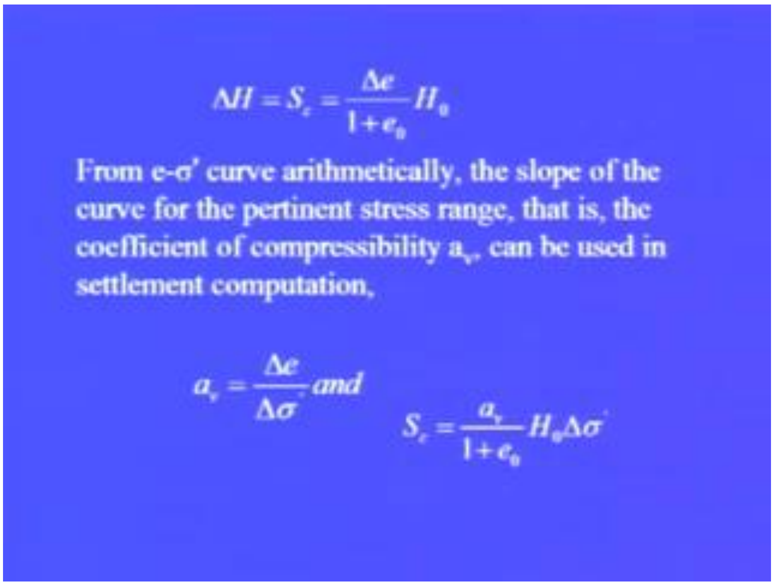
So, the volume of solids Vs is assume unity and volume void volume, equal to e 0 the void ratio of soil before compression. If ef is the void ratio after the primary compression is complete the decrease in volume is delta e that is e 0 upon ef e 0 minus ef. Then delta H the thickness change in thickness in the height of consolidating layer or its settlement is given by delta H upon H equal to delta e upon 1 plus e 0 into H 0. Now, here this e 0 is corresponding to the stress sigma 0 a dash that is the effective overburden pressure at the middle of the consolidating layer. And e final is the effective stress, after the application of the load or when we include je sigma 0 plus delta sigma and that delta sigma is determined by the 2 is to 1 method

(Refer Slide Time 17:42)



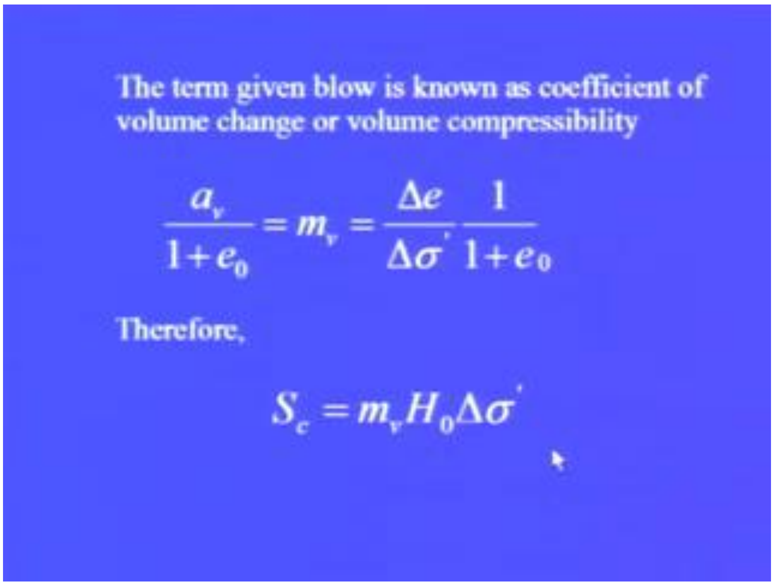
This is that void ratio versus effective stress relationship, which we get from the consolidation test. So, from this at different stresses we find out what is the void ratio? Either using height of solids method or change in void ratio method, and then we plot our relationship. Now, as can be seen from here that this is the void ratio corresponding to the initial condition and this is the void ratio corresponding to the final condition. So, and this delta sigma bar is the increase in stress from sigma 0 to bar to this sigma f bar that is the effective stress after the load has applied. Now, this slope of this curve is nothing, but the coefficient of compressibility av. Now, for the stress range, which we which are encountered in field, what we can do? We can consider this segment. The change in void from this to this that is equal to delta e divided by delta sigma bar or the delta sigma dash effective stress, we call it as the coefficient of compressibility.

(Refer Slide Time 19:00)



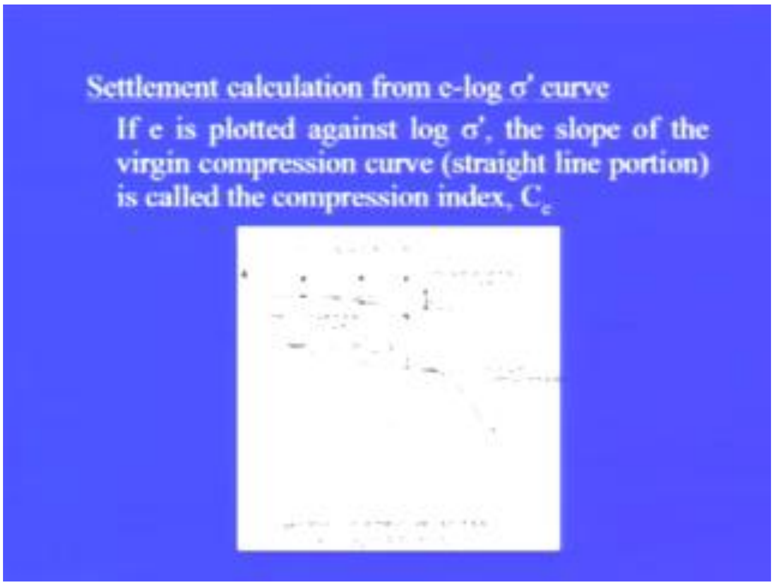
And from that coefficient of compressibility, we can determine the settlement. So, this delta H equal to nothing, but the consolidation settlement of the layer, that will be given by delta e upon 1 plus e 0 into H 0. Where H 0 is the thickness of the compressible layer e 0 is the initial void ratio. And this delta e can be determined if we read values of a void ratio at sigma 0 dash effective stress and sigma 0 plus delta sigma dash. And whatever is the difference we substituted here, and then we calculate consolidation settlement. Now, from this e sigma dash curve arithmetically the slope of the curve for the pertinent stress range, that is the coefficient of compressibility av can be used in the settlement computation. So, this av is delta e divided by delta sigma dash. So, this delta e can e written as av into delta sigma dash. Now, when we substitute in this equation, we will get sc equal to av upon 1 plus e 0 into H 0 delta sigma dash.

(Refer Slide Time 20:10)



The term given below is known, as coefficient of volume change or volume compressibility that is coefficient of compressibility divided by the initial volume, 1 plus e 0. So, this av upon 1 plus e 0 is mv and that is equal to delta e upon delta sigma dash 1 upon 1 plus e 0. Now, from this we can get this delta e as mv into delta sigma dash 1 plus e 0 and when we substitute here in the consolidation equation, then this sc equal to mv H 0 delta sigma dash. So, this expression can also be used to determine consolidation settlement.

(Refer Slide Time 20:55)

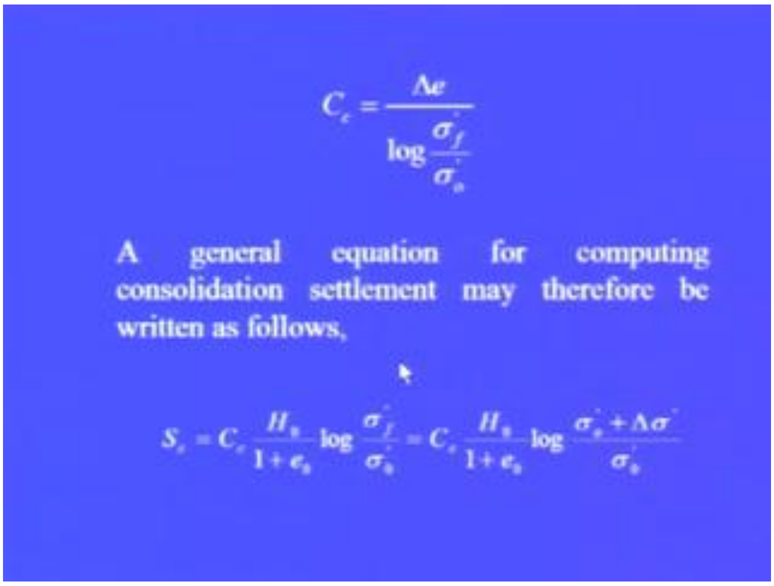


This particular data, of which we obtain from the consolidation test, can also be plotted in different form in the form of e versus log of sigma dash curve. And this 1 such curve is shown here now. From here when we start that comp consolidation test at different sigma dash we obtain value of e and we keep on plotting these values. Now, it is possible that after a particular stress has reached we can unload this and we allow this soil to expand and that expansion curve is shown by this line. It can be seen from here, that it does not follow the path of the compression curve. So, this is the expansion line there will always be permanent displacements or deformations which takes place due to readjustment of the particles. So, on the elastic compression or the elastic rebound, we obtain here when we unloaded.

And after unloading, it we can again reload, by the same intensity of load, and then we plot the e log sigma bar curve. You will find that after this particular pressure, has reached the at the pressure more than this the shape of the curve or the nature of the curve, it merges with the nature as if the soil has not been unloaded. So, here I am trying to explain the difference, between the normally consolidated soil and the over consolidated soil. Now, this is the curve for the normally consolidated soil and normally consolidated soils are those which have never experienced a pressure, which is existing at present. Now, in this particular case, we can see if we have the load the void ratio and sigma bar curve for a particular soil, then this particular soil has already, experienced load corresponding to this particular stress.

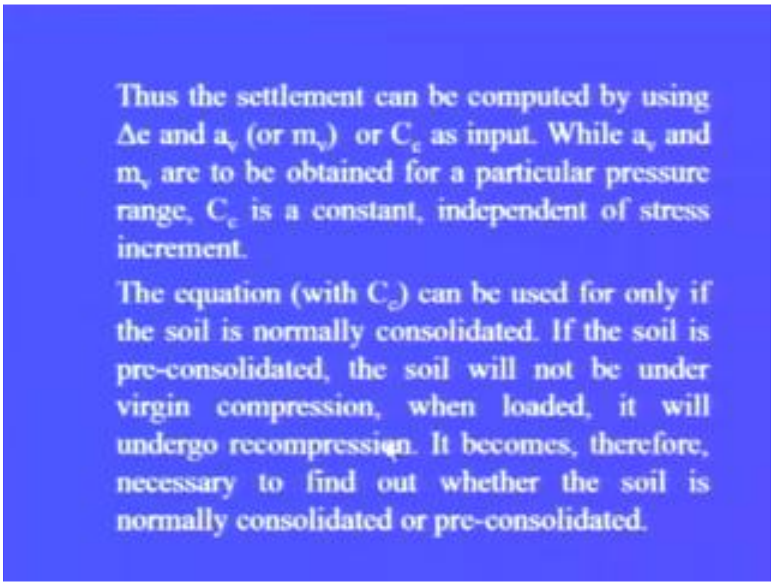
Now, when we unload, it and when we load it reload it again, then we can say that this particular curve from here to here belongs to the over consolidated clays. And after that it merges with the normally consolidated clays. So, over consolidate clays are those clays which have been subjected to the pressure intensity, larger than the present pressure intensity. So, here from these 2 we can obtain the compression index Cc as well as recompression index Cr. Now, this recompression index is the average slope of this curve of the expansion as well as recompression. So, here this compression index is nothing, but the slope of the virgin compression curve, for the case of the normally consolidate clays. Whereas this slope is the slope of the recompression curve now, this you must have studied in the soil mechanics course.

(Refer Slide Time 24:20)



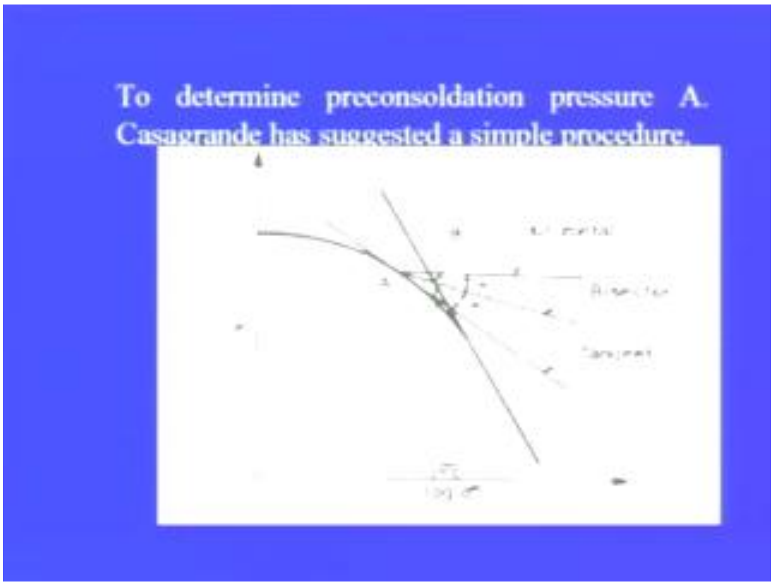
So, from the e log sigma worker of the normally, consolidate clays we can obtain compression index as delta e upon log of sigma f dash divided by sigma 0 dash. Now, from this, we can express this delta e in terms of Cc compression index. And hence, the general equation for computing consolidation settlement may therefore, be written as Sc equal to Cc H 0 upon 1 plus e 0 log of sigma f dash divided by sigma 0 dash.

(Refer Slide Time 25:01)



Thus the settlement can be computed by using delta e or av or mv or Cc as input. While av and mv are to be obtained for a particular pressure range, Cc is a constant, which is independent of the stress increment for normally, consolidated clays only. The equation with Cc can be used for only, if the soil is normally, consolidated if the soil is pre-consolidated the soil will not be under virgin compression. When loaded it will undergo recompression it becomes therefore, necessary to find out whether the soil is normally, consolidated or pre-consolidated.

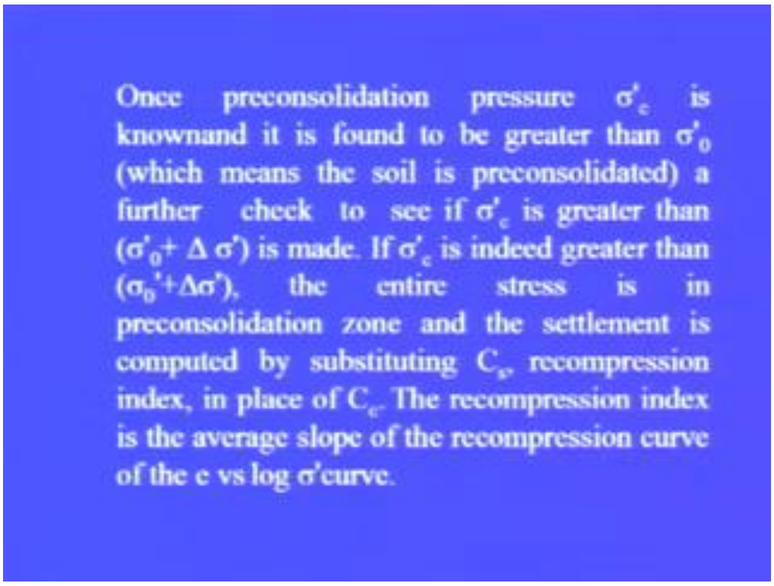
(Refer Slide Time 25:47)



So, for that a method is given by A. Casagrande to determine, whether the soil will behave as normally, consolidated clay or the soil will behave as the pre-consolidated clay. So, from the same e sigma bar log of sigma bar curve, we can obtain the preconsolidation pressure or over consolidation pressure. And then decide whether the soil will behave, for that particular stress range as the normally, consolidated or the pre-consolidated clay. Method is very simple, now, once we plot that e log of sigma bar curve log of sigma bar curve. Now, from in this curve, we find out a point which has the maximum curvature and from that point let us say that point here is 0 point 8. We draw a horizontal line and tangent is drawn to this e log sigma bar curve from this point A. After drawing this tangent, we obtain the angle between the horizontal and tangent. Now, then we draw an angle bisector of this angle passing through point A.

Now, we have this as the angle bisector, then we extend the straight line portion of the normally consolidated soil. And when we extend it back wherever, it intersects the angle bisector, that is the point corresponding to preconsolidation pressure or over consolidation, pressure that is sigma C dash. Now, the soil if the pressure range is on the side from here, to sigma C bar the soil will behave as the pre-consolidate clay. nd for the pressure range more than sigma C bar it will behave as the normally, consolidate clay. So, in this range we will have to use coefficient of recompression whereas, in this range we will have to use coefficient of compression to determine consolidation settlement and that may be, may become more clear by the next explanation.

(Refer Slide Time 28:02)

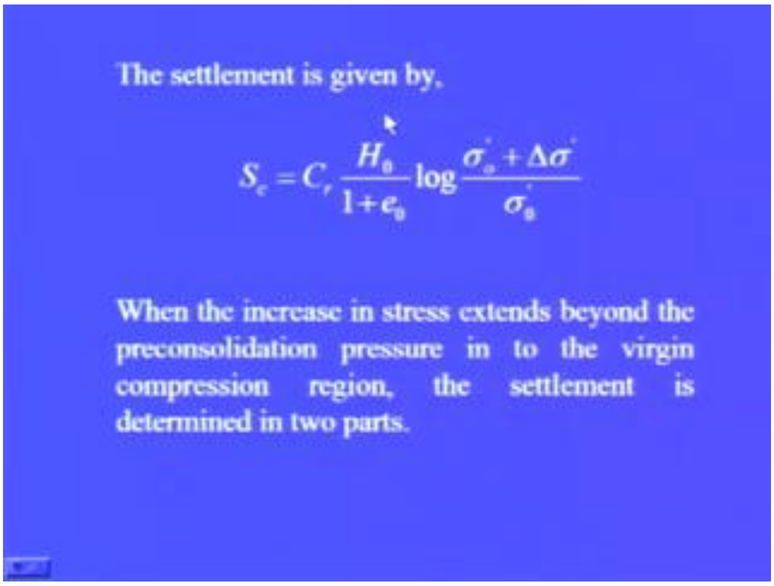


Once preconsolidation pressure, sigma c dash is known and it is found to be greater than sigma 0 dash, which means the soil is pre-consolidated a further check to see, if sigma c dash is greater than sigma 0 dash plus delta sigma dash is made. If sigma c dash is indeed greater than sigma 0 dash plus delta sigma dash also the entire stress range is in the preconsolidation zone. And the settlement is computed by substituting coefficient of recompression Cr are recompression index in place of Cc. The recompression index is the average slope of the recompression curve of e versus log of sigma dash curve.

(Refer Slide Time 28:51)

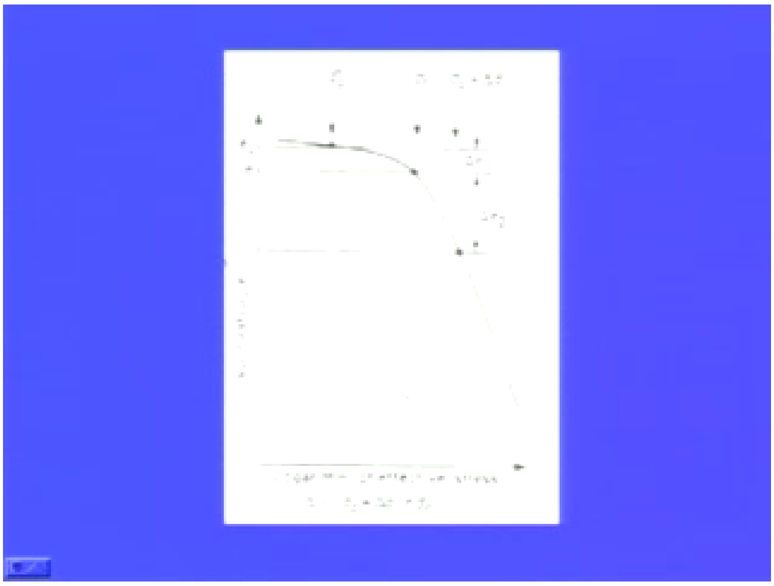
Now, here again the same trans, slide is projected now, this can be made more clear once we get sigma C dash bar reconsolidation pressure. So, if sigma C 0 bar is less than, this it means this is in the preconsolidation range and sigma 0 bar plus delta sigma bar again, that is also less than this. It means, this total stress range is less than sigma C bar and hence the, this is the case of pre-consolidated clay.

(Refer Slide Time 29:34)



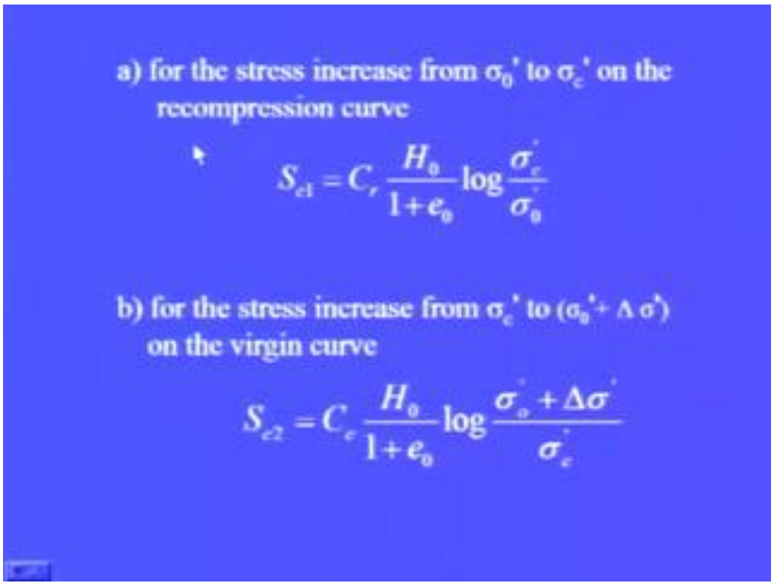
Then the settlement is given by Sc equal to Cr H 0 upon 1 plus e 0 log of sigma 0 bar dash plus delta sigma dash divided by sigma 0 dash. Now, when the increase in stress extends, beyond the preconsolidation pressure, in to the virgin compression region the settlement is determined in 2 parts.

(Refer Slide Time 30:00)



Now, let us consider another case sigma, this is sigma C bar that is the preconsolidation pressure and sigma 0 bar is less than sigma C bar. So, the original overburden pressure are initial overburden pressure is less than preconsolidation pressure, but when we apply a load intensity of delta sigma bar at that middle of the clay, layer. Then if sigma 0 dash plus delta sigma dash, becomes more than sigma C dash then it is it is in the range of the normally, consolidate clay. So, in such cases the settlement is determined in 2 parts either we can find out the change in void ratio from sigma 0 bar to sigma C C bar and substitute in the settlement equation. We will get settlement of this particular layer, we substitute this as delta e 1. And in the next part from sigma C bar to sigma 0 bar plus delta sigma dash we get this delta e 2. And for both these we can calculate settlement and then when we sum up we get the settlement of the consolidating layer. Or otherwise what we can do?

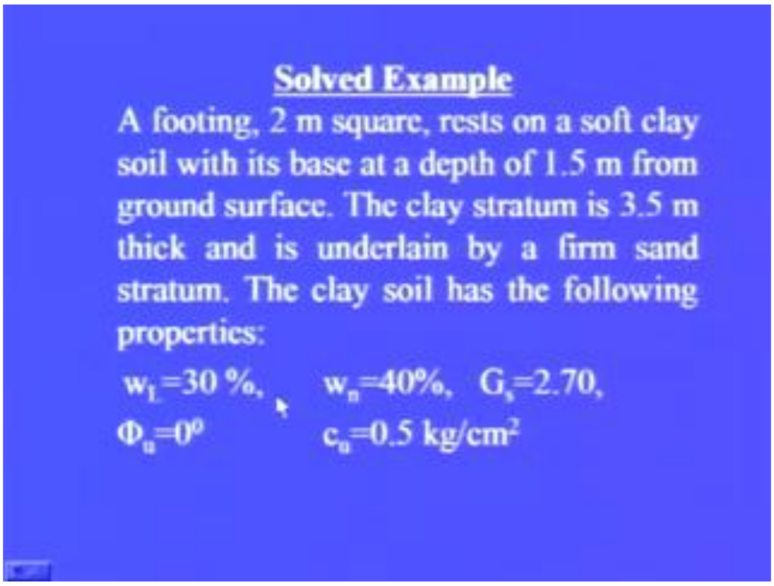
(Refer Slide Time 31:27)



For the stress increase from sigma 0 dash plus sig to sigma c dash on the recompression curve, we find out the recompression index and using this recompression index we calculate settlement Sc 1. For the part which is in the preconsolidation range. And for the stress increase from sigma c bar to sigma 0 dash plus delta sigma bar on the virgin compression curve, we calculate this. Sc 2 compression index H 0 upon 1 plus e 0 log sigma 0 dash plus delta sigma dash.

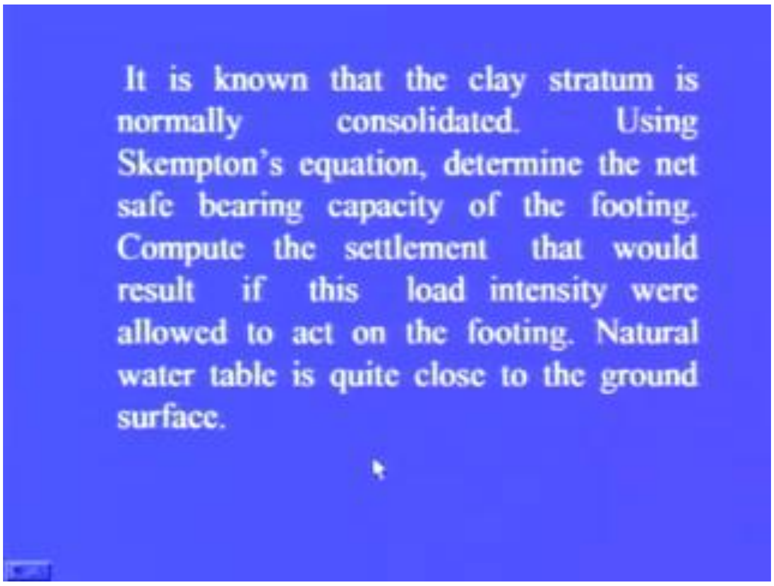
Now, if we can see from here that this is the final stress and this is the initial stress for the preconsolidation range. This sigma c bar is final stress, and for and initial stress is sigma c 0 bar whereas, for the normally, consolidated range sigma 0 dash plus delta sigma dash is the final stress and sigma c bar or sigma c dash is the initial stress. So, we will have to keep this things in min and we can calculate for Sc 1 and Sc 2 and when we sum up we get the consolidation settlement of the compressible layer.

(Refer Slide Time 32:45)



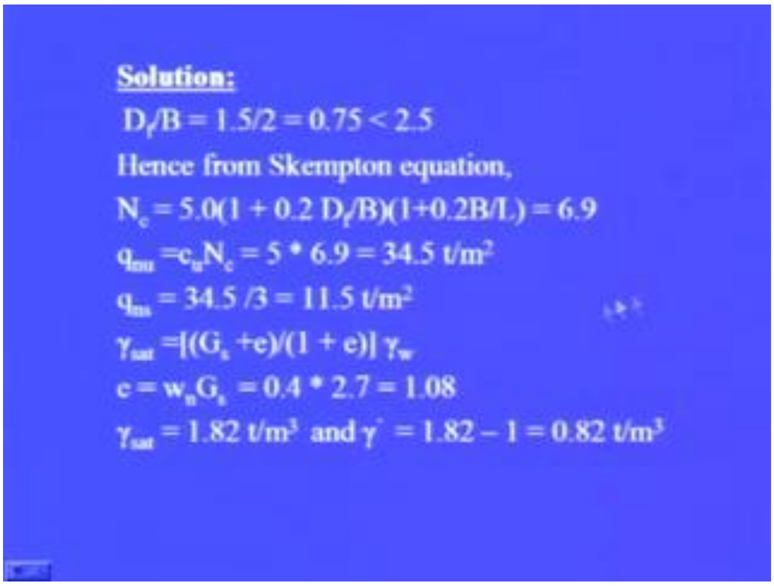
Now, this can be explained by a solved example. A footing 2 meter square rests on a soft clay soil, with its base at a depth of 1.5 meter from ground surface. The clay stratum is 3.5 meter thick and is underlain by a firm sand stratum; the clay soil has the following properties. Liquid limit as 30 percent, natural water content as 40 percent, specific gravity of soil solids is 2.7 Phi u that is 0, because this is the case of clay and cu equal to 0.5 kg per centimeter square.

(Refer Slide Time 33:27)



It is known that the clay stratum is normally consolidated. Using Skempton's equation determine the net safe bearing capacity of the footing, compute the settlement that would result if this load intensity were allow to act on the footing, natural water table is quite close to the ground surface.

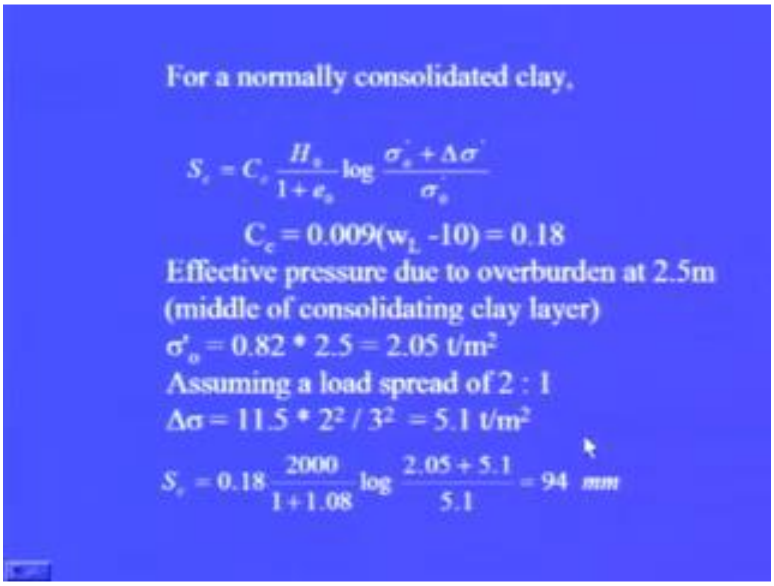
(Refer Slide Time 33:48)



Now, using Skempton's equation, we can find out the net ultimate bearing capacity of the foundation. In order to get that we first of all find out what is Df by B. In this particular case this Df by B is equal to 1.5 divide by 2 that is 0.75 that is less than 2.5. Now, when it is less than 2.5 then Nc is given by this expression, this we have already, discussed in the lecture, Nc equal to 5 plus 1 plus 0.2 Df by B 1 plus 0.2 B upon L. When we substitute these dimensions, we will get this Nc equal to 6.9. And net ultimate bearing capacity is, for the case of clay soils in which phi equal to 0 is cu Nc. So, Nc we obtain from here, cu is given, so, 5 into 6.9 that is 34.9, 34.5 ton per meter square. Now, q net safe can be obtain, if we know net ultimate bearing capacity if we divide it by factor of safety, we get this as 11.5 ton per meter square. Now, in order to determine the overburden stress, we need to know the value of saturated unit weight of the soil. So, for that this saturated unit weight can be determine by this formula Gs plus e upon 1 plus e, whole multiplied by unit weight of water gamma w. Now, this e can be determined, for a saturated soil as water content into the specific gravity of soil solids. So, here water content is 40 percent, 0.4 into 2.7 the void ratio will be 1.08. And hence gamma, if we substitute this void ratio here and unit weight of water. We get gamma saturated,

saturated unit weight of soil, solid soil as 1.82 ton per meter cube. And submerged unit weight, if simply we detect minus 1 that is gamma saturated minus gamma w, that is equal to 0.82 ton meter cube.

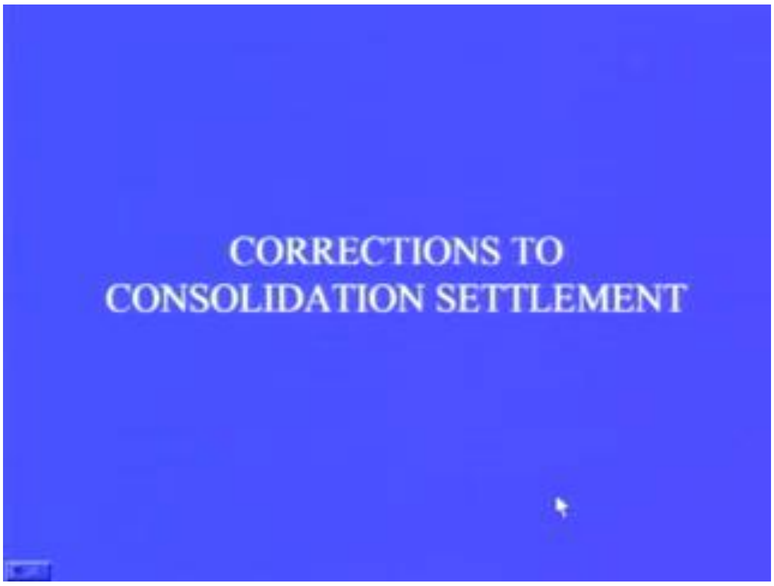
(Refer Slide Time 36:01)



Now, for a normally, consolidated clay Sc will be given by Cc H 0 upon 1 plus e 0 log to the base 10, sigma 0 dash plus delta sigma dash divided by sigma 0 dash. Where this compression index can also be determine by imperial correlation, which says that Cc equal to 0.009 WL minus 10. So, in the absence of the consolidation test data, there are many correlations given by various researchers, by which we can determine compression index also. Now, a popular relationship is Cc equal to 0.009 WL minus 10, when we substitute liquid limit of the soil then we of get this compression index as 0.18.

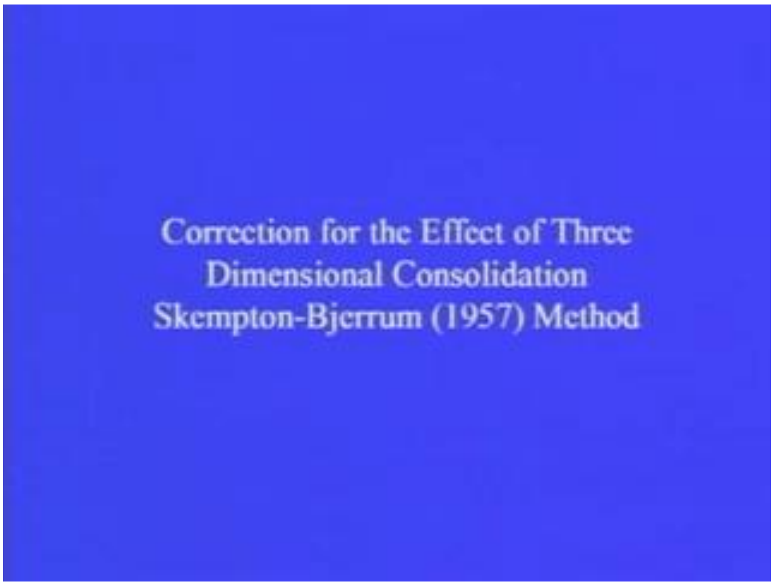
So, in order to determine, effective pressure due to the overburden at 2 point 5 meter, which corresponds to the middle of consolidating layer that will be equal to effective unit weight that is the submerged unit weight 0.82 multiplied by the depth that is 2.5 equal to 2.05 ton per meter square. Now, assuming, a load spread of 2 vertical is to 1 to find out the increase in stress at the middle of the consolidating layer. We get delta sigma equal to 11.5 now, 11 point is the net safe bearing capacity, which we have obtained earlier into this 2 square divided by 3 square. That is equal to 5.1 ton per meter square. So, by using the approximate approach of 2 is to 1 we can find out delta sigma now, when we substitute all these values into consolidating consolidation equation we get consolidation settlement as 94 millimeter.

(Refer Slide Time 38:07)



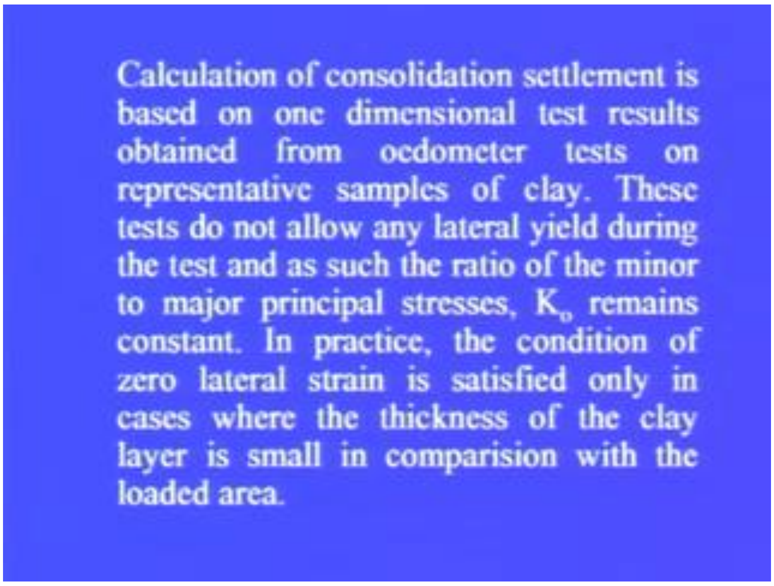
Now, whatever consolidation settlement or whatever settlement, we have discussed so far those settlements are to be corrected for three dimensional, consolidation. Now, here whatever, settlements we have determined that is for the 1 dimensional case, but actual in c 2 soil is confined from all the directions. So, that is the case of 3 dimensional consolidation.

(Refer Slide Time 38:34)



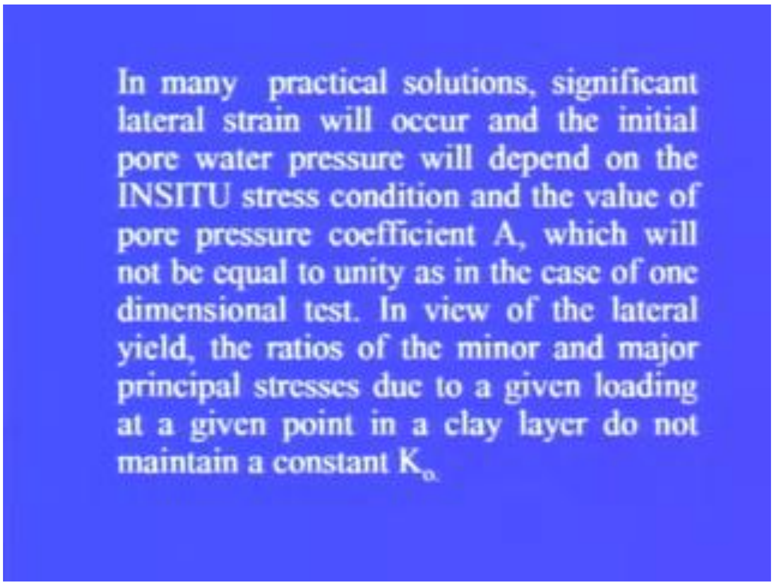
So, we will have to make corrections, on the settlement, which we obtain from 1 dimensional consolidation test data. A method is suggested by Skempton's.

(Refer Slide Time 38:46)



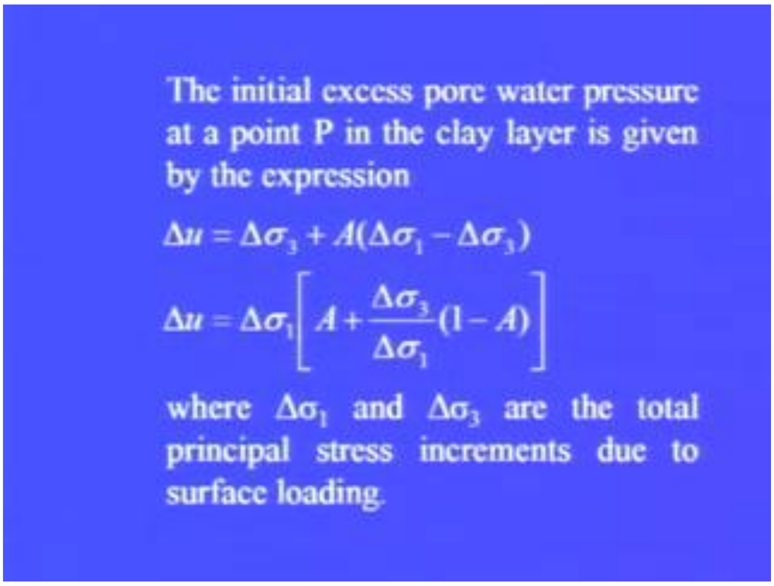
Now, calculation of consolidation settlement is based on, 1 dimensional consolidation test results, obtained from oedometer tests on representative samples of clay. These test tests do not allow any lateral yield during the test and as such the ratio of minor to major major principal stresses, that is K naught remains constant. In practice the condition of 0 lateral strain, satisfied only in case where the thickness of the clay layer is small in comparison with the loaded area.

(Refer Slide Time 39:24)



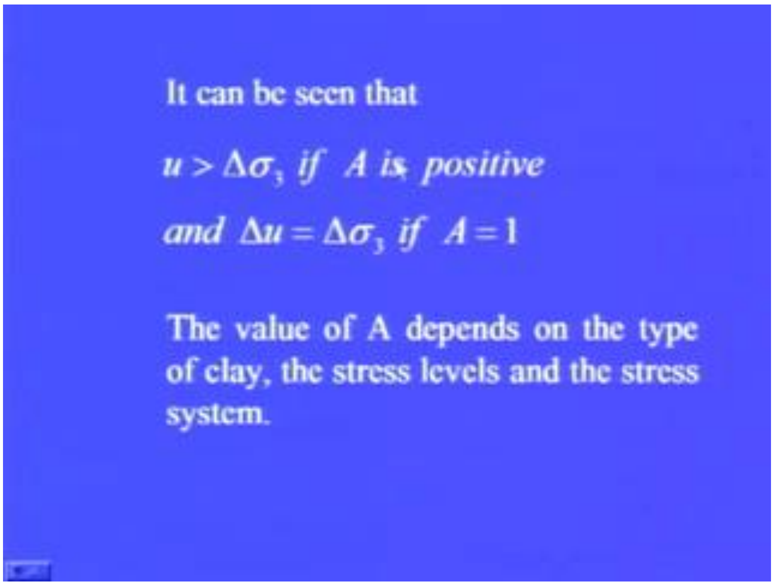
In many practical solutions significant lateral strain will occur and the initial pore water pressure will depend on the INSITU stress condition, and the value of pore pressure coefficient A. Which will not be equal to unity as in the case of 1 dimensional test in view of the lateral yield the ratios of the minor and major principal stresses dye to a given loading at a given point in a clay layer, do not remain maintain a constant K naught.

(Refer Slide Time 39:59)



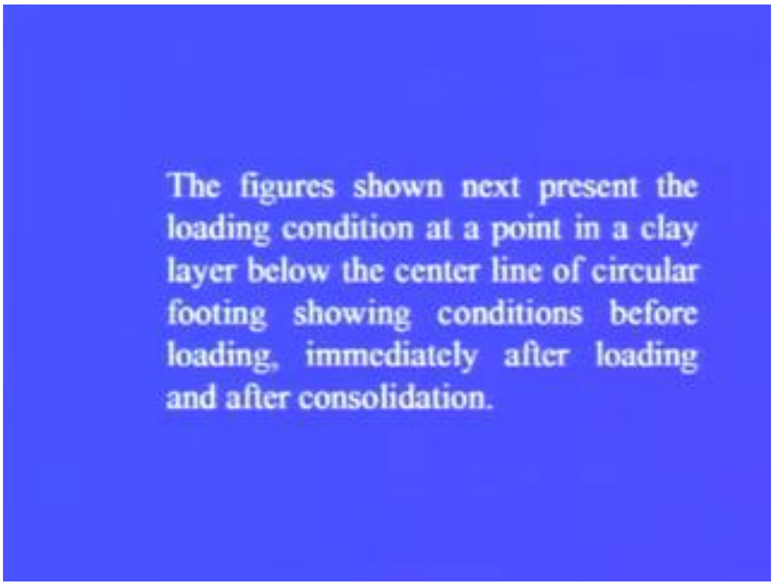
Now, this initial excess pore water pressure, at a point P in the clay layer is given by the expression delta u equal to delta sigma 3 plus, A sig delta sigma 1 minus delta sigma 3 this is the given equation given by Skempton. And delta u is the increase in pore water pressure, which we call as the excess pore water pressure, and this is the pore water pressure due to dissipation of which consolidation settlement takes place. And that will be equal to increase in the minor principal stress delta sigma 3 and multiplied by a pore pressure coefficient A, multiplied by delta sigma 1 minus delta sigma 3 that we can. We call it as increase in the deviatoric stress. So, this delta u can be written as delta sigma 1 total multiplied by A plus delta sigma three upon delta sigma 1 1 minus A. Where delta sigma 1 and delta sigma 3 are the total principal stress, increments due to surface loading.

(Refer Slide Time 41:17)



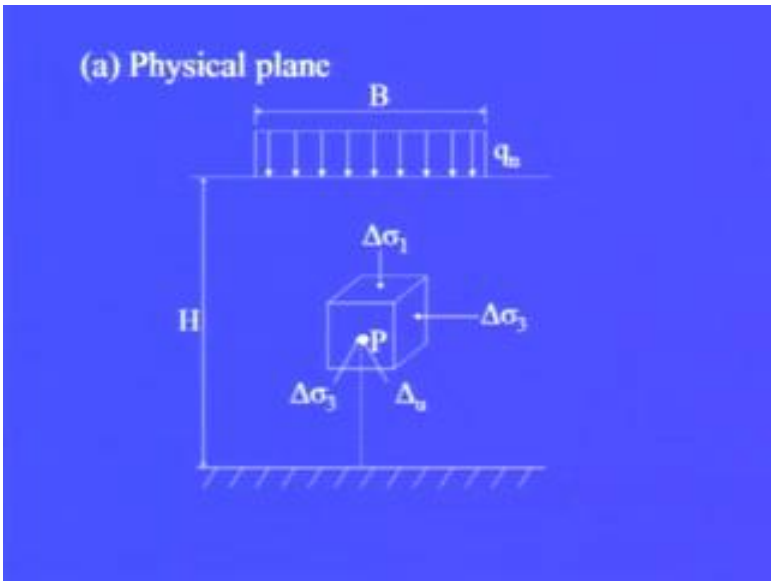
It can be seen that, u is greater than delta sigma 3, if A is positive and delta u is equal delta sigma 3 if A is equal to 1. The value of A depends on the type of clay the stress levels and the stress system, it also depends on the over consolidation ratio.

(Refer Slide Time 41:42)



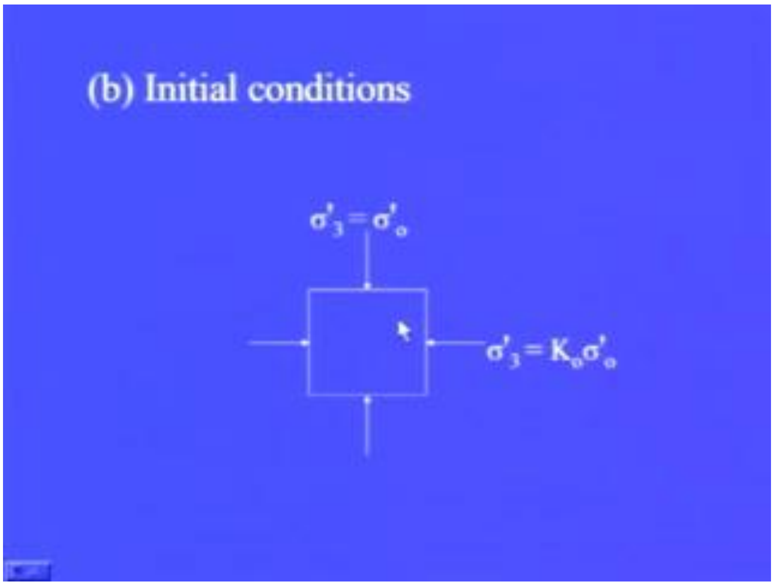
The figure shown, next present the loading condition at a point in a clay layer below the center line of circular footing showing conditions, before loading immediately after loading and after consolidation.

(Refer Slide Time 41:58)



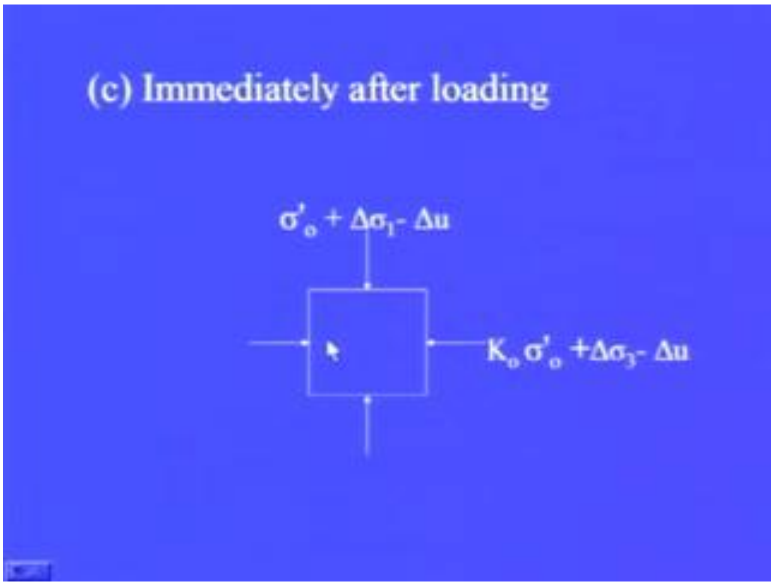
Now, this is a point p which is consider in the consolidating layer of thickness H. Now, at this point P, if we consider a differential element, the then the stress conditions are delta sigma 1, delta sigma 3, delta sigma 3 and the pore water pressure delta u. So, if a load is applied of net load intensity qn. So, this, the stress conditions at point P are as given below.

(Refer Slide Time 42:37)



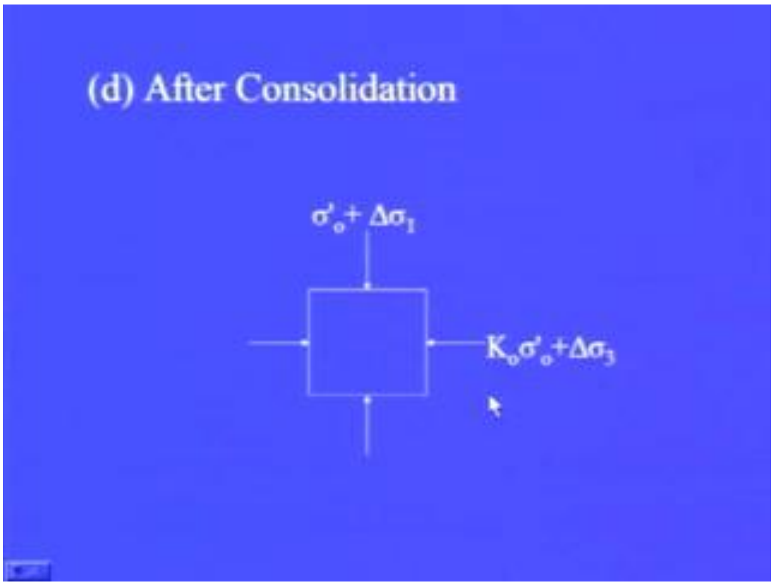
Now, initial conditions are sigma 3 dash equal to sigma 0 dash whereas, in the vertical direction. This can, we can determine as gamma dash into the depth under consideration of the point P, below the ground surface. And sigma 3 dash is equal to K not sigma 0 dash that is the lateral stress.

(Refer Slide Time 43:02)



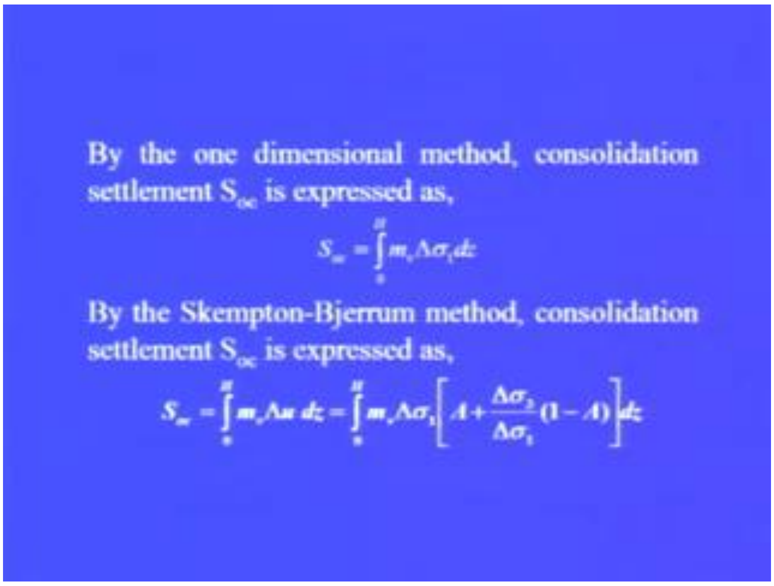
Now, immediately after loading there will be an increase in delta sigma 1. So, this will be sigma 0 dash plus delta sigma 1 minus delta u. And lateral stress will be K not delta sigma 0 dash plus delta sigma 3 minus delta u.

(Refer Slide Time 43:25)



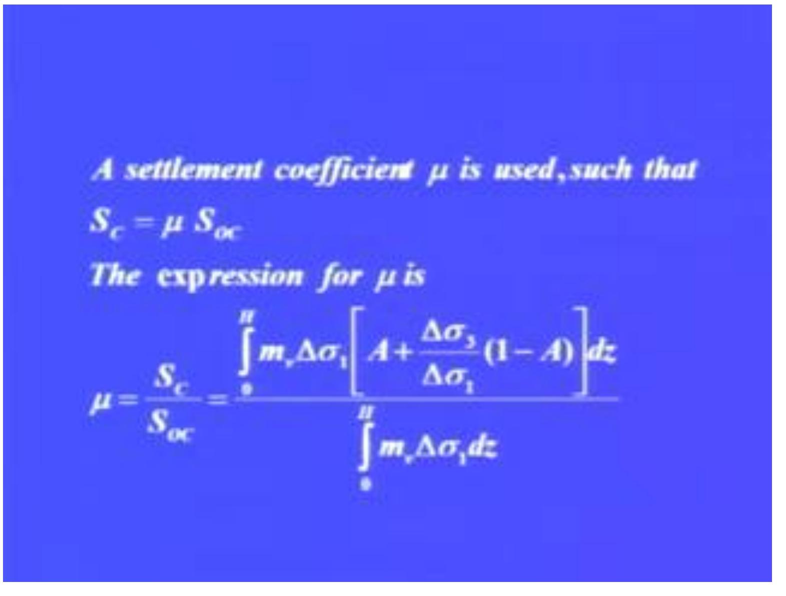
After consolidation is over all the delta sigma u, which has got increased has got dissipated. And that is the excess pore water pressure. Now, after consolidation process over then this will, become sigma 0 dash plus delta sigma 1 k 0 delta sigma, sigma 0 dash plus delta sigma 3.

(Refer Slide Time 43:50)



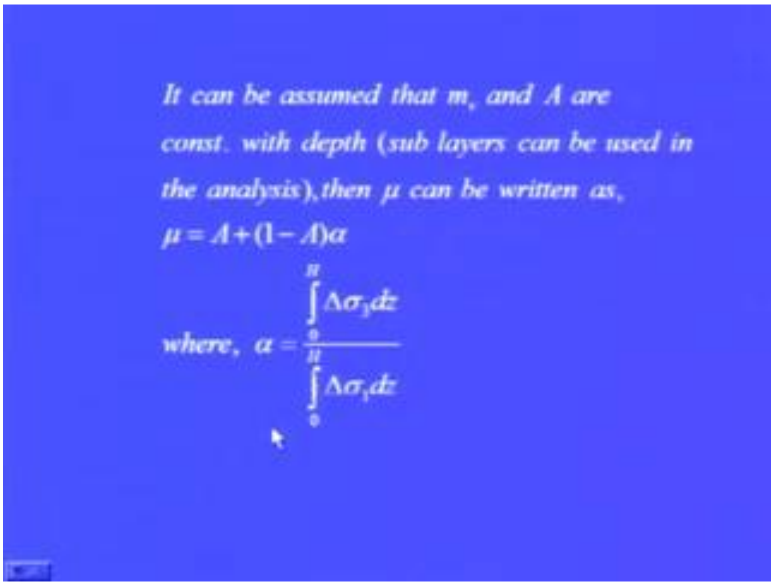
By the 1 dimensional method, consolidation settlement is expressed as, in terms of coefficient of volume change 0 to H mv delta sigma 3 dz, where d dz is the differential layer of consolidation. By Skempton-Bjerrum method consolidation settlement is expressed as this in in place of delta sigma. We substitute this as delta u and it will be equal to 0 to H mv delta sigma 1 A plus delta sigma 3 upon delta sigma 1 1 minus A into dz.

(Refer Slide Time 44:30)



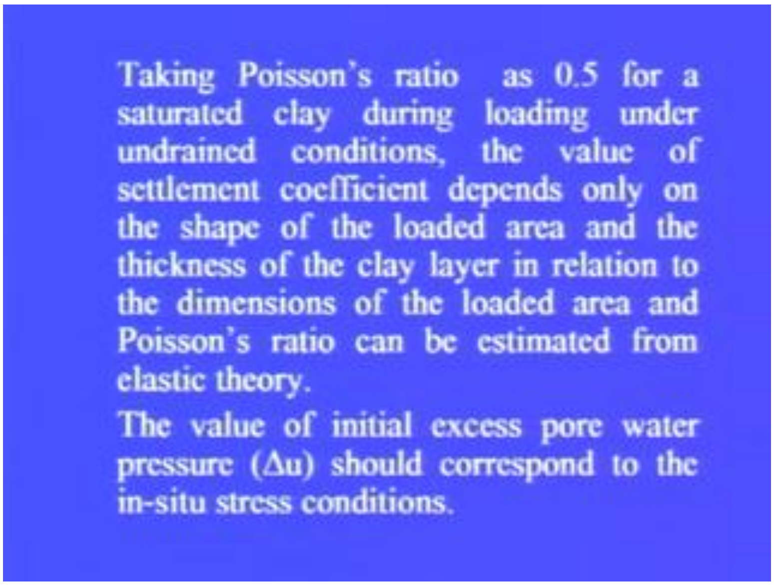
A settlement coefficient mu is used such that Sc equal to mu into Soc which we determine from the method. The expression, for mu will then be equal to Sc upon Soc. So, this will be equal to 0 to h mv delta sigma 1 in bracket multiplied by A plus delta sigma 3 divided by delta sigma 1, 1 minus A into dz divided by 0 to H mv delta sigma 1 dz where this we determine from the 1 dimensional consolidation equation.

(Refer Slide Time 45:08)



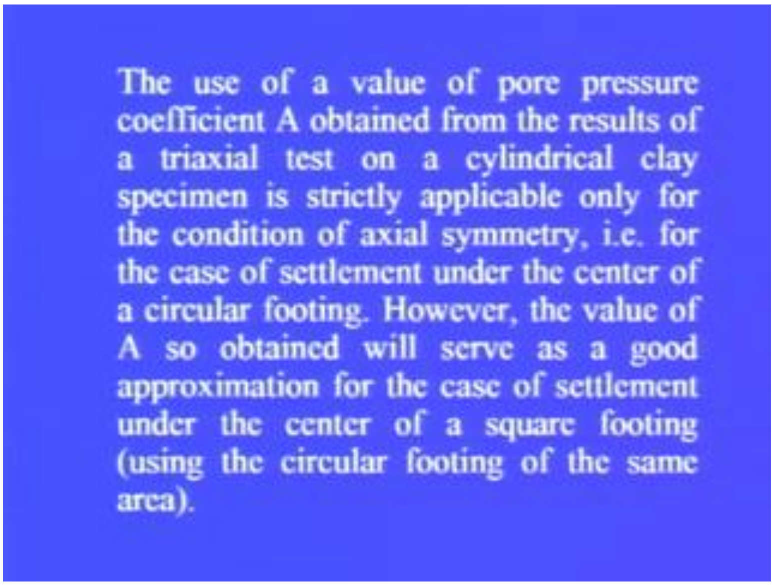
It can be assume that mv and A are constant, with depth. Sub layers can be used in the analysis. Then mu can be written as mu equal to A plus 1 minus A into alpha where alpha factor is given, by 0 to H delta sigma 3 dz divided by 0 to H delta sigma 1 dz.

(Refer Slide Time 45:31)



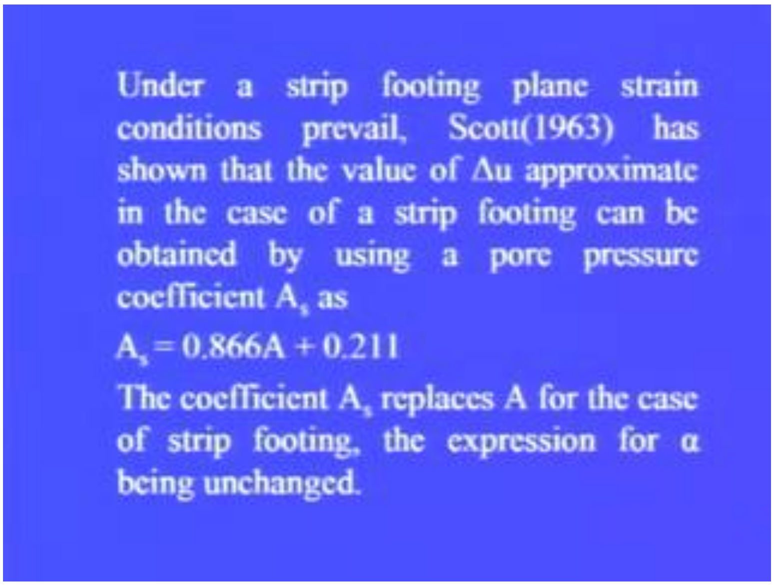
Taking Poisson's ratio as 0.5 for a saturated clay, during loading under undrained conditions the value of settlement coefficient depends, only on the shape of the loaded area. And the thickness of the clay layer in relation to the dimensions of the loaded area, and Poisson's ratio can be estimated from elastic theory. The value of initial excess, pore water pressure delta u, should correspond to the ((refer time: 45:59)) stress conditions.

(Refer Slide Time 46:01)



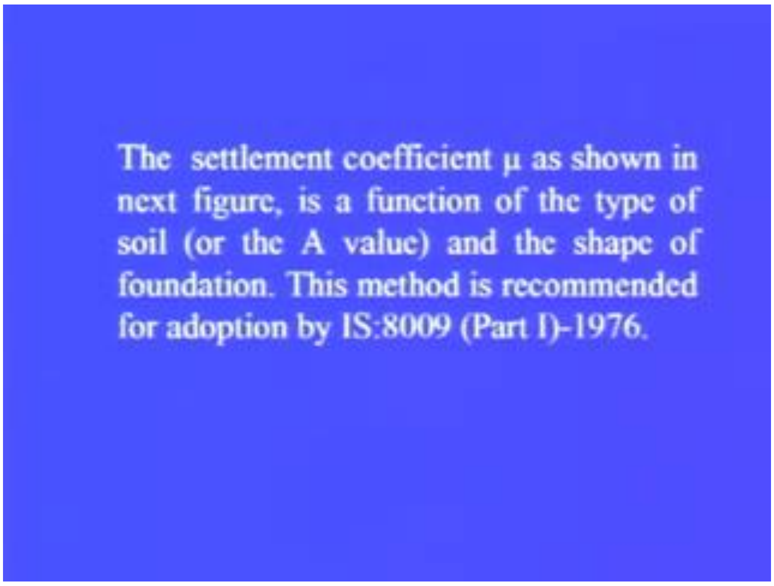
The use of a value of pore pressure coefficient A, obtained from the results of a triaxial test on a cylindrical clay specimen is strictly applicable only, for the condition of axial symmetry. That is for the case of settlement under the center of a circular footing, however, the value of A. So, obtained will serve as a good approximation for the case of settlement under the center of a square footing using the circular footing of the same area.

(Refer Slide Time 46:34)



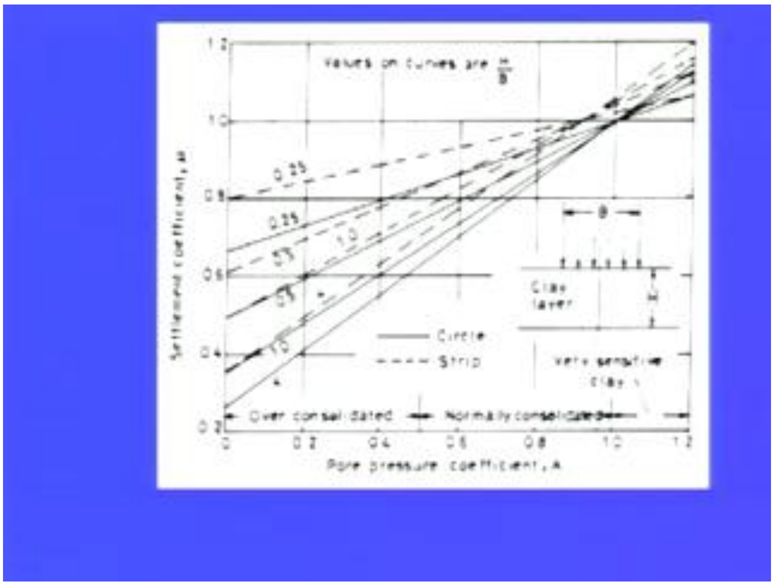
Under a strip footing plane strain conditions prevail. Scott 1963 has shown that, the value of delta u approximate in the case of a strip footing, can be obtained by using a pore pressure coefficient A as, As equal to 0. 866 A plus 0.211 the coefficient as, replaces A for the case of strip footing the expression for alpha being unchanged.

(Refer Slide Time 47:05)



The settlement coefficient mu, as shown in next figure is a foundation of the type of soil or the A value and the shape of the foundation. This method is recommended for adoption by IS 8009 part I 1976.

(Refer Slide Time 47:22)



So, this value of mu, settlement coefficient mu can be obtained from this particular figure. Now, here this is the index figure in which this B is the width of the loaded area, H is the thickness of the clay layer and pore pressure coefficient A is given here ranging from 0.2 to 1.2 for various clays like. Over consolidated clay, from here to here for normally consolidated clay from 0.45 to 1 and for very sensitive clay 1 to 1.2. Now, this settlement coefficient is a function of the ratio H by B. So, these are given for different values of H by B, this settlement coefficient is also a function of the shape of the footing. So, for a circular footing, these are solid, solid lines are for the circular footing and dash lines are for the strip footing.

So, this particular chart can be used to determine value of the settlement coefficient mu, for different values of over consolidation pore pressure co coefficient A. So, for we can make use of this for example, let us say we want to determine the value for a circular footing. So, depending upon the value of A, let us say value of A is 0.4. So, for the circular footing for H by B ratio equal to let us say 0.5 this is the H by ratio equal to 0 point 5 and from this, we can get this value of settlement coefficient. This settlement coefficient can be used to correct the settlement to take into account the 3 dimensional state of the stress. So, in today's lecture I have tried to explain the methods to determine consolidation settlement.

And then, this consolidation settlement is actually for the 1 dimensional consolidation condition whereas Nc 2 conditions are always 3 dimensional. So, we will have to make corrections for 3 dimensional state of a stress and I have this discussed this method given by Skempton to find out the coefficient of a settlement. That is the settle settlement coefficient mu for different shapes of footing and for different value of H by B ratio. And if we know the value of A we can get this settlement coefficient and the settlement can be corrected. In the next lecture, I will further extend different other corrections, which are used in the settlement one dimensional consolidation settlement and then finally, how to obtain the settlement of the structure.

Thank you