Soil Mechanics
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Lecture – 41
Consolidation and Settlement
Lecture No. 8

Students we start once again yet another lecture. This is the eighth lecture on the topic of consolidation and settlement. Let us take a look at the first slide. This shows what we have covered particularly the last lecture. We covered 2 very important concepts in the last lecture and also saw a few examples to illustrate some of the concepts we had seen a little earlier. The topics we covered were the so called secondary consolidation and the so called radial consolidation. Now its obvious that the consolidation phenomenon that we studied in the first few lectures was only due to the hydrodynamic lag that is the expulsion of water from the pores and because of the delay or the slow process of expulsion of water there was a lag between the transfer of the load to the solid grains and the expulsion of water. So it was called the phenomenon of hydraulic or hydrodynamic lag.

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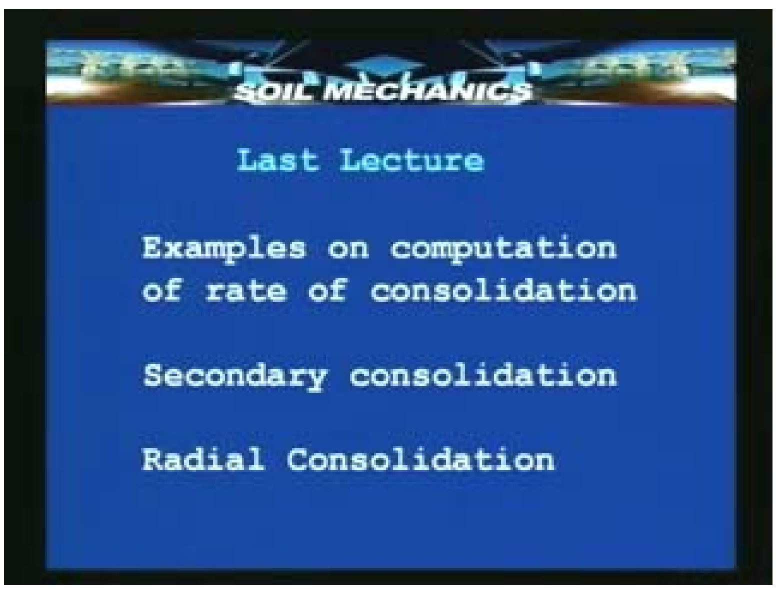

Now it has been observed that even though the consolidation is 100% complete due to the phenomenon of hydrodynamic lag that means the entire load increment that has been applied to the soil has been transferred to the soil grains. It's been found that the settlement of soil continues. Actually the observation has been made in a sample in the laboratory in the odometer test. It shows that the settlement continues even after transfer of the entire load increment that had been applied and this phenomenon is due to a number of reasons but primarily due to what is known as plastic lag that these solid particles go on adjusting themselves even at constant stresses they continue to undergo some deformation even at constant stress and readjust themselves. And this movement, gradual movement of the particle and readjustment even under the constant stress is what is known as the secondary compression behavior or secondary consolidation. In the last we just had a glimpse of what is secondary consolidation, a little bit about the basic reason for the secondary consolidation and its explanation.

Then it was followed up with a brief mathematical explanation of three dimensional consolidation phenomenon, you remember that we simplified the actual consolidation problem considerably. We used the concept that had been proposed or postulated by Terzaghi and under ideal conditions we assumed that consolidation is one dimensional. That is the load is applied vertically and expulsion of water also in vertical direction, may be in one direction or may be in both downward and upward direction depending upon the drainage conditions. But actual phenomena of consolidation is three dimensional and a special case of interest of three dimensional consolidation which is known as the radial consolidation phenomenon and which has got a tremendous practical applications also I expose to you in the last class. Let us take a look at the next slide.

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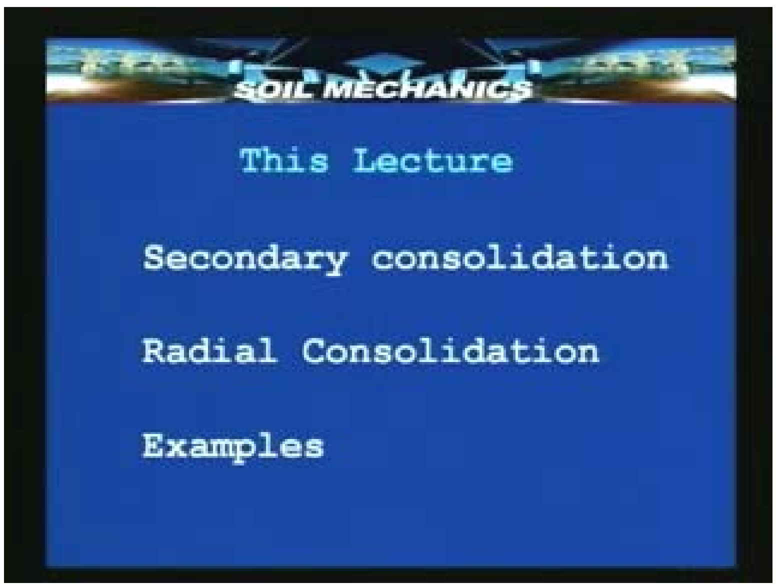

In this lecture we will review what we saw on secondary consolidation last time and add a little more information to it. Similarly we will review the phenomenon of radial consolidation as we understood last time and a little more material and then subsequently solve a few problems. Actually this is a penultimate lecture, there will be one more lecture the last one where I will be covering the sequence of examples, graded examples give some general information and also broadly give a summary and wind up this set of lectures on consolidation and settlements. So continuing this let us take secondary consolidation, let us quickly read what is here is in this slide.

The compression due to hydrodynamic effect is known as primary consolidation. This is very well known, we have already discussed in substantial detail. The consolidation that we have been discussing all this time is primary consolidation but some clays continues to exhibit compression and reduction of voids even beyond the primary consolidation stage at constant effective stress due to what is known as the plastic lag and this is what is known as the secondary consolidation. So this is merely an emphasis on what I have just now mentioned. So that is secondary consolidation is essentially due to plastic lag.

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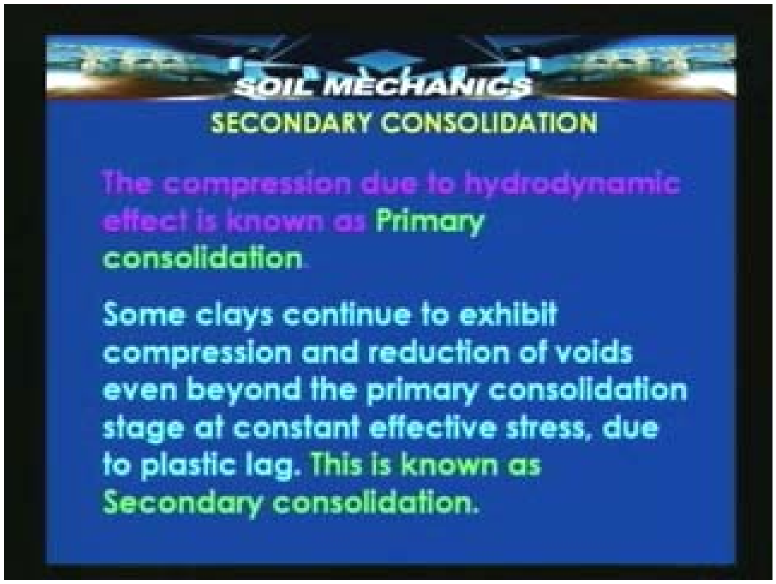

Let's take the next slide, the probable reasons or the explanations or the various phenomena that are simultaneously taking place during secondary compression are There are 4 in numbers, they are listed in this slide, the first one is the most important point, here it is the phenomenon of compression is not due to the phenomenon of expulsion of water and reduction or dissipation of excess hydrostatic pressure. So point number one is this phenomenon of secondary consolidation is not governed by dissipation of excesses pore pressure. Then what is going out. There is some water in spite of all this water that is being expelled out of this soil going out, there is still some water always adhering to the solid particles which can only be removed by means of either some mechanical action or by means of heating. So the well-known examples are to remove that water we normally put the soil in the laboratory in an oven and heat it at 105 degrees for 48 hours or 24 hours as the case may be. So this is nothing but a thermal way of removing that water which is adhering to the surface of the solid particles.

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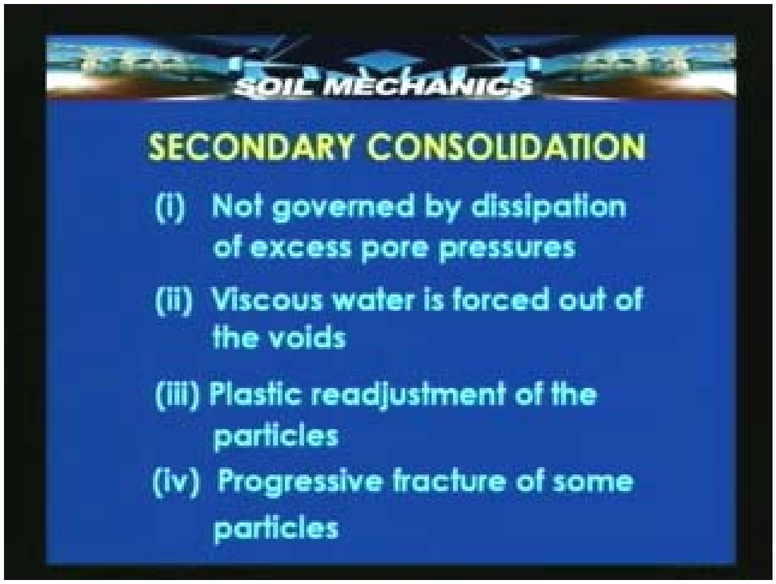

A mechanical way an example is compaction. During compaction some amount of water also goes out that which is adhering to solid particles do go out but basically this water which is viscous in nature is what is driven out during constant effective stress during the secondary consolidation process. And it is during this process of viscous water moving out that plastic adjustment or deformation of particle takes place and so the third point is plastic readjustment of the particles.

And lastly the particles not only just move and reorient, readjust, relocate themselves under the effective stress sometimes they even undergo some breakage or some damage some fracture. So progressive fracture, gradual fracture of some of the materials or some of the particles of the soil solids is also contributing to the secondary consolidation. So these are the 4 phenomenon that are going on simultaneously. Let's see a typical consolidation, dial gauge reading verses time graph of an odometer test, let us take a look at this slide.

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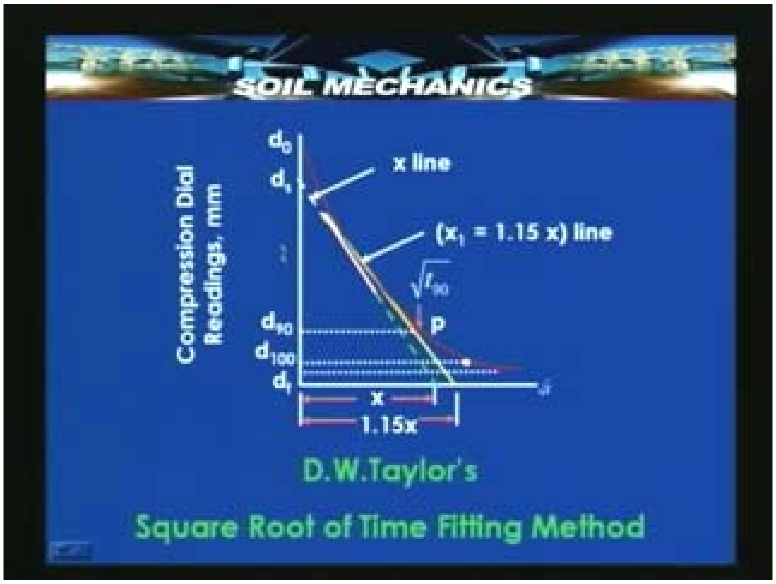

Here I have got the dial gauge reading and here I have got the root of time plotted. This is well known Taylor's square root of time fitting method, the graph for this method which we had discussed earlier. My endeavor is to focus your attention on these dial gauge reading which are shown here, d_0 is the initial dial gauge reading and when you apply a certain amount of load, it takes a little bit of time for the load to settle down and seat itself properly and that's because some air may there in the pores of the soil and unless they are either compressed or expelled out, the dial gauge reading will not be stable. So immediately on application of the load the dial gauge reading changes to a value of d_s and in the square root of time method this location of d_s is obtained by drawing an extension of the straight line portion of this dial gauge root of time method graph.

So d_s actually is little difficult locate the dial gauge reading at which the actual consolidation phenomenon starts that is the dial gauge reading d_s actually corresponds to u is equal to 0% is slightly difficult to locate this and that's why we have this graphical procedure to locate d_s in this particular method. We have also seen another graphical procedure in the lock time method of casagrande. Now this point corresponds to 90% consolidation, if we allow further consolidation and allow the reading to stabilize then we come to d_{100} that corresponds to 100% consolidation. That means the dial gauge reading is practically stabilized, there is no noticeable change in the dial gauge reading within a reasonable period of time or at least the change in dial gauge reading is much less than the least count of the dial gauge count and therefore cannot be detected by the dial gauge.

And then even after reaching this 100% consolidation stage keeping the load constant, if we allow further time after about 24 hours or a little more, the dial gauge shows a slightly different readings and that is the final real final stabilized reading and that's the one corresponds to the secondary consolidation stage. Next slide we will show you how we compute, therefore what is the primary part of the total consolidation that we have observed in an odometer test. So in a typical odometer test we start with d_0 go to d_s then d_{90} d_{100} and then finally d_f, so d_0 to d_f is the total compression.

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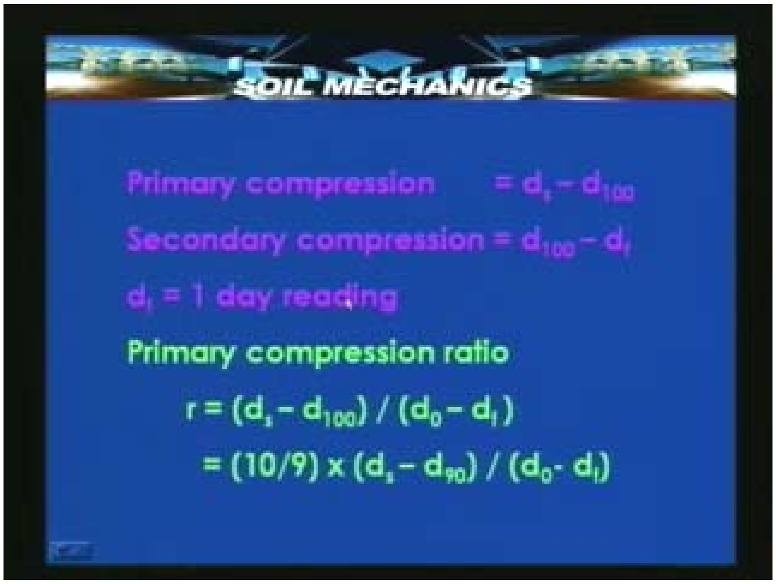

And primary compression portion out of that corresponds to d_s that is from d_s to d_{100} and the secondary compression part is from d_{100} to d_f. Now d_f is usually taken after 24 hours or sometime 48 hours. The ratio of d_s- d_{100} that is the primary compression part to the total compression d_0- d_f is what is known as primary compression ratio. If we are using the logarithm of time method of determining c_v there we determine d_{100} directly and therefore that can be used in this to find out the primary compression ratio. But if we are using the square root of time method, we actually locate only 90% consolidation point precisely, 100% point is only an esteemed an estimated value it's not determined. And therefore in order to get the primary compression ratio we need to multiply d_s- d_{90} which is what we mentioning in the experiment by a factor of 10 by 9, so as to get or transform this into a reading corresponds to d_s- d_{100}. This is the primary compression ratio that is the part of primary compression out of a total compression, so the remaining part is the secondary compression. Now a little more about the secondary consolidation.

If you take a look at this curve you find that the 60% point corresponds to somewhere something here and the deviations from the straight line nature of the curve start somewhere here itself. So this makes one thing as to why the deviation is occurring. It has been noticed that around 60% consolidation itself the secondary consolidation is gradually starts manifesting itself. Though in a very small way minor way it starts coming into a force even at the 60% of the primary consolidation. However we ignore this and we generally consider the secondary consolidation to be occurring only after the 90 or 100% consolidation stage of the primary consolidation.

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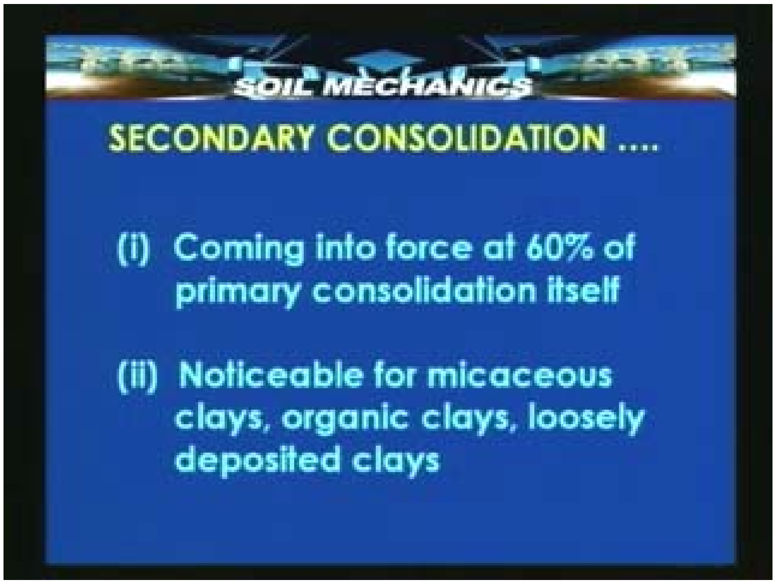

Now why does it occur? In what kind of soil does it occur? It is particularly noticeable in micaceous clays, organic clays and in other loosely deposited clays. Normal inorganic clays do not exhibit much of this secondary compression. The secondary compression is more noticeable in highly compressible material like micaceous material organic material etc which are present in the clays under certain condition of their formation. Now how does one compute this secondary compression and how it progresses or in other words in the case of primary consolidation we where interested in two aspects basically the total compression or total settlement and time rate of settlement or time rate of compression. In the secondary consolidation as well it will be of interest to us to also know the rate at which secondary consolidation proceeds because as we saw in the discussion just preceded, we can compute the total secondary compression from the dial gauge reading verses time curve from the odometer test data. So the total secondary compression is easily obtainable however the rate at which it takes place is also important and that can be done by approximating the e p curve again.

Suppose we take the void ratio and pressure. Actually during the secondary consolidation the pressure or the effective stress remains constant and the void ratio is therefore varies not with respect to the pressure but with respect to time for a given pressure for a given constant load increment the void ratio actually varies with respect to time. So it is the e versus t curve which of important and that will tell us at any given time what is the change in void ratio and correspondingly what is the amount of compression or the amount of secondary consolidation that has taken place. The change in void ratio is a direct measure of the degree of consolidation that has occurred during the secondary stage. Now what is the relationship between e and t? As I said this is a plastic phenomenon, the secondary consolidation is a plastic phenomenon and basically it is logarithmic in nature and that has been burn out by experiments with the odometer and therefore e versus t curve is usually logarithmic, the relationship is exponential and therefore we generally plot e against log t which generally gives us a fairly linear relationship between e and t.

Take a look at this slide, here void ratio is plotted against logarithm of time and the relation is very nearly a straight line. I have idealized into a exact straight line here however in practice it will be very nearly a straight line and it can be approximated very easily into very reliably into a straight line.

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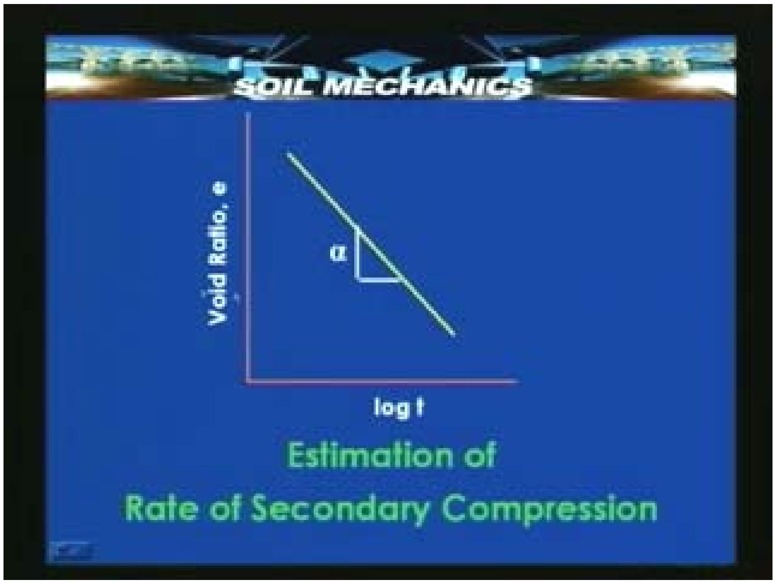

The slope of this straight is obviously a measure of a relationship between e and t and this slope suppose that is alpha it is this slope alpha which is a measure of the rate at which secondary compression take place. Let us take a look at the next slide. Suppose I take two void ratios, two successive void ratios corresponding to different time periods time instance rather. Then e_1- e_2 as per this e log t curve which we saw just now during the process of secondary consolidation as to be equal to alpha times logarithm of t_2 upon t_1. So the change in the void ratios is alpha the slope of the graph into log 10 t_2 upon t_1.

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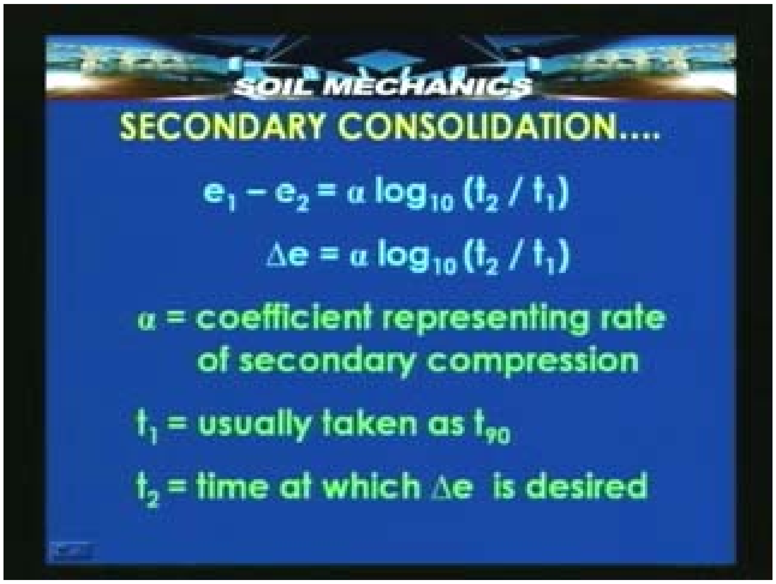

So alpha that is coming here has the slope of the curve is the constant between the two relationship and that constant connecting the 2 parameters is known as the coefficient of secondary compression, t_1 and t_2 are 2 time instance and generally t_1 is taken as the time instance corresponding to t_{90} and t_2 is any other time during the secondary consolidation stage for which we take the change in void ratio.

So if we assume the e t relationship to be a logarithmic relationship then we have a way to compute the rate of compression during the secondary compression stage. That's by this formula; delta e is equal to alpha log 10 t_2 upon t_1. So if this straight line relationship is assumed then the slope of the straight line can be computed by knowing two void ratios and corresponding times and it will be simply equal to the ratio of e_1- e_2 to log t_2 upon t_1.

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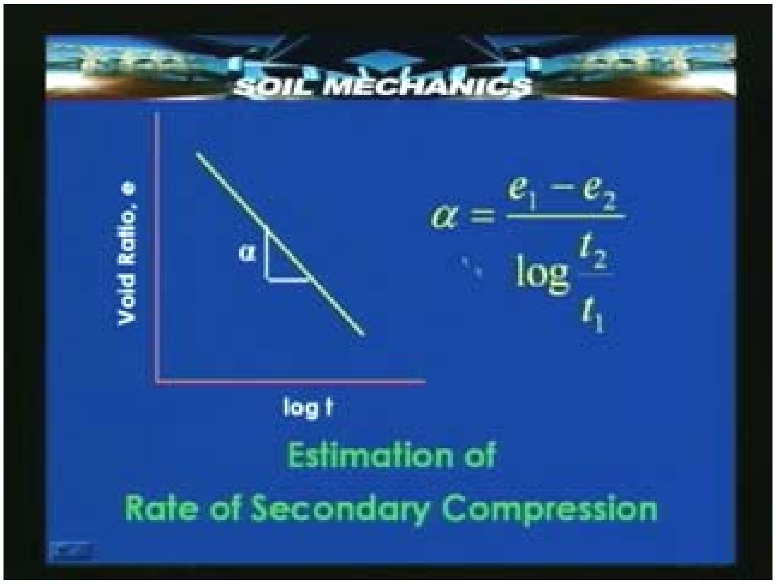

So if from an experiment we plot the e t curve and compute alpha then that will serve us in helping to compute the degree of secondary compression at any given time. Now we pass on to 3 dimensional compression or consolidation particularly with reference to radial consolidation.

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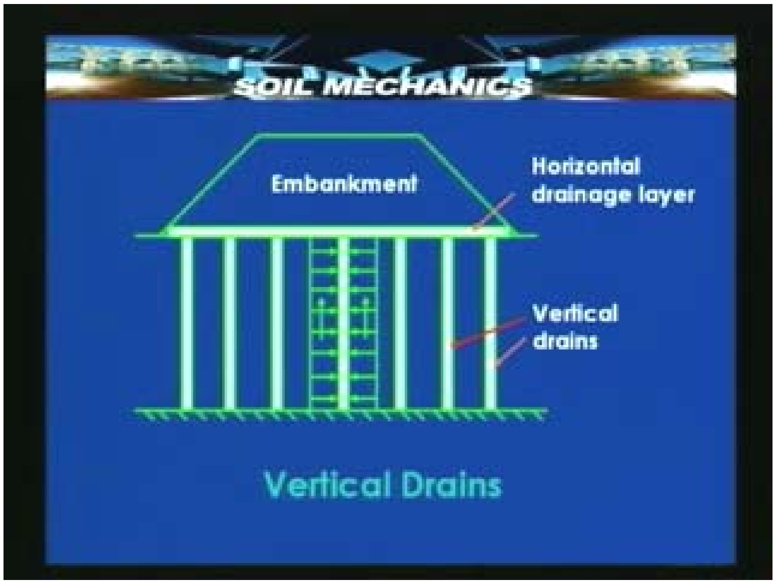

Take look at this slide. We have here as I already explained in the last lecture an embankment sitting over soil surface. There is a compressible soil layer here and what you see here driven or placed or erected in this compressible soil layer are vertical drains. These are nothing but vertical bore holes which are filled with some highly pervious materials like sand and on top again there is an embankment below which, at the base of which we have a horizontal drainage layer that is a layer consisting of some highly pervious material. And this is a typical arrangement which is known as the pre loading arrangement. I had mentioned in the last lecture but let us recapitulate.

Suppose we have a highly compressible soil and we want to build a structure on this, may be a building may be a tank. Now some of these buildings may not be able to withstand the time rate at which the consolidation takes place. If the consolidation takes place over a very long duration of time, these structures may not able to adjust beyond a certain limit. And therefore it is seen that it is advantageous in such cases to first consolidate the soil and then put the structure because we have seen a phenomenon known as pre compression in which we said that in the past a soil is subjected a certain amount of load which for various reasons is taken of later on either naturally or artificially by natural process or by manmade techniques.

Now that means a certain soil if it has already undergone consolidation under a pressure larger than what you find now then that's a pre compressed soil and an over consolidated soil. And such a soil if it is subject to load gain until it reach load up to which it had been compressed in the past it won't show any compression or even if it show it show very marginal compression only. But beyond that stress value, corresponding to the previous pressure it will start behaving like a normally consolidated soil and show compression as any other soil does. So if we take e versus p curve, if this is the virgin consolidation curve and suppose some pressure is acted on a soil and reached a value corresponding to p maximum and then the stress has retreated due to either natural or other causes. And once again now by building a structure we are overloading it and up to the value of p max pressure corresponding to p max there will be very little compression because the soil has already experienced under that compression and it will continue like a normally consolidated soil beyond this point corresponding to p max.

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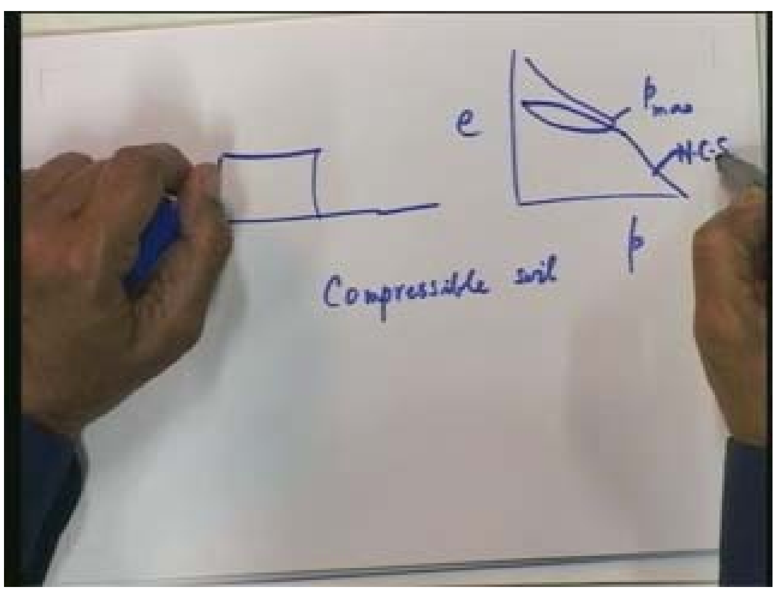

And this shows that if we apply a stress like p max on a certain soil which in fact is going to be the stress that a building is going to apply then what it means is that any soil which has already been pre compressed with a pressure corresponding to that which is going to come to a due to a structure will now not show any settlement because it has already settled under the load which is expected to. So this is taken an advantage of in practice and very often in such situations where we want an accelerated consolidation then we use a method known as the method of precompression or method of preloading with sand drain preloading with sand drain. These are nothing but vertical drains filled with sand pervious material, so that flow from the soil under pressure also takes place radially and then flows out and the water flows out. This is to increase the number or the length of the drainage path so that consolidation takes place faster because we saw during primary consolidation that it is the hydrodynamic lag which is responsible for the time dependency of the compression and obviously therefore time of compression can be considerably reduced if the rate of expulsion of water is accelerated.

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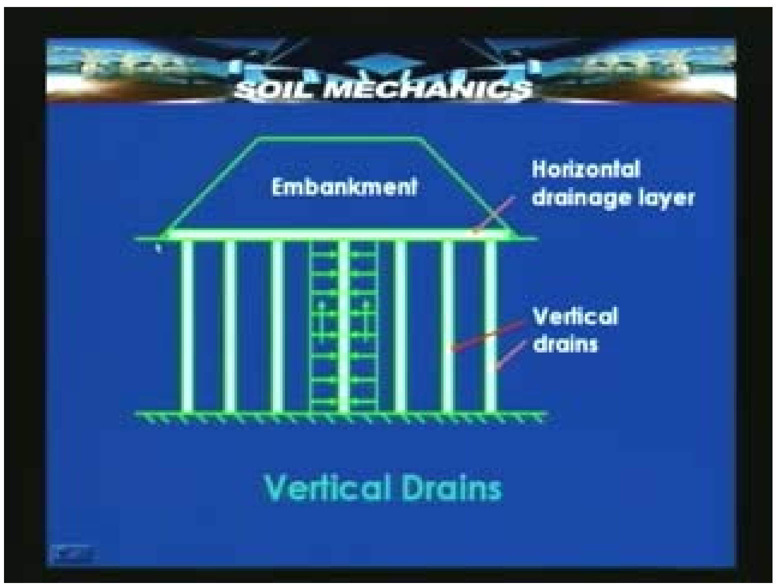

And this is an arrangement where an embankment is built over a natural soil and this serves as a preload and under this load if which sand drain if we allow this soil to get consolidated. Then the soil undergoes compression due to this preload and now after it has undergone compression, you can remove this preload and replace with the actual structure and since already compression had taken place there will be no further compression and the structure can be expected to perform without distress comfortably. Now this preloading and compression is a phenomenon of radial consolidation because here we have water flowing radially as shown by these arrows and then subsequently water flows out vertically outward because there is a drainage layer here which carries the weather the water that comes up. So the excess hydrostatic pressure has now more avenues for dissipation and therefore the consolidation is faster.

If you look at this actually the compression is taking place not only radially but also vertically that is the compression is vertical in fact the flow of water is not only vertical but also radial and therefore we have to find out the resulting vertical compression as a result of two component, one is the usual vertical compression that takes place due to vertical flow and another is a vertical compression corresponding to radial drainage which is known as U radial. So we will see the method for computing the total average consolidation U as the function of the average consolidation due to radial drainage and average consolidation due to vertical drainage. It might also be worth noting here that the preload should be applied in such way that it doesn't cause failure of the soil. That is in an attempt to accelerate the consolidation of the soil if we indulgence some over resalece (Refer Slide Time: 29:34 min) loading that is excessive loading then shear failure may take place. So the preload that can be applied also has a limit. We need to compute the preload from point of view of ensuring no shear failure of the clay soil and thus the preload which you can apply, if that preload is equal to the load due to the actual structure. Then the actual structure when constructed will hardly experience any consolidation or any settlement but if the preload is unavoidably less than the actual load that is going to come on the soil. Then even under the actual load a little bit of consolidation will ensue but in practice there are ways in means to apply this preload in such a way by gradual construction that is the preloading embankment can be constructed in stages so that the final required load can still be applied without causing shear failure of soil.

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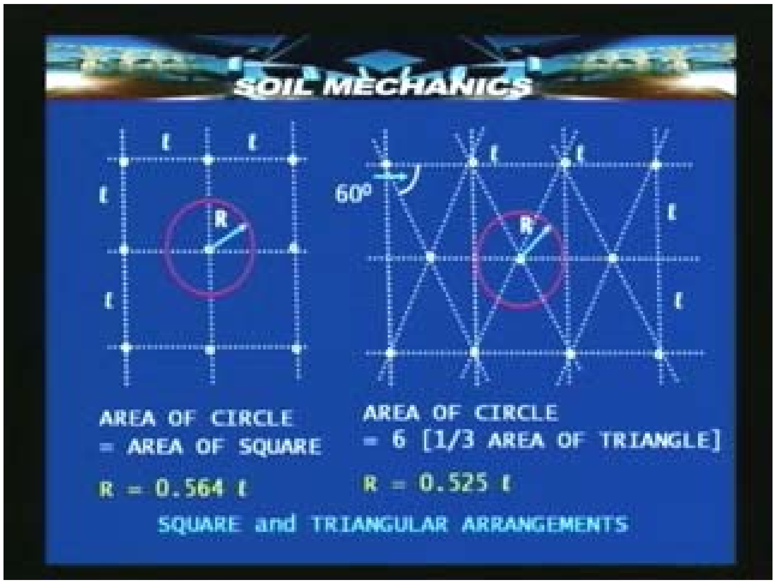

Now this drainage in the radial direction is a function of arrangement of drains. There are two kinds of arrangements, two patterns actually 2 geometrical patterns of arrangement, take a look at this slide. There are 2 arrangements shown here, one is the square arrangement and another is the triangular arrangement. You take the square arrangement here we have dimension l l l here and each dot here represents one sand drain. And what is the possible influence zone of one sand drain? It's very easy to understand that this sand drain and adjacent sand drain here will share the half the area lying in between them. That is a sand drain here basically because it is circular in or cylindrical in shape will have an influence zone which is also cylindrical, that influence zone which is cylindrical will have a radius r which will by normal ordinary detection can be shown to be equal to l by 2.

So this r is nothing but a radius which will take care of at least half the area that it can possibly covered within the square. Similarly if you take the triangular arrangement as shown here, this circular or the cylindrical drain will cover 6 triangular areas as shown here and each triangular area can be shown to be equal to one third of the bigger triangle. Now the sand drain must be so dispersed so layed out that it should cover the entire area that needs to be preloaded. Therefore as I was saying now each drain covers half this area but in fact if we arrange the drains this way then and if only half the distance is covered then there will be several pockets within each one squares which will remain uncovered and therefore this influence of this is so calculated that any drained here which get us to 4 of this square will have an aerial coverage equal to one fourth, one fourth, one fourth that is totally one square.

Similarly here one cylindrical drain will have one third one third, one third or in other words 6 triangles making up a certain amount of area which will be nothing but twice the area of one triangle. Then what is the value of this capital R which will have the same influence area as that of one square or for that matter here what will be the influence area of cylindrical drain which will have a radius or which will corresponds to a total area coverage equal to 6 times the smaller triangles. That means the radius of influence zone can be computed by equating the areas. So if I equate the area of circle with area of square I can get capital R which is the radius of the influence zone and if I equate the circle with 6 times one third of the area of triangle then I can get the radius capital R as 0.525 l. Let see how, this simple computation. So when we take these two arrangements and see the computation of capital R this is what we will find as shown in this slide. See for square arrangement, the area of the cylindrical drain must be equal to area of one square, it's only then that the drains will be able to leave no pockets of uncovered effect. So pi R square is equal to l square will R equal to l square by pi under root and that's equal to 0.564 l.

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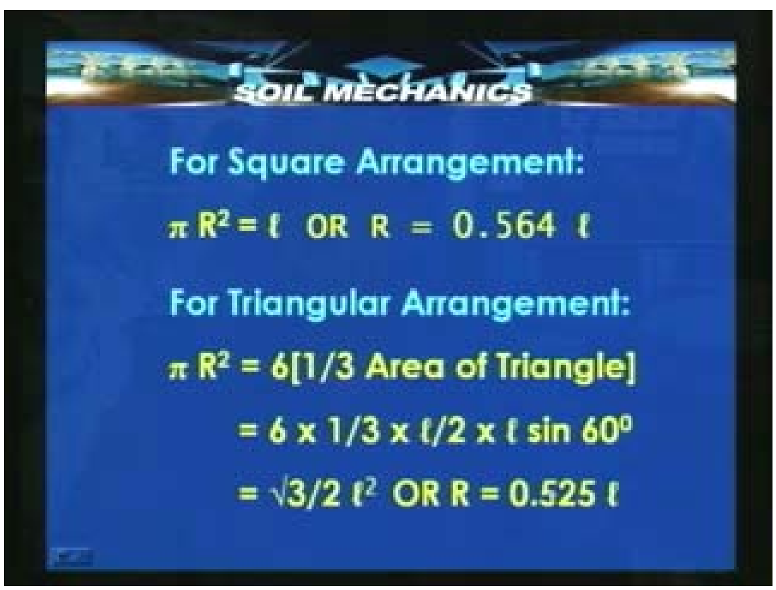

On the other hand if you take the triangular arrangement each cylindrical drain will have to cater to one third of the neighboring triangular triangle and therefore totally 6 triangles of this area should be equal to pi R square. So from here that we will find 6 into 1 by 3 into l upon 2 into l upon sine 60 which is nothing but the area of the triangle. See here l upon 2 into l this angle is 60 degree because this is a an equilateral triangular arrangement and therefore this length will be this height will be sine 60 degree and the area will therefore be l by 2 into l by 60 which gives us finally capital R equal to 0.25 l.

So this shows that the drains have slightly smaller radius in the triangular arrangement than in the square arrangement and if you take a look at the slide once again the previous slide see here the triangular arrangement is such that the drains in one row namely the second row here for example are located in the middle of the corresponding drains in the upper row and so on so in the bottom row. So the arrangement is probably a little more close and effective compared to the square arrangement. Therefore in practice depending upon the importance that's been attached to the preloading effort, if the preloading is important very important then we often go for this triangular arrangements but if a square arrangement which is a quietly a dispersed arrangement is sufficient in any given instance, then the square arrangement will be adopted. So the radius of that cylindrical influence zone is not just half the distance of this or half the spacing between the drains it is more than that.

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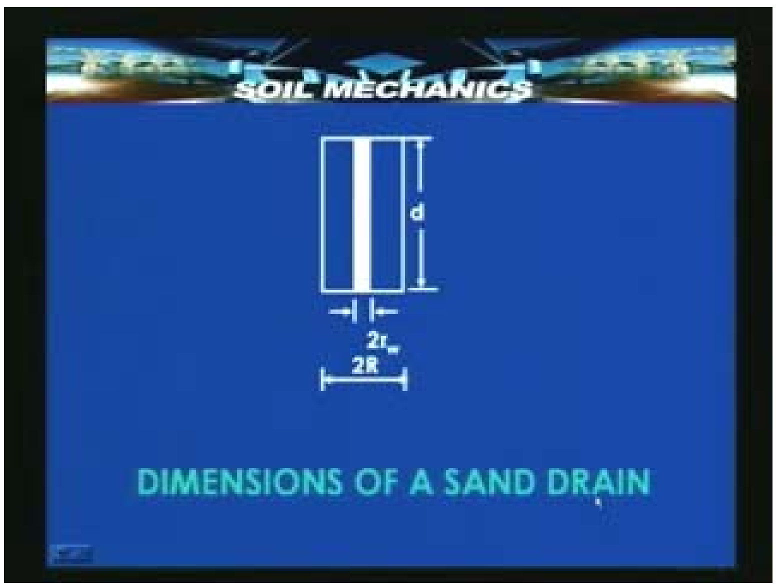

Now this shows the dimension of the typical sand drain that cylindrical drain which I have mentioned. So the total diameter is 2 R, total influence zone has a diameter of 2 R but the drain itself has got a radius r w or the diameter is equal to 2 r w where r w stands for radius of the well, the drain is often called as the well. And this is a height or the depth of the clay layer and the drain usually completely punchers or penetrates the compressible layers and therefore its depth or height is d. Now we saw that the consolidation phenomenon of this kind has a vertical component and a radial component and the vertical component was in fact no way different from the vertical component of the one dimensional consolidation. So we had 2 differential equation for the consolidation one for the vertical consolidation another for the radial consolidation part.

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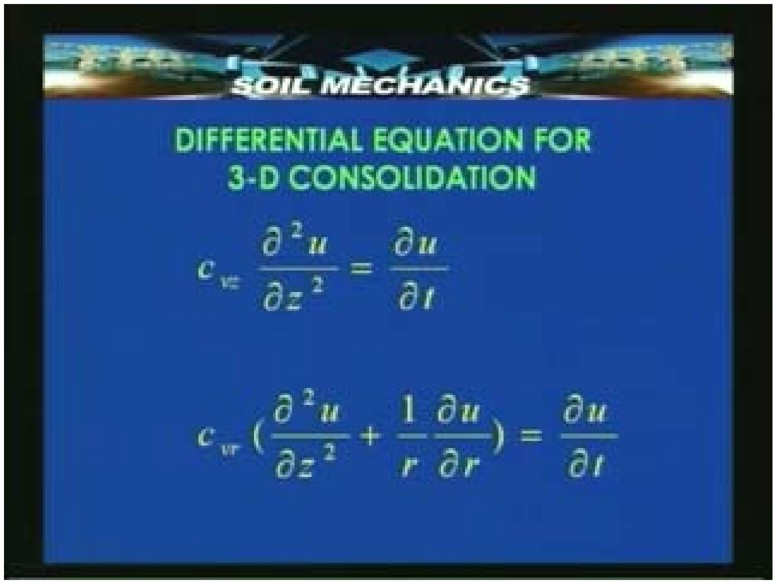

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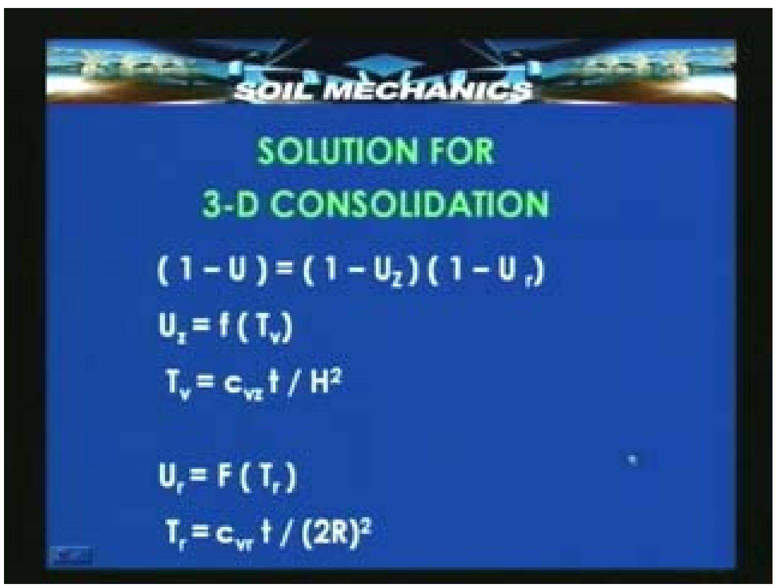

We saw that a solution has been advanced for this. That is the average degree of consolidation U which is a function of the vertical and the radial components can be expressed by this expression 1- U equal to 1- U_z into 1- U_r. So it is U_z which is a function of the conventional T or the time factor which is calculated by c_{vt} t by H square. The radial degree of consolidation U_r is now proportional to time factor for the radial consolidation T_r and by analogy this T_r can be written in a manner similar to that for T_v and it can be written as c_{vr} t divided by an equivalent of drainage path H of the vertical consolidation. Here we see that the influence zone of a cylindrical drain extends over a diameter of 2 r. So within that influence zone any point from which movement of water takes place will cover a distance equal to 2 r and that becomes therefore the drainage path.

There are two possible solutions for this U_r, one corresponds to the so called free strain conditions which is a situation that arises when you have a flexible loading at the top and another is the equal strain condition which arises when you have a rigid loading on the surface. The flexible loading case has been solved theoretically by two different scientist, mathematicians who have given solutions one in 1930 and another in 1935, the rendulic 1935 solution is quiet popular for the free strain condition that is if the pre loading embankment is flexible, 1935 rendulic solution is preferable but if the preload is rigid or even semi rigid very often we apply the barrons solutions corresponding to equal strain which was evolved sometime in 1948.

This graph shows the Barrons solution, the radial consolidation value U_r as a function of T and drainage path 2r is shown here, U_r is a function of T_r. If we know the parameters of the sand drains then we can find out from this table what is the radial consolidation corresponding to any given time factor. What is that parameter for which we can find out this degree of radial consolidation? The parameter is nothing but the radius R divided by the radius of the actual drain or W. So how many times the radius of the influence zone is as compared to the radius of the actual drain. That ratio N which is the ratio of radius capital R to the small R w serves as parameter very good parameter to define the geometry of different combinations of sand drain and for different values of N that is depending upon the size and the influence zone of the drain that is adopted, we will have a different U for a given T.

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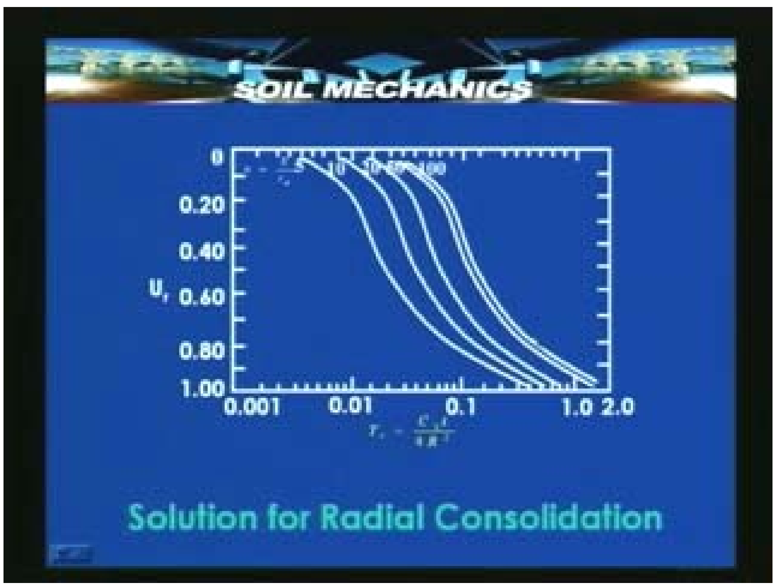

So U versus T for different N values is what we normally use in order to compute U_r. So for a given U_r we can find out a suitable arrangements of the sand drains, once we decide the layout square or triangular we can work out capital R suitably or if we have a certain arrangement in mind and certain dimensions in mind we can from this chart correspondingly find out U_r. Very often it's the design of the sand drains which is required in practice so we arrange this sand drains, their relative values of the capital R compared to the diameter of the drain itself in such a way as to achieve a desired degree of consolidation within a desired period of time. After all the very purpose of preloading and radial consolidation is to accelerate the consolidation and therefore there is no point in getting the same degrees of consolidation over long period of time. So in order to really accelerate we can define what is the degree of consolidation which we want within a given time t and find out what combination of sand drains will give you that degree of radial consolidation.

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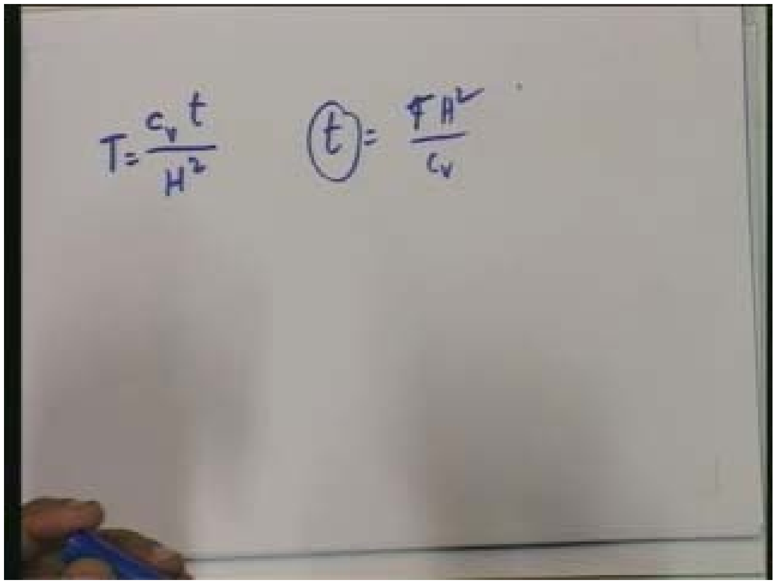

This is an interesting point which probably requires a little more elaboration. See we are having vertical consolidation corresponding to which we have a formula c_{vt} t by H square and T or t is equal to T H square by c_v. What time really important which is significant in a vertical consolidation phenomenon? In a vertical consolidation phenomenon, in primary consolidation we are not now dealing with the secondary consolidation effect. It is the T_{90} which we comfortably determine from an odometer test and very reliably too and so it is T_{90} which is important and from a practical point of view also the last 10% of the consolidation also may be treated as not very significant and the structure can be expected to withstand during its life time of that last 10% of the consolidation. Therefore it is T_{90} that is the time at which 90% of the vertical consolidation takes place which is of significant.

So in a phenomenon like this, we design the sand drains by orienting ourselves from this considerations. We decide what is the T value that we desire for or rather what is the T value that is required for 90% consolidation. This can very easily determined from a simple odometer test so if we know what is the time that is required for 90% consolidation vertical then we can find out corresponding same time, same t what is the corresponding T_r and then what is the corresponding U_r for different values of n equal to R upon r_w and then we can organize this n in such a way that we get a value of U_r which will be such that 1- U which is equal to 1- U_z into 1- U_r will be corresponding to a U value which is really significant. This is going to be point 9, 90% consolidation. So we will decide what value of U we want and workout what value of U_r we get from here from this and corresponding to this U_r what combination of the drains will give us a T_r which will give us a time t which is equal to T_{90}.

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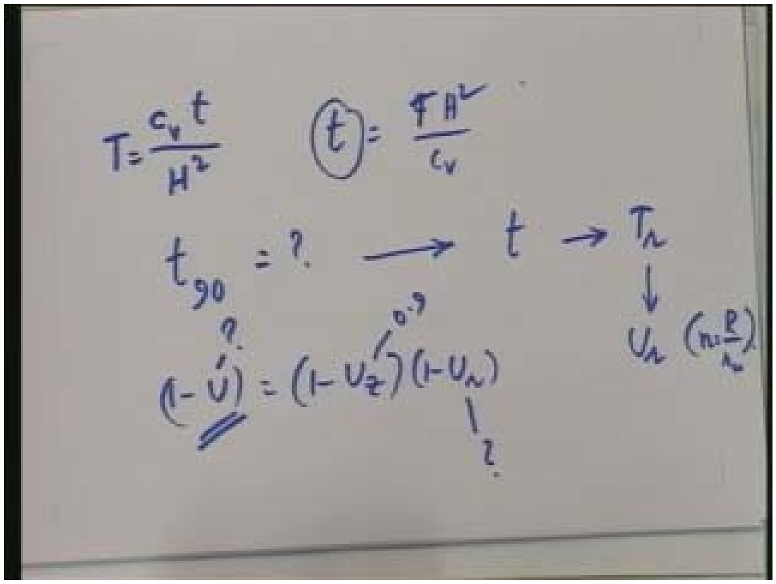

And so in practice we start with a vertical consolidation find out what’s the time for 90% consolidation then corresponding U_r and by deciding this U_r, defining this U_r we find out the layout of the sand drain that is required that’s what is known as design of the sand drains. Now let us see a few examples, we have already seen 6 different examples during the course of the previous lectures. So we shall now see these phenomena that we have covered so far through a few examples. First of all we have seen this pre consolidation pressure. It’s very important to know in practice whether a given soil is already consolidated or not in the past, so a question of this type is very common.

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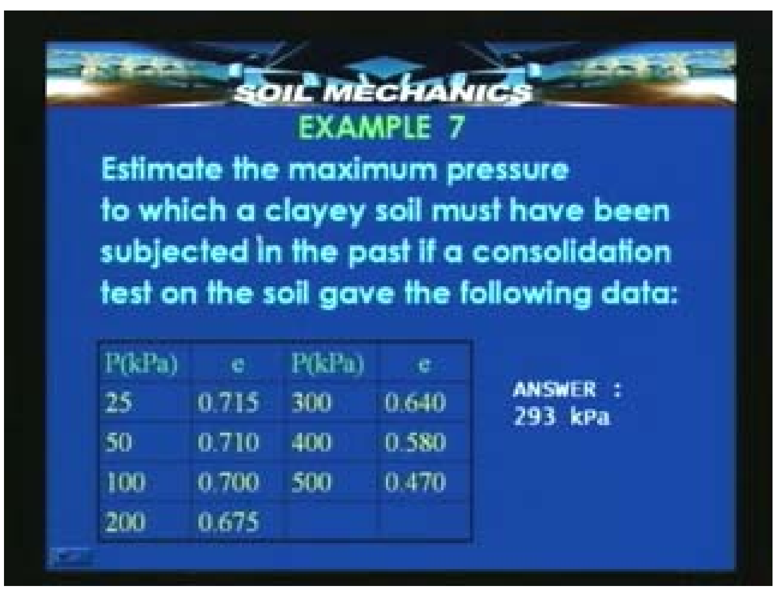

That is estimate the maximum pressure to which a clayey soil must have been subjected in the past if a consolidation test on the soil gave the following data. So a typical consolidation test has given the data corresponding to the load and void ratio. So in kilo pascal if this are the load and these are the void ratios then we can plot this load verses void ratio in the form of k e verses load curve that is e verses p curve. This e verse p curve is the one which we shall be using for locating the pre consolidation. If this curve is something like this so we have one part of it then from here this recompression part is the one which we have, then we draw a tangent we draw a horizontal at from the point of maximum curvature and then the bisector where the bisector meets these straight line portion of this graphs that is the point corresponding to pre consolidation and that's the pressure which corresponds to the pre consolidation pressure p_c.

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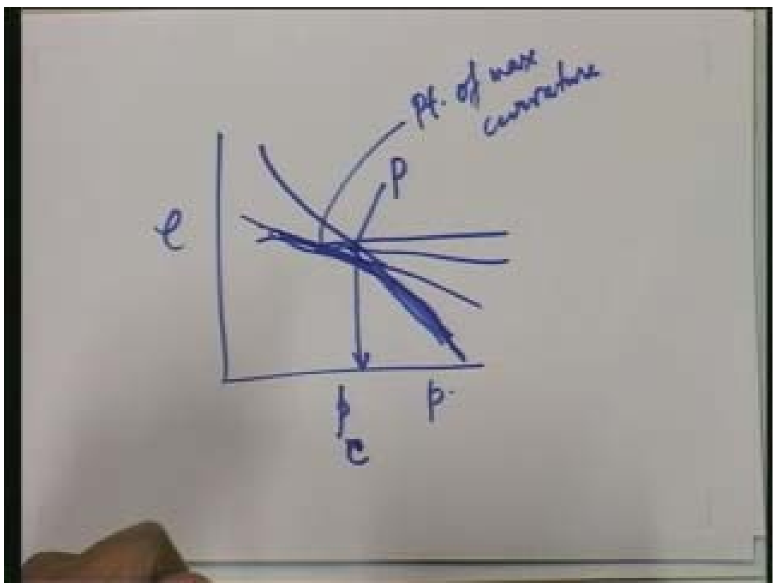

So if you plot this graph then particularly this is usually plotted to a log scale. So on a log scale if you plot this graph and estimate the pressure corresponding to this point where the bisector cuts the linear part of the curve you will get a value of 293 kilo Pascal for the pre consolidation pressure.

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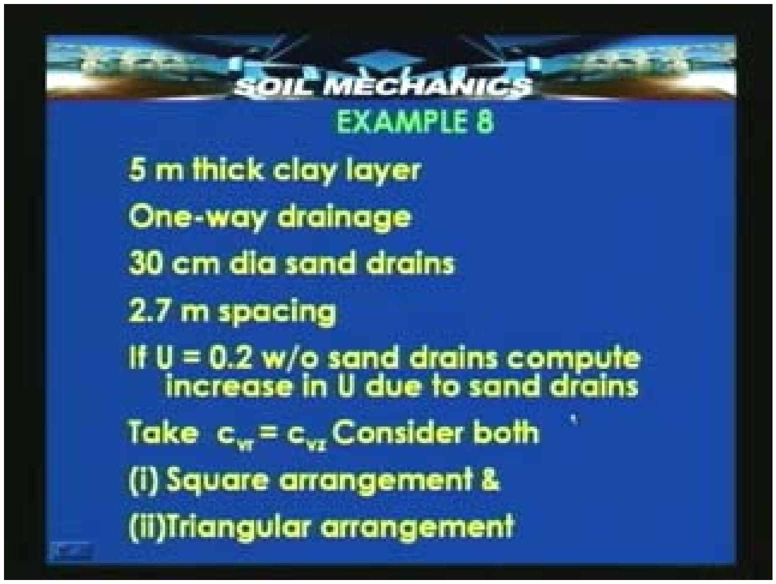

Now suppose we have a 5 meter thick clay layer which is undergoing one way drainage and we are using 30 centimeters sand drains at 2.7 meter spacing. If without sand drains if U is equal to 0.2 then what happens to this value of capital U when we use sand drains and this is going to be the subject matters for the discussion in the next lecture. So today we have covered some essentials of secondary consolidations and radial consolidations and seen an example. In the next lecture we shall see some more examples and we shall see some special points or special aspects and then close with a summary.

Thank you