Geotechnical Engineering I
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Lecture-26
Compression Characteristics of Soils - II

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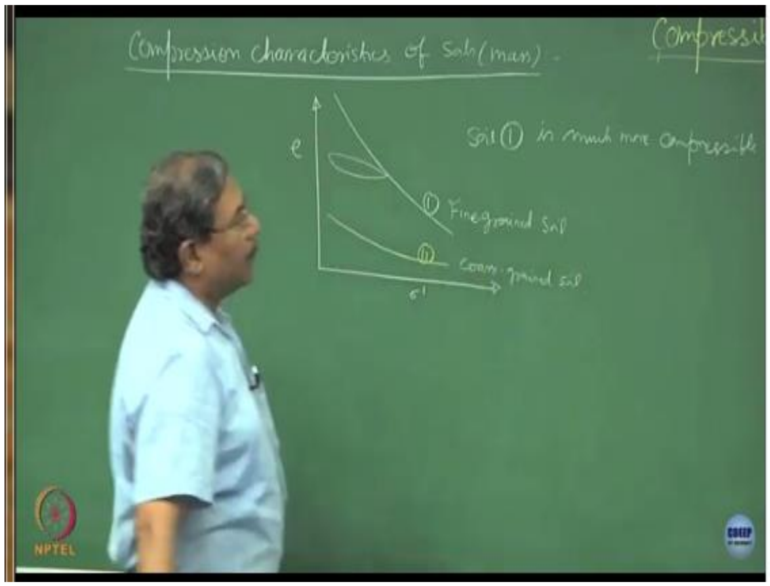

If you talk about the compression characteristics of the soil mass is e over sigma prime and suppose if I give you a curve like this and another curve like this. So it is understood that this soil 1, so I can right here that soil 1 is much more compressible as compared to soil 2. So this is how also you can characterize the soil and differentiate between them. So most probably 1 is going to be a fine grained material and the 2 is going to be a coarse grained soil.

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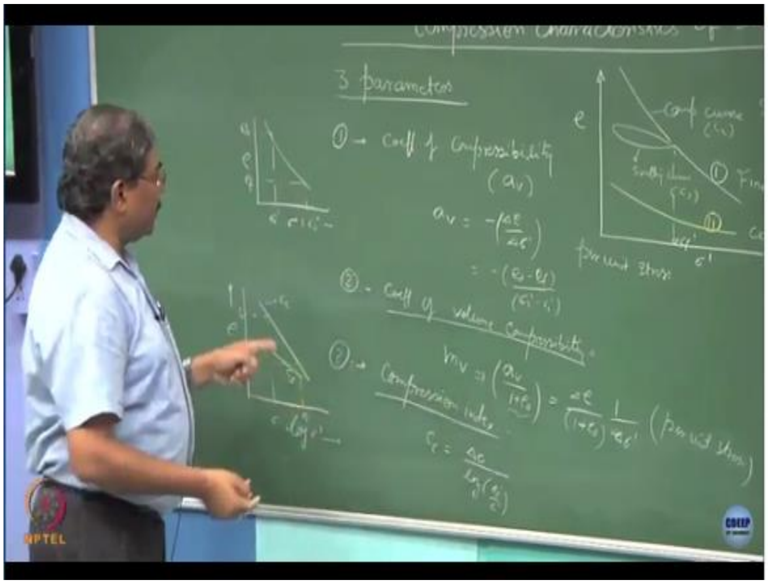

Now what we want to do is, we want to quantify the compression characteristics and these are the 3 parameters which are used to quantify the compression characteristics of the soils. The first one is known as coefficient of compressibility this is defined as a v, now if I plot e versus sigma prime and this is the compression curve, a starting from e 0 you get to e f and this is sigma 1 prime, sigma 2 prime.

So a v is defined as - delta e over delta sigma prime which is equal to - e 0 - e f upon sigma 2 prime - sigma 1 prime, what will be the dimensions of coefficient of compressibility per unit stress alright. The second one is the coefficient of volume compressibility, we define this as m v and this is equal to a v over 1 + e 0. So this is equal to delta e over 1 + e 0 into 1 upon delta sigma prime ok, 1 upon e 0 is the a specific volume.

So a v upon 1 + e 0 a v is delta e upon delta sigma prime 1 + e 0. The third term which we normally use is defined as the compressibility index or compression index. This is defined as C c and C c is, if I plot this data on a log scale. Now this curve is going to be a linear curve or a straight line this is e 0, e f sigma 1 prime, sigma 2 prime this is nothing but the slope of this curve, so the slope of this line is C c compression index.

So this is delta e over log of sigma 2 prime upon sigma 1 prime, so C c is dimensionless and m v will be again per unit stress our C c is dimensionless. So C c is utilized to characterize the soils when I said that soil 1 is much more compressible as compared to soil 2.

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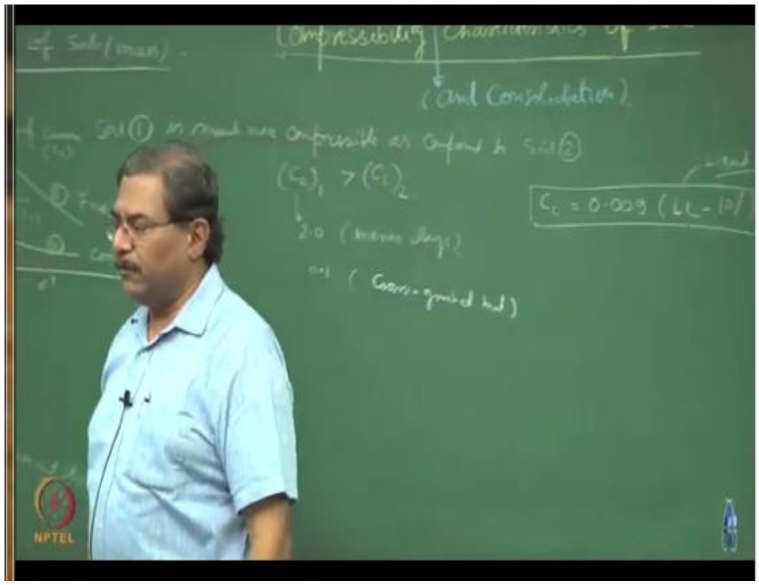

I can say that C c of 1 is higher than C c of 2 marine clays particularly, the biggest problem is that C c could be as high as 2.0. So this is one of the ways to differentiate between the soils, I can do 1 dimensional consolidation test compressibility test, I can get the C c value and if C c value comes out to be a extremely high, it is a highly compressible soil you know. And C c is extremely low let us say 0.1, 0.2.

You know this is what is going to be stiff clay sometimes or this could be sandy material. The interpretation could be these are the coarse grained materials. Soils with very high liquid limit would show a absolutely high C c value. On the same graph if I plot, so this if you remember was the compression curve and this is the swelling curve. So here if I plot the swelling curve this is how the swelling curve will look like and the slope of this will be C s, this is C c, C s and C c can be utilized to characterize the soils.

There is a empirical relationship which is given and which can be utilized for you know most of the practical work C c = 0.009 liquid limit - 10% or what 10% this is liquid limit. And this is a value of C c is a empirical relationship which was given by a Skempton in 1944. So this is normally used by the designers, if you obtain the liquid limit of the soil mass, you can use this empirical relationship get C c value for a initial design.

You know, if you are doing the finer designs is then you have to do this test and get the parameters. Now comes a question that how would you find out the sigma p prime value the pre-consolidation pressure.

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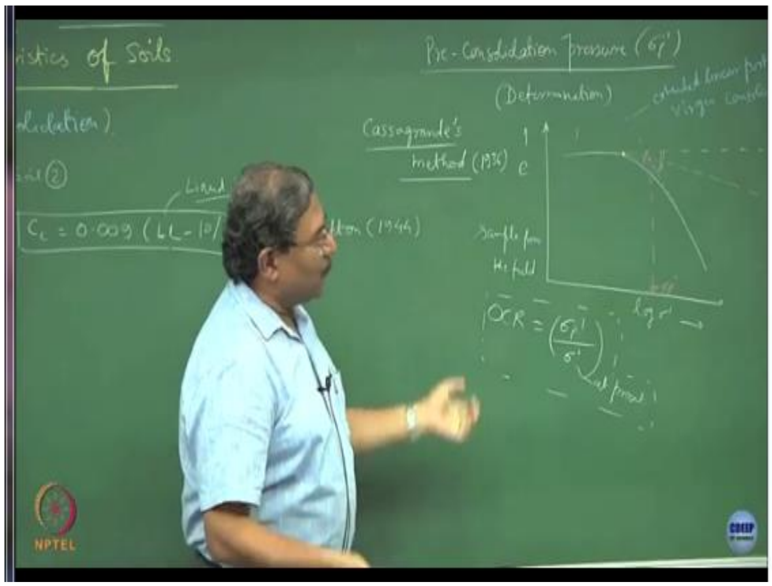

So suppose if I want to determine this, suppose if I take a sample from the field and then conduct this test on it, what I will be getting is, I will be getting a relationship like initial portion is horizontal, almost and followed by a rapid drop in the e verses lock sigma curve. So this is on, suppose if you take out a sample from the field this flat portion resembles your swelling curves or recompression curve sort of a thing.

This is because of the pre loading which system has already been expose to. So this sigma p prime is a pre consolidation pressure and the question is how to obtain this. So this method was given by Cassagrande it is a bit of geometrical construction 1936. So on this graph get the point of maximum curvature and once you have done that, draw a horizontal line, draw a tangent passing through this point.

So this is a tangent fine divide this angle alpha in equal half, so this becomes equal to this extend the virgin consolidation curve which you have obtained backwards. So this is the linear portion of extended linear portion of the virgin consolidation curve. This is the horizontal line, this is the bisector of the angle and this is a tangent the point of intersection of this extended curve with the bisector gives you sigma prime.

So find out the intersection of the extended linear portion of the virgin consolidation curve with the bisector of the angle and this point gives you sigma p prime. So what we have done is, we have got the pre consolidation pressure and hence we can redefine the OC NC behavior alright. So OCR is the ratio of pre consolidation pressure divided by the pressure acting on the system at present. So OCR is pre consolidation pressure divided by the pressure which is acting at present fine.

So what this indicates is if you can define and you can obtain like this the OCR value, you can define both ways the NC OC behavior can be defined the way I have defined earlier, OCR can be obtain like this. So this is the practical application of pre-consolidation pressure. Now, why this initial portion is horizontal because once you take out a sample from the field the sample is already under gone through the pre consolidation pressure because of the movement of the vehicles or whatever.

So we want to find out what is the pre consolidation pressure which is going to control the mechanisms of you know, the soil mass fine, any questions. Somehow these things you have to remember because these are the parameters which are utilized for characterizing the soils and this is more of a fundamental response of the material. Now if this part is clear, I will move on to the consolidation settlement and it is computation.

Most of these methods have been proposed by the person alright. So this is how they propose things this is their philosophy, we are simplifying following it. You may say that this is the point beyond which the reversal in the mechanism is occurring. So this is the pressure which is controlling or switch over between the OC and NC response also. So you must have realized that this is a very flat portion clear.

So, that means, the void ratios are not changing much with respect to implemental pressure. So this is a peculiar OC response, so if you realize here that this is a point where the sigma p prime lies. So this is the boundary between OC and NC behavior, so this is a hypothesis which is proposed by Cassagrande to obtain the sigma p prime, make sense also. So you must have realized that beyond sigma p prime the fine grained soils continue exhibiting deformation or compression or consolidation clear.

So all these soils which are of the marine origin form under the category of NC materials normally consolidated clear. The soils which have got desiccated have become stiff, they are all OC materials.

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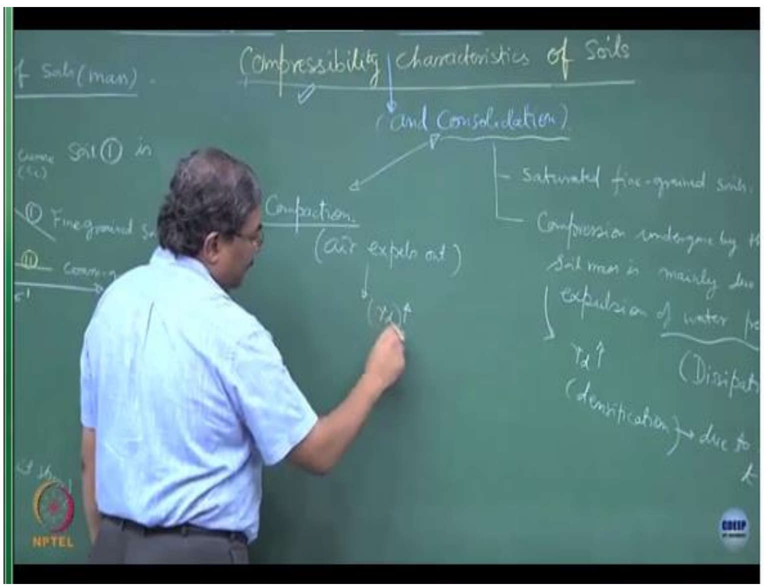

The compressibility part has been discussed now and let me start discussion on the consolidation part. So basically consolidation is a phenomena which happens in saturated soils fine grained soils and this is compression of the soils mostly because of expulsion of water alone. So compression is compression under gone by the soil, by the soil mass is mainly due to expulsion of water from the pores.

I hope you can realize that this expulsion of water from the pores is nothing but the pore water pressure which is getting dissipated. So we call this as dissipation of pore water pressure also, clear, and dissipation of pore water pressure is a function of hydraulic conductivity and the type of the soil. So what is the difference between consolidation and compaction, so in case of compaction air only oozes out alright air expels out this also leads to gamma d becoming higher correct, however in case of consolidation what is going to happen.

It is the oozing out of the pore water pressure from the pores, so this also results into gamma d becoming higher. So this also densification process and this densification is because of the sustained loading remember. So this is the densification due to sustained loading**,** sustained loading means t tends to infinity alright the place, the time when 3, 4 dial gauge readings seize to change, they become constant, this is a instantaneous process air gets expelled out, gamma d increases fine.

Normally we do not talk or do we do not associate time with compaction. Now this is the time which is getting intruded into the expulsion of water because of the permeability. So permeability is nothing but a sort of resistance offered by the soil mass or the flow of water, which is a time dependent phenomena. So, sustained loading t tending to infinity densification occurring because of oozing out of the water but soil is still remains saturated.

Because you are forcing it to by putting it in the water clear. So these are the minor differences or the major differences between compaction and consolidation process. I hope you can realize even if I compact the soil alright.

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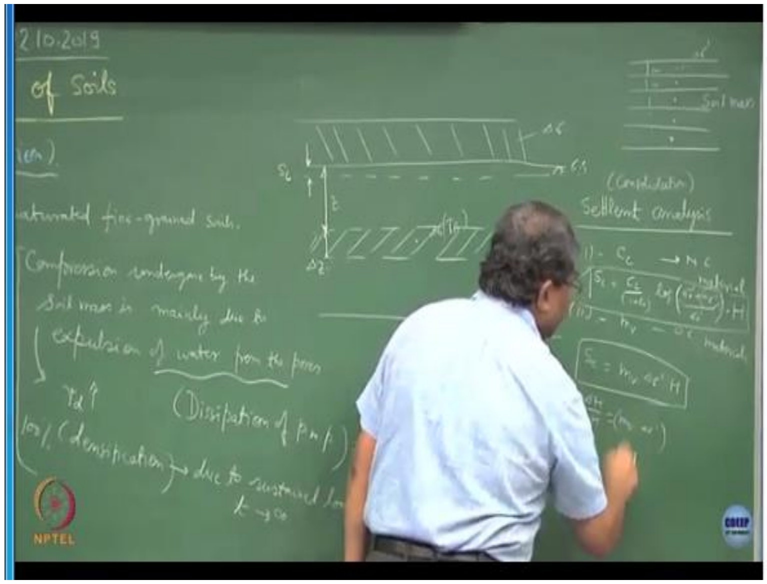

Suppose the top layer has been compacted, this is the momentary process the pad which I have created on the ground surface because of this delta sigma, what is going to happen. Now this delta sigma is going to get delegated over this point clear. So this will become delta sigma multiplied by I B ok, you know what is I B influence factor very good, the consolidation is going to get initiated because of this delta sigma over a period of time under sustained loading.

So whatever you are compacting and creating a pad, if you are not careful in designing the systems, keeping in view there the settlement response of the soil mass they are going to fail, is this part ok. So even if I take the soil and compact and create an embankment which is going to sit on a marshy marine clay type of a soil which is highly compressible, once the system is formed, initially there is no problem.

But in the due course of time what will observe is, that this whole system will start slowly and slowly settling down and most of the time these settlements are not very uniform, why. Because the thickness of the deposits might be different, soil properties might be different, pore structure might be different and so on. So what is going to happen is, the whole thing will result into differential settlements, so one portion will settle more as compared to the next portion and so on.

And hence the entire system will result in a failure. So suppose, if I consider this type of situation and if I ascribe the settlement undergone as S c clear this is a settlement of the system. And if I assume a thin layer of the clay, mostly clays will exhibit consolidation settlement at a depth of z and made up of let us say delta z, to compute how much settlement the system is going to exhibit I require 2 parameters.

So suppose if I say settlement analysis and this settlement is because of consolidation. So one of the ways of doing the settlement analysis is to utilize C c, what we have written over here, compression index clear. Normally C c is associated with NC response of the material fine marine clays with very high liquid limits because what you have observed in the formula for C c there is a liquid limit component coming to picture, however if I am dealing with these stiff clays then I require m v.

So this is normally use for OC materials you should not go for between the 2. So depending upon NC OC response the settlements can be obtain, is this part ok, have you understood this, so please never goof it up. So what we do is the entire region or the soil mass is divided in a small strips, normally a 1 meter thick ok and at the centre of these strips we compute delta sigma prime and this delta sigma prime is going to cause the settlement of the system.

So if C c is known, the S c can be written as C c over 1 + e 0 into log of sigma prime initial + delta sigma prime over sigma 0 prime, is this ok multiplied by the total height h. So if you are doing it for layers, h becomes 1 meter, C c is of each layer void ratios are initial void ratios, you know what is the initial stress acting delta sigma prime computation is very important and for this you require influence factor, is this ok.

So the stresses which are getting induced in the soils because of external loading has to be obtain C c is known, void ratios are known, S c is known settlements clear consolidation settlement of the NC material. However when we are dealing with the OC materials, the S c will be equal to m v delta sigma prime into h, how did we get this relationship s is nothing but delta h and delta h upon h = m v delta sigma prime, do you think it is alright.

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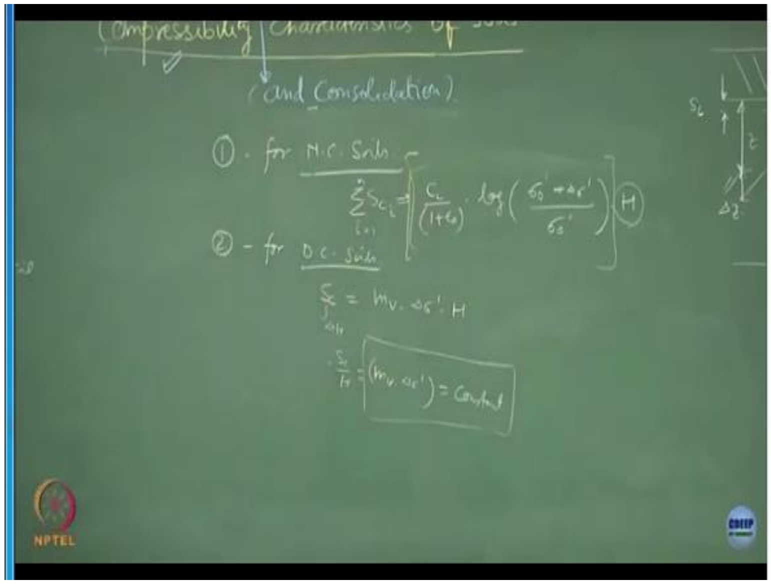

So for NC materials alright, the consolidation settlement is equal to C c divided by 1 + e 0 into log of sigma 0 prime + delta sigma prime divided by sigma 0 prime into H, H is the thickness of the clay layer. Sigma 0 prime is the initial pressure at the point o delta sigma is the incremental pressure because of external loading which is causing settlement to occur. C c is the compressibility index, e 0 is the initial void ratio, number 2 is your m v case.

So here the consolidation settlement will be equal to m v delta sigma prime H and this is what we were discussing delta S c is nothing but delta H. So, S c upon H will be equal to m v into delta sigma prime, now this term remains constant, clear. So m v into delta sigma prime term remains constant and this is a material property. Now what you have realize is, the way we have defined m v, see truly speaking, a v is a function of the stress increment.

So, what happens to a v, as sigma prime increases a v keeps on decreasing clear, what happens to m v as sigma prime increases m v also keeps on decreasing clear. Now C c is the linearization of the graph within a certain range of delta sigma prime fine. So what we have obtained from here is m v into delta sigma prime remains constant. And this constant is nothing but a sort of a volumetric strain. For OC materials m v remains constant, for NC material C c remains constant in finite thickness of the layers.

And hence we have done discritization of the entire soil mass these are infinitesimally thin layers for which we have obtained C c e 0 and H and delta sigma prime, is this part clear. So what we have done is, we have now quantified how much consolidation settlements the system will undergo when it behaves the soils behave like NC and OC for an incremental pressure of delta sigma prime, incremental pressure is important to remember.

On their own if delta sigma is not applied consolidation settlement will not occur. So if delta sigma is 0 what is going to happen, the consolidation settlement is insignificant, how would settlement depends upon H height. So you must have noticed, what we have done, this we have multiplied this factor with H is it not. So if H is going to be more very thick deposit of the soil, you should be expecting very high consolidation settlements for the given properties, if H is a small consolidation settlement will be small.

So when we do this type of analysis what we do is, we keep on summing up all this settlements which are going to come from different layers. So here I can write, this is S c i and then I can sum it up from i = 1 to n fine, so this becomes the total settlement of the system. So there is something known as spring analogy for consolidation which was given by Terzaghi proposed by Karl Terzaghi.

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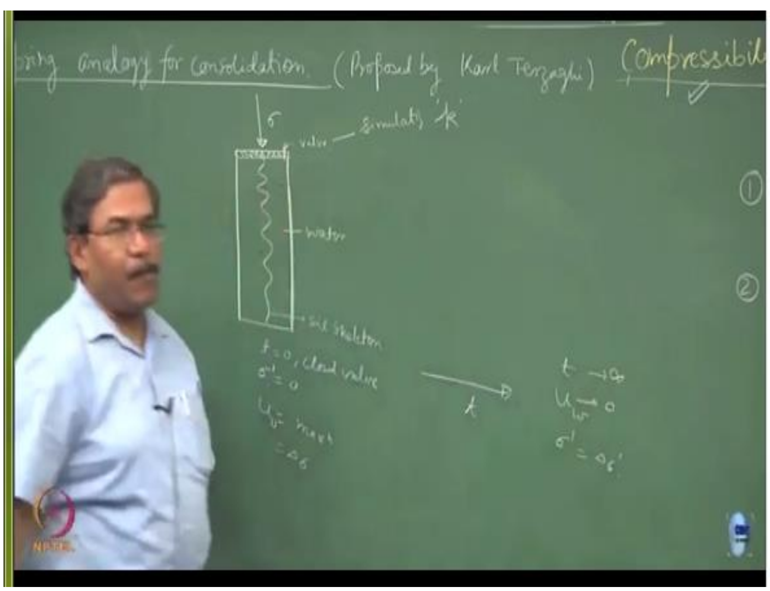

Now, what it indicates is, this is based on the assumption that if I take a, you know cylindrical container and the walls are rigid. And if I fix a piston into it and if I leave a wall over here and if I apply a load which is giving me sigma, if this container contains water which is the pore fluid in the soils, there is going to be a spring which depicts the skeleton of the soils alright. So at t = 0, I hope you can realize that if I keep this wall closed and this wall actually is simulating the k hydraulic conductivity of the soil mass.

So for very low hydraulic conductivity materials like clays wall remains practically closed. The moment you apply stress, no pore water pressure is going to get dissipated so soon because of the low hydraulic conductivities, what is going to happen. The pore water pressure is going to be maximum is it not and this will be equal to delta sigma which you have applied. Because the entire pressure is being taken up by the water, so sigma prime will be equal to 0.

Now as time passes by what happens and if I open the wall that means if I allow dissipation of pore water pressures, this piston will move further, spring will get shrunk, water being incompressible will come out, so what is going to happen. The pore water pressure will tend to 0 at t tending to infinity, now when pore water pressure tends to 0 the sigma prime will be equal to delta sigma prime.

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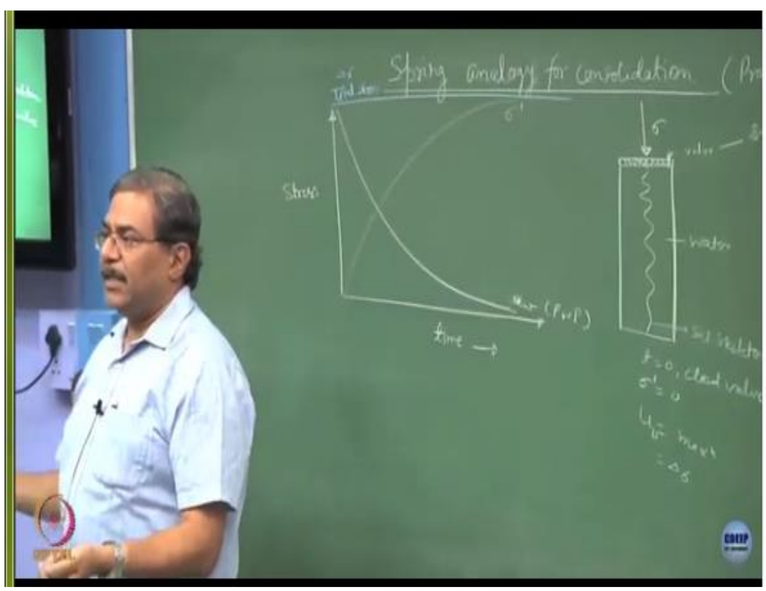

So the way Terzaghi has proposed this, if I plot stress on the y axis and time on the x axis, I hope you will realize this is the first time we are talking about any property which is a function of time in geotechnical engineering clear. And this time is representative of the hydraulic conductivity of the material. So, if I plot let us say pore water pressure, I hope you can realize the pore water pressures being maximum delta sigma with a period of time it will tend to 0 clear.

So this is the variation of the u w pore water pressure, is this ok, the release of pore water pressure because of application of external stresses which is hydraulic conductivity dependent over a period of time is the consolidation process. So the pore water pressure starting from a very high value they tend to 0, what happens to the effective stress. The effective stress initially was 0, as time goes to infinity, pore water pressure decreases this will become like this.

So this is your effective stress and the summation of the 2 is nothing but the total stress which remains constant, so this is let us say delta sigma, is this part clear. So what is spring analogy for consolidation says is, that the soil must can be considered to be a system containing water and the spring when you load it, the entire load is taken up by the, you know water, you cannot remove it.

So the pore water pressures go maximum, the moment you load any system, the entire net pressure gets transmitted to the pore water. So effective stresses are 0 immediately, very difficult situation I hope you can understand. So criticality is when sudden loading is done I hope you can realize because suddenly the blood pressure shoots up, pore water pressure shoots up and the effective stress becomes 0 slowly and slowly system becomes stable.

So what I have to do is, I have to do something to negate this type of a situation. And that is why people preload the soils and pre treat the soils to make them stable, is this part ok. Now this theory was given by Karl Terzaghi and there are few assumptions which I will write down, so that you can understand better.

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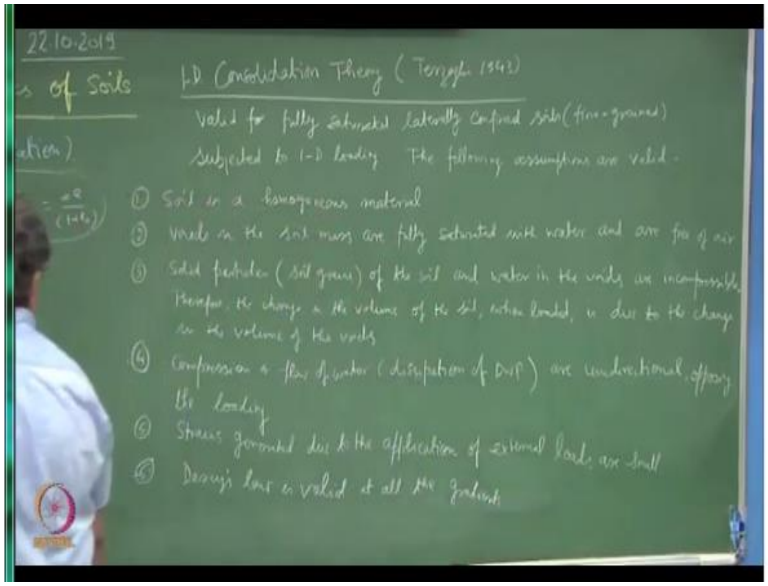

So Terzaghi theory of consolidation we should call it as 1 dimensional consolidation theory proposed by Terzaghi 1943 is valid for fully saturated laterally confined soils particularly fine grained soils subjected to 1 dimensional loading. Now if this is a situation the following are the assumptions, number 1 soil is a homogeneous material, number 2 voids in the soil mass or fully saturated with water and are free of air, number 3 solid particles of the soil.

Solid particles are nothing but the soil grains of the soil and water in the voids are incompressible therefore the change in the volume of the soil when loaded is due to the change in the volume of the voids. Number 4 the compression and flow of water, now flow of water is basically dissipation of the pore water pressures are unidirectional opposite to the direction of the application of the load or the stress are unidirectional opposing the loading alright.

Point number 5, strains generated due to the application of external loading are small why because we have still assuming that delta h upon h is the strength.

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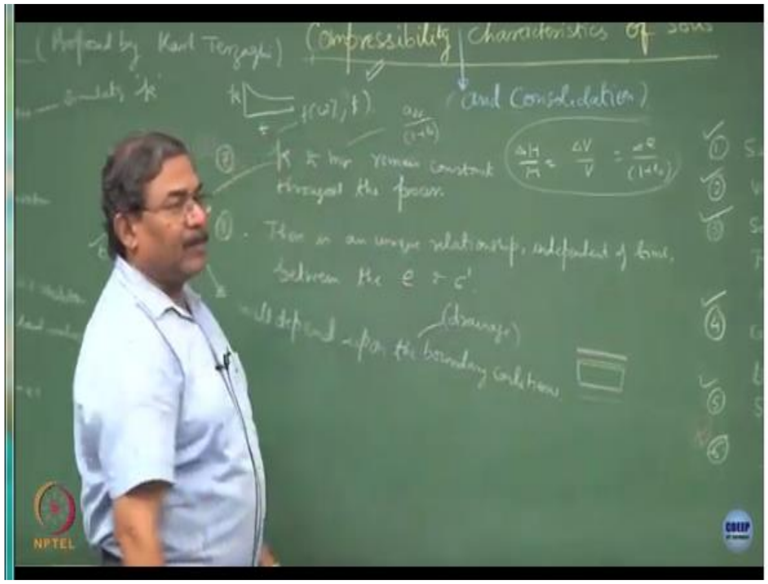

You know this is the assumption which we have made this is equal to delta v upon v. So this relationship is valid when the deformations are small this is a stain term and this we have related as delta e over 1 + e 0 clear. So this is valid when a strains are a small and hence that is the reason why in the previous case we have discretize the entire soil mass in a small, small strips. So suppose 20 meter, take soil mass we have discretizing layers of 1 meter clear.

So that this condition does not get violated, you were asking this question. Number 6 Darcy's law is valid at all the gradients. Number 7, the hydraulic conductivity and m v remain constant throughout the process. Point number 8, there is a unique relationship independent of time between the void ratio and effective stress. So these are the assumptions which have been imbibed into the 1 dimensional consolidation theory which is given by Terzaghi.

I hope you can realize that is fair enough to assume the soil in the homogeneous material no issues. Voids in the soil mass of fully saturated and divide of free of air we are immersing the entire specimen in the water bath, so this can be achieved, no issues these 2 are ok. Solid particles of the soil and water and the voids are incompressible therefore the change in the volume of soil when loaded is due to the change in the volume of the voids it is fine.

Compression and flow of water are unidirectional opposing the loading is ok, a strain generated due to the application are stain loads as small is fine, there is no problem I can deal with it my discretizing a small, small layers. Darcy's law is valid at all the gradients, what is your understanding this is not correct truly speaking. Because for very small gradients Darcy’s law might not be valid.

So this is the one violation in real life, hydraulic conductivity and m v remain constant throughout the process is also not correct, it is over simplification why. Because k, you know is a function of moisture content as well as k is the function of time. When we were doing constant head test, I think I talked about this that is for a fix time duration, we take 3 readings for the drop of head. If time intervals are very high, then your hydraulic conductive the bound to change.

Otherwise also, if you plot k versus t, what you will observe is that hydraulic conductivity decreases with time, particularly when the times are of the order of months and years. There could be the changes at the pore level, which might occur, depending upon the type of contaminants which you might be dealing with, the k might increase with respect to type bacterial activity k might change with time.

So it is not a good idea to assume k remains constant fine m v, m v also does not remain constant just now we have seen. So m v is a v over 1 + e 0 and a v is delta e upon delta sigma. So truly speaking m v is a term which depends upon the delta sigma, so as delta sigma increases, m v value decreases. So this is a violation of the Terzaghi theory in real life, there is a unique relationship independent of time between the e and sigma prime is also not correct, truly speaking.

Because if you remember when we were talking about the e versus sigma relationship, I said that the third scale should be tiny scale. Because ultimately what we have done, the entire process we are super imposing on time scale clear. So for the simplicity sake Terzaghi assume that the void ratio and sigma prime are related to each other. And there is no time dependence in these processes in these parameters.

There is another issue k will depend upon the boundary conditions and these boundary conditions are the drainage boundary conditions. This I will be discussing much more in details in the next lecture. So I hope you must have realized when we were discussing today's consolidation test what we discuss in the class, I was intentionally I have put 2 porous stones on the top and bottom of the specimen.

So that means I am forcing the drainage of pore water pressure to take place in this porous stone and in this porous stone, as if there is a line of symmetry along this point. So because of the application of stresses, whatever pore water pressure develops, it gets dissipated from this porous stone as well as from this porous stone about a line of symmetry. So k will also depend upon the boundary conditions, fine.

So I have discussed a lot of things today about the compressibility behavior of soils and I also talked about the consolidation characteristics and I have introduced the concept of 1 dimensional consolidation theory which is proposed by Terzaghi.