Finite Element Analysis and Constitutive Modelling in Geomechanics
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Lecture – 42
Soil Behaviour and an Introduction to the Existing Soil Models

So, today, in this course namely Finite Element Methods and Constitutive of Modelling in Geomechanics. So, we will have a discussion on the Soil Behaviour and an Introduction to the Existing Soil Models. So, I am Dr. J. Jayapal a former student of Professor Dr K. Raj Gopal from Indian Institute of Technology Madras.

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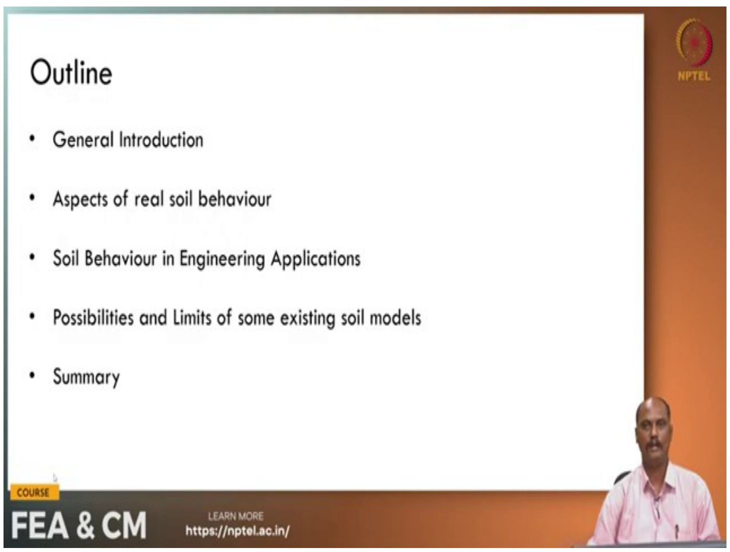

So, the outline of my lecture will be general introduction, aspects of real soil behaviour, soil behaviour in engineering applications and then possibilities and limits of some existing soil models and then final summary and recapitulation.

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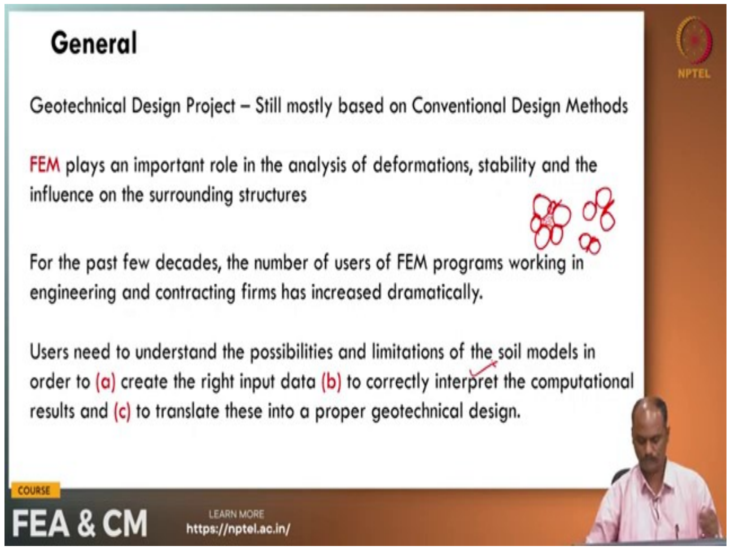

So, when it comes to the general idea, so, even till today we are having the geotechnical design project still based on conventional design methods. So, this being a very this being a very major point here in this particular lecture. Why we have to go for a finite element method when it comes to geotechnical engineering? And as we all know, in the present day scenario, finite method plays an important role in the analysis of deformation problems, stability problems.

And most importantly, the influence of the same on the surrounding structures. So, before getting into this finite element very recently if you can observe soil being a three phase material. We have solid grains and then the water inside the pore fluid can be either water or air. So, we need to model this soil grains, this pore fluid namely water or any other flow fluid or air, sometimes being a three phase material.

And we all know that the forces that are coming from the superstructure they are transmitted to the soil grains through the frictional contacts. I mean at these points and then the pore water pressure is nothing but the pressure that is created by the water that is present there in the pores. So, all these three components, all these components have to be modelled in the finite element simulations to get a better picture or a good picture of the real soil behaviour.

If you see that in the present day scenario, we have discrete element modelling, wherein we model the soil on a particle by particle basis or all these particular particle contacts are being modelled. And the problem is yes, we can model, it is not a problem. But the time involved in the modelling of these particle contacts and then the size of the model is a matter of fact. In fact because of this, we actually move to the more simplistic continuum mechanics principle that is the finite methods.

So, the very few decades very past few decades there are a number of users of Finite element programs working in the engineering and contracting firms. And this has increased very dramatically. And if you, if you see in the present scenario, we can say that mostly almost all projects, almost all field projects which are massive, especially in the case of metro rails, excavation type of problems, deformation problems, stability problems.

So, this kind of finite element modelling or finite element simulations has become unavoidable. And most importantly, if you see the users need to understand the possibilities and then the limitations of the soil models in order to have three main aspects number 1 to create a right input soil data and then to correctly interpret the computational results. The correct interpretation is very, very important.

So, this correct interpretation is very, very important and finally, to translate these into proper geotechnical design is a further challenge. So, as a geotechnical engineer, it is a bit complex to model the soil, material or the geologic material or I would say that among all the geological materials among all the construction materials, it is a bit difficult to model the soils. Why? Because it is a three phase material with solid grains, air or water pore fluid.

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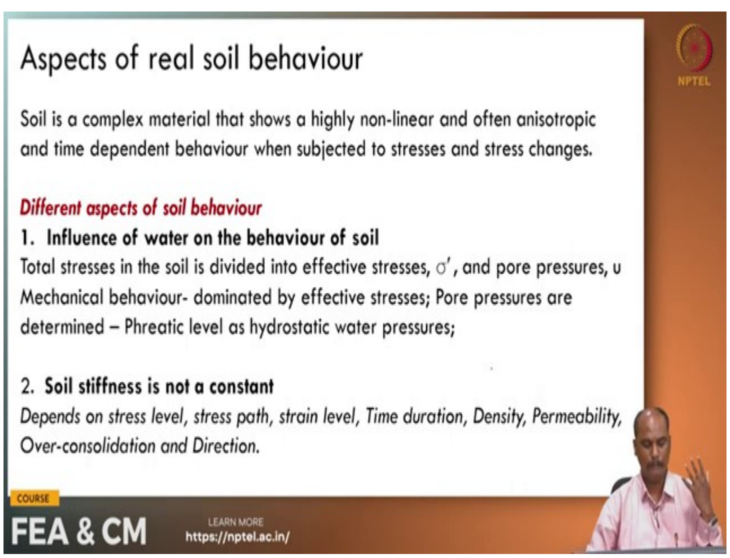

So, stepping into the real aspects of soil behaviour aspects of real soil behaviour soil is basically a complex material that shows a highly non-linear and often anisotropic and time dependent behaviour when subjected to stresses and stress changes. So, usually these stresses keep changing day by day or after t = 0 or t = 1 or t = n. The stresses are not all constant, they are changing, they are changing you know with time.

So, the response of the soils will solve will also change with respect to time. So, we will just discuss the different aspects of real soil behaviour on a point by point basis, very firstly the influence of water on the behaviour of soil. We all know that we have the total stresses the effective stresses and the pore water pressure and we all as geotechnical engineers are as civil engineers. We know the importance of the computation of effective stresses.

In order to compute the effective stresses we need to understand or we need to capture the pore pressure behaviour in the soils. So, total stresses in the soil are basically divided into effective, stresses sigma dash and then the pore water pressures U. And then the mechanical behaviour is usually dominated by the effective stresses. So and then the pore water pressures are determined with the phreatic level a hydrostatic water pressures.

And if they are not at the phreatic level then we have the flow of ground water. And most importantly, if you observe the soil, stiffness is not at all a constant. So, this depends on the stress level, stress path, strain level, time duration, density, permeability, over-consolidation and most importantly, the direction. So, as I said before, the stress level keeps changing with respect to time and hence the soil stiffness also changes with respect to time.

And then it comes to stress path this is also a very important parameter wherein the soil stiffness is being influenced. And then the amount of strain the soil undergoes, especially if we speak about the engineering strain levels that is between close to 0.5 percent or usually, the range is between 0.01 percentage to 1 percentage when it comes to the strain levels that are attributed to geotechnical structures.

And then the density if you see the density of the soil is more influenced by the external factors. And then again, if you see that the style stiffness is again related to the density of the soils and then the permeability of the soils. When it comes to the direction, I would say that we all know what is anisotropic or isotropic. So, the way in which soil is getting deposited also is going to influence the stiffness behaviour.

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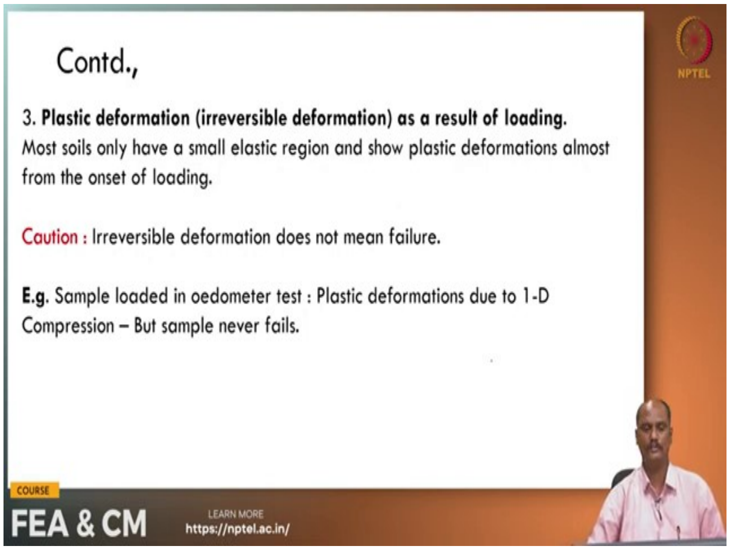

So, the third important thing is I would say that the plastic deformation or irreversible deformation. So, all that a geotechnical engineer is interested is about to have the deformations to be on the elastic side. But we cannot avoid but we cannot avoid plastic deformations as we can see that as a result of loading, mostly after a small elastic region, soils undergo plastic deformations almost from the onset of loading.

And the important point is irreversible deformation or that is called as your plastic deformation does not mean that the soil has failed but it has to be under certain limits. So, for example, if you take a our simple consolidation test, what we do in our with our odometer? The sample is loaded in the odometer test plastic deformation sucker due to 1-D loading around decompression but the sample is never going to fail.

But the sample never fails but we have that plastic deformations so, this has to be properly understood when it comes to the behaviour of soils.

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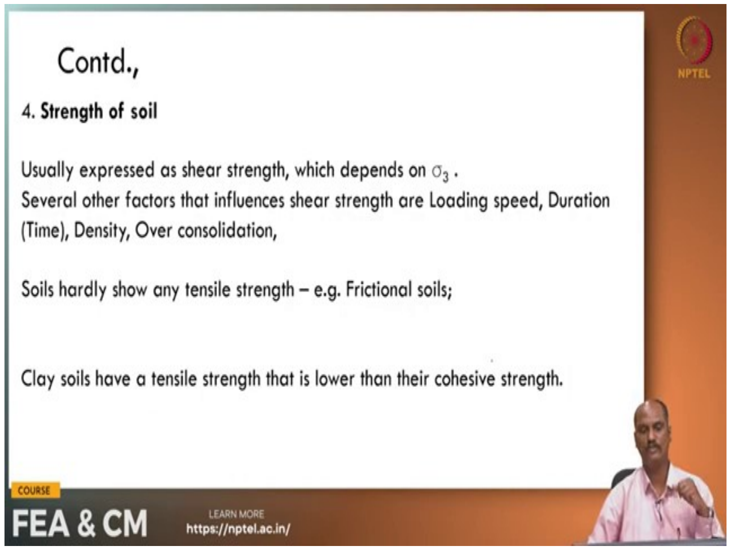

When it comes to the strength of soils usually the strength of the soil means that the shear strength of soils. The shear strength which is mainly dependent upon the confining stress levels what we call as sigma 3 is a very, very important point to note. So, several other factors that influences shear strength or loading speed. How quickly the soil is getting loaded?

And the duration at which the load is prevailing and then the density and over consolidation or OC behaviour. And we can see that since hardly we have any tensile strength with respect to soils. The tensile strength is mostly associated with the clays and not with sandy soils or frictional soils. So, clays do have some tensile strength but they are pretty much lower on compared to that of the cohesive strength.

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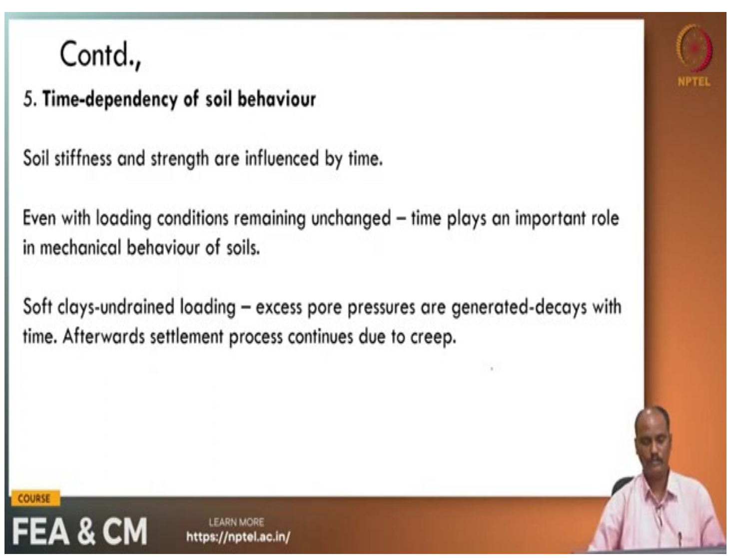

When it comes to the time dependence of soil behaviour, the soil stiffness and strength are mostly influenced by time, even with loading conditions remaining unchanged. The time plays an important role in the mechanical behaviour of soils. If you can notice that especially in the case of soft clays or undrained loading, especially in the case of excess products when they are getting generated, it decays with time.

Afterwards the settlement process continues due to creep. So, we have to understand the time dependency of soils or the nature of soils to correctly capture the behaviour of the soil in the form of modelling.

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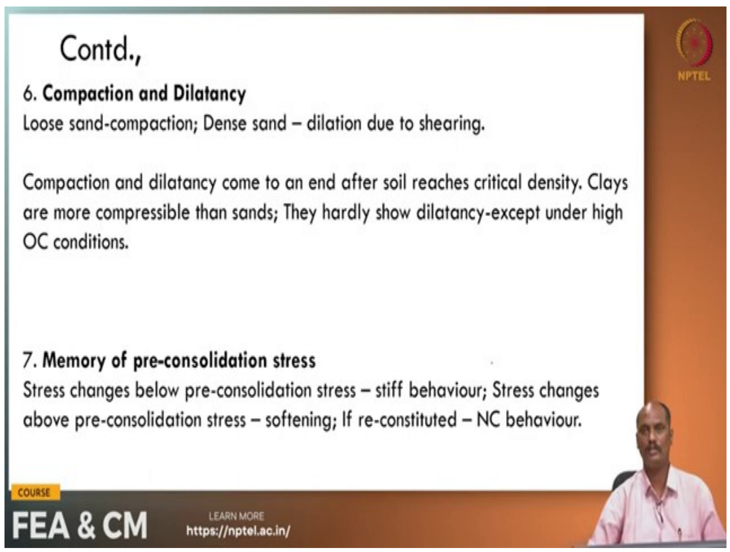

The next important factor is compaction and dilatancy when it comes to compaction or dilatancy the mostly the botheration is about the sandy soils or frictional soils. When it comes to loose sand the undergo compaction. When it comes to dense sand they dilate due to shearing. And then compaction and dilatancy they come to an end at a critical density or at a critical state.

So, Clays are more compressible in compared to that sands. And then they hardly show dilatancy-except under highly over consolidated conditions. The next factor is the memory of pre-consolidation stress, so, stress changes below pre-consolidation stress or the stress that the soil remembers. We have the stiff behaviour and stress changes above pre-consolidation stress we have the softening behaviour.

We will see all these kind of behaviour in the subsequent slides. And what happens if the soil is reconstituted is that the memory of the soil is getting lost or the pre-consolidation stress is being erased? Say, for example, in the case of normally consolidated clays if the soil is reconstituted with 1.5 times the liquid limit. Usually, the pre-consolidation stress or the memory is the memory of the soil is being lost. So, we can reconstitute it to rework on it.

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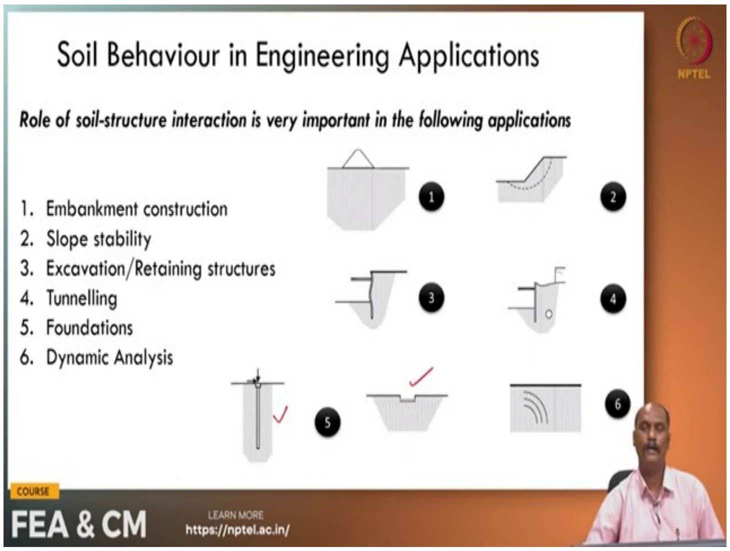

So, when it comes to engineering applications, we have a wide range of applications that are available. So and most importantly, if you see that the role of soil structure interaction is very, very important in all these applications. We will see one by one, number 1, say, for example, if you have embankment construction. We have the stage wise loading that means layer by layer compaction on a stage based construction or consolidation basis we build it.

And then the second application is being called as the slope stability. We have natural slopes, man-made slope and I would say that slope instability is very, very important. And then the third one is excavation and retaining type of structures. So, when it comes to metro rails constructions, this excavation is inevitable. So, you have to excavate at deeper depths, provide proper bracings and build suitable, retaining structures to retain the soils.

And when it comes to tunneling, again metro rails they go in tunnels, especially below the soils and this is also a challenging job when it comes to a geotechnical engineering profession in the field. And point number 5 is foundations we have shallow and deep foundations here. If I could notice, we have the pile foundations and then the strip footings or the footings here and then the dynamic analysis.

So, dynamic analysis is also a very important topic to discuss but this particular lecture will not focus more on the dynamic part. And this is beyond the scope of this lecture and you may get another speaker who would speak much more on the dynamic analysis part.

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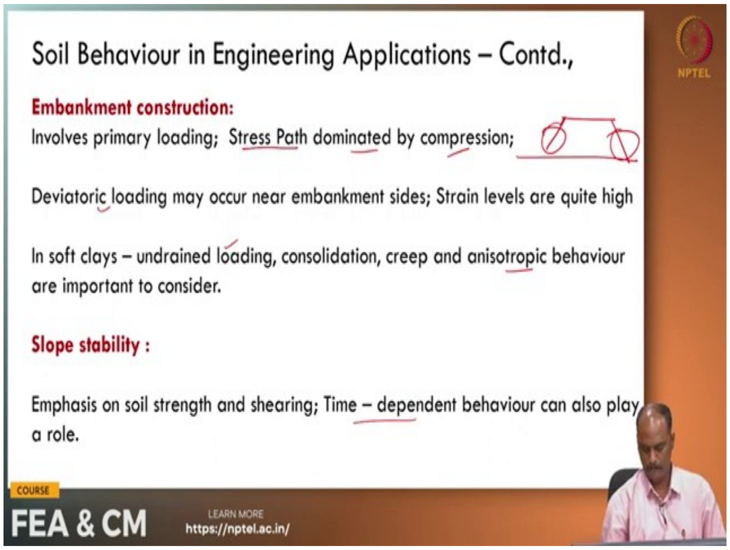

So, when it comes to embankment constructions, we have the primary loading and then we can see that the stress path is dominated by compression, usually the compression. And the deviatoric loading when it comes this may occur near embankment sides and this part. And then the strain levels are quite high when it comes to embankment. And especially in the case of soft clays we have this undrained loading, consolidation, creep and then the anisotropic behaviour that are important to consider.

So, we all know what is consolidation and then creep, we are aware of it. And then the anisotropic behaviour, especially with respect to soft clays or even peat, are very, very important to consider when it comes to the construction of embankments. And the next aspect is the slope stability aspect. So, the emphasis on the soil strength and shearing is very, very important or more concentrated.

And then the time dependent behaviour is also plays a vital role when it comes to slope, stability.

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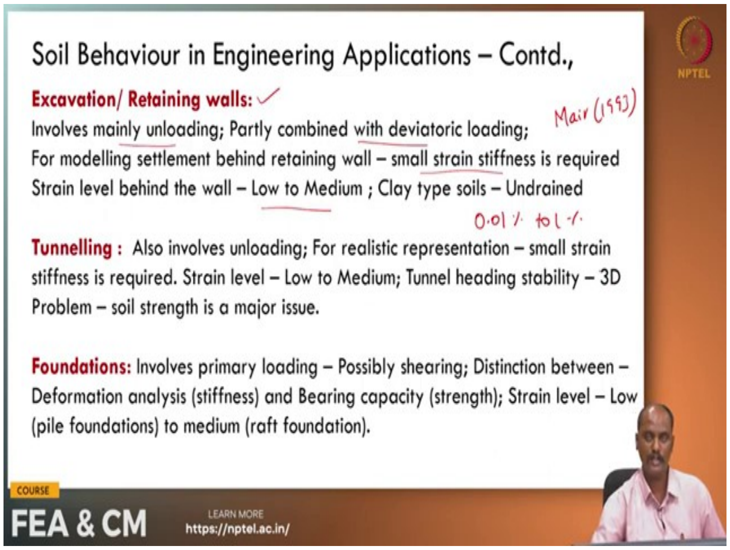

Soil, behaviour and engineering applications if we continue with the respect to excavations as well as retaining walls. So, these problems are bit unique. Why unique in the sense? This mainly involves unloading, whereas the previous cases the main point is loading problems, whereas here in this particular case we have this unloading problems and partly combined with deviatoric loading.

So, for modelling, the settlement behind the retaining walls or excavation small strain stiffness is required that has to be noted down. And then the strain level behind the wall could be either between low to medium, as I would like to recall. Usually the range of strain levels that we encounter in the case of geotechnical structures is between 0.01 percentage to 1 percentage.

You can have this strain levels being noted from the works of Mair 1993 you can just refer the paper. And then when it comes to tunneling, this is again a unloading type of problem wherein for realistic representation we need small strength stiffness. The strain level is going to be low to medium and then the tunnel heading stability is very, very important to be considered. And usually, we go for a 3D type of simulation or a 3D problem.

And soil strength is a very major issue when it comes to tunneling applications. So, when it comes to foundations, I would say that involves primary loading and then possibly shearing also and there should be a distinction between the deformation analysis and then the bearing capacity analysis. So, the former that is the deformation analysis, is concerned about the stiffness of the soil, whereas in the bearing capacity analysis is more concerned towards the strength.

And when it comes to strain levels, we have low to medium strain levels. For low we have the pile foundations and then for medium we have this raft foundations.

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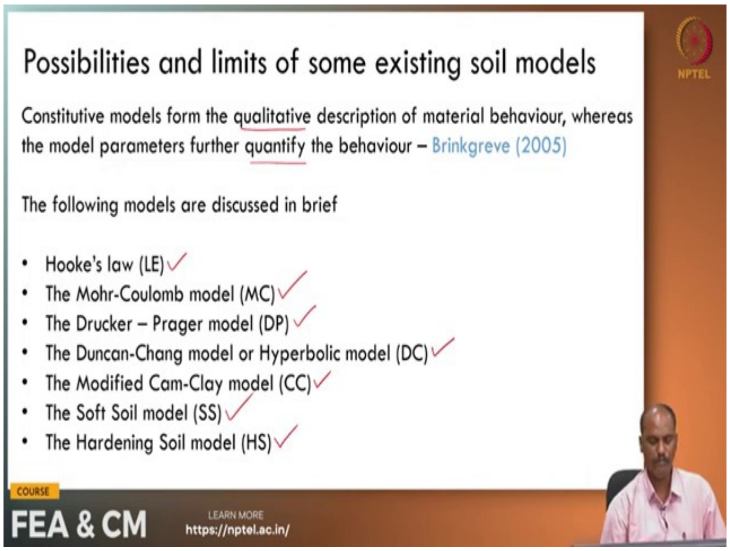

Stepping on to the next topic that is the important topic on the possibilities and limits of some existing soil models. So, there are some lists of soil models which I have indicated in the PowerPoint slides. So, these constitutive models or constitutive relations or stress trend relations form a qualitative description of material behaviour of the soil behaviour, whereas when it comes to model parameters, the further quantify the behaviour.

So, this is a very, very important point to note down the words, namely qualitative and quantitative. So, no soil model can exactly capture the real-time soil behaviour. It is actually a challenge, in fact but the major aspects of soil behaviour. The major stress strain relations between in the soil behaviour is being captured by a variety of soil models. We would just start the discussion from the Hooke's law based on the limit equilibrium analysis.

The most famous linear, elastic, perfectly plastic, Mohr-Coulomb model, the Drucker-Prager model DP model, the Duncan-Chang model or the hyperbolic model. Then when it comes to soft clays, we have this modified Cam-Clay type of model from UK and then the soft soil model SS model. And then the hardening soil model are called as your HS model.

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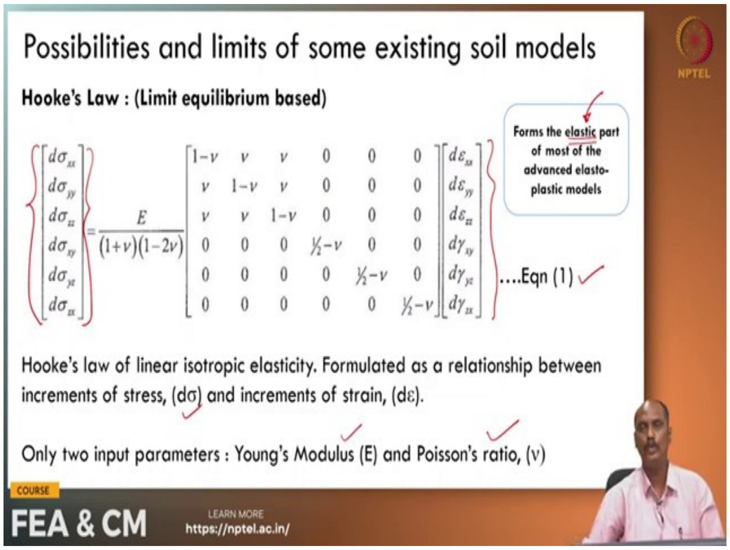

So, if you can see here, the Hooke's Law said to be the fundamental, the most fundamental relation which people developed which Hook has actually developed centuries back. When we have the stress strained relations exhibited as d sigma and d epsilon. And we have this matrix, has shown in equation 1. Here we can see that the Hooke's Law of linear isotropic elasticity this has been formulated as a linear relationship or a relationship between the increment of stress d sigma here.

Hooke's Law : (Limit equilibrium based)

$$\begin{bmatrix} d\sigma_{xx} \\ d\sigma_{yy} \\ d\sigma_{zz} \\ d\sigma_{xy} \\ d\sigma_{yz} \\ d\sigma_{zx} \end{bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & 1-\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2}-\nu & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{2}-\nu & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{2}-\nu \end{bmatrix} \begin{bmatrix} d\varepsilon_{xx} \\ d\varepsilon_{yy} \\ d\varepsilon_{zz} \\ d\gamma_{xy} \\ d\gamma_{yz} \\ d\gamma_{zx} \end{bmatrix} \quad \text{....Eqn (1)}$$

Forms the elastic part of most of the advanced elasto-plastic models

Hooke's law of linear isotropic elasticity. Formulated as a relationship between increments of stress, (dσ) and increments of strain, (dε).

And then the increments of strain d epsilon along three directions x, y and z and then we have only two input parameters, namely the Young's Modulus E and then the Poisson's ratio nu. So, this is basically the simplest of all the existing relationship I would say constitutive relationship. And then this forms the elastic part of most of the advanced elasto-plastic models.

If you see that most of the advanced elasto-plastic models the elastic part of the representation is based on the Hooke's law.

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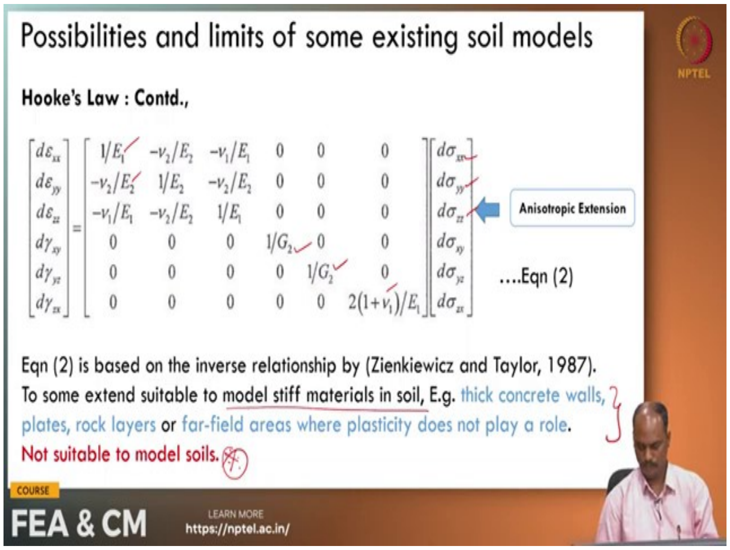
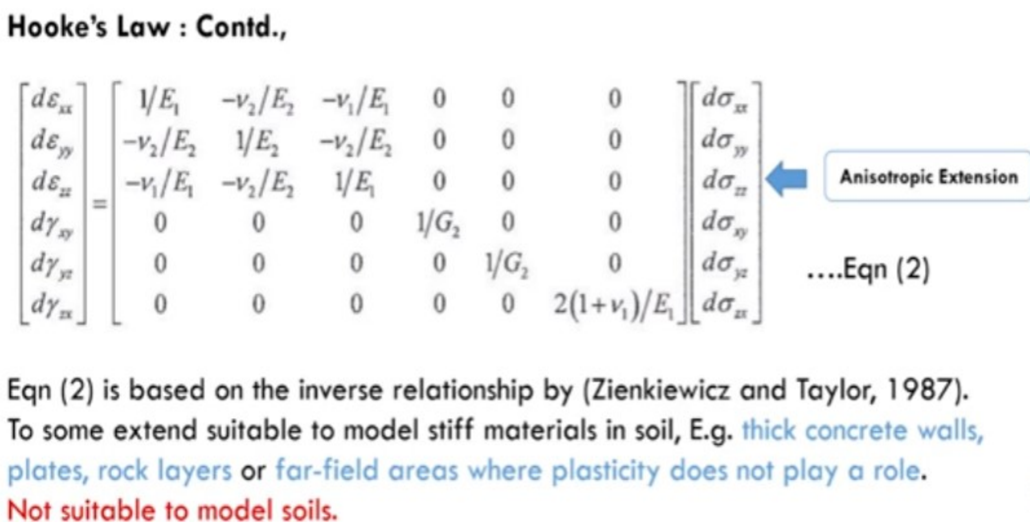


So, way back in 1987 if you can see that Zienkiewicz and Taylor, they gave an anisotropic extension by inverting the relationship as we have noticed in the previous slide. They have inverse the relationship to get a new matrix, wherein you have additional shear modulus in two directions G 2. And then we have this x, y, z directions for sigma as well as epsilon.



And then two directions of Young's Modulus and then the Poisson's ratio. So, to some extent, what I can say is that? We can model the stiff materials in the soils, for example thick concrete walls, plates, rock layers or most importantly, the far-field areas where plasticity does not play a role. So, in these conditions we can try to pick up this Hooke's law for modelling the material.

And as a caution, I would say that it is not suitable to model soils. This is a very important point. And what I would like to say is? You can have this relationship or we can consider this relationship where plasticity does not even play a role.

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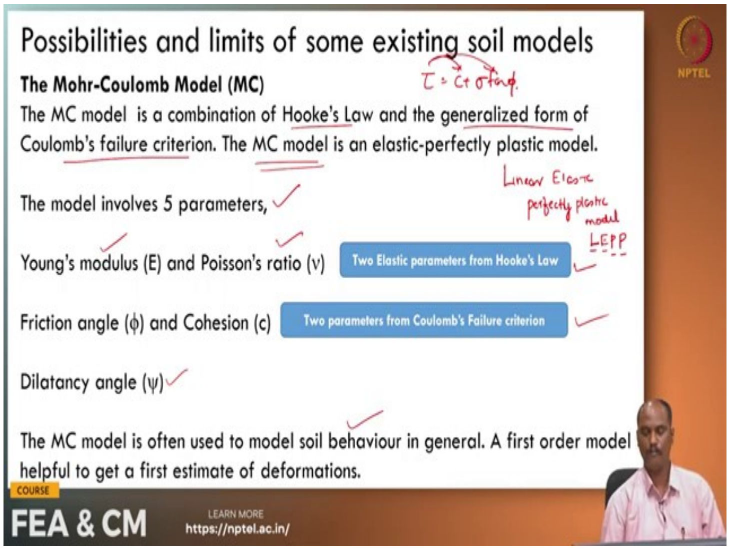

So, the next relationship or the next soil model is called as the Mohr-Coulomb model called as the MC model or very famously called as LEPP linear elastic perfectly plastic model. So, this is one of the very famous model which has been practiced even today. So, this is basically a combination of Hooke's Law which we discussed previously and then the generalized form of your Coulomb's failure criterion.

So, this is a combination of Hooke's Law plus the Coulomb's failure criterion. So, we know the Mohr-Coulomb relationship tau = C + sigma tan phi. So, the shear stress is related to these two components cohesion and the friction component. So, this Mohr-Coulomb model is an elastic perfectly plastic model or LEPP model. So, this basically involves five parameters one is the Young's Modulus E and Poisson's ratio nu.

The friction angle phi and then the coefficient C and the first two parameters are the elastic parameters that are derived from the Hooke's Law. And then whereas the next two parameters C and phi are Coulomb's failure criterion. And then additionally, we have this dilation angle psi. So, this Mohr-Coulomb model being the preliminary model or first order type of model. It is helpful to use it is full it is helpful to get the first estimate of deformations.

In order to get the preliminary estimate of deformations, it is always advantageous to use a simple type of model. So, this model is often used to model soil behaviour in general, even today.

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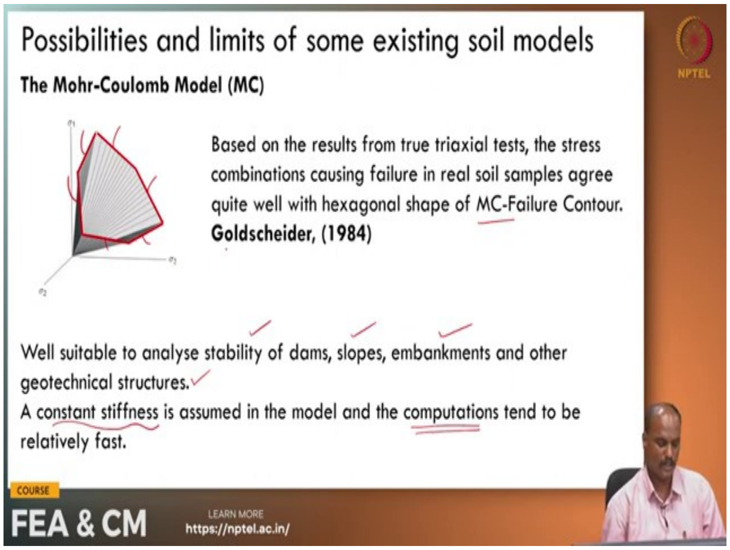

So, when it comes to this Mohr-Coulomb model, we can see that the comments from Goldscheider when based on his true triaxial test the stress combinations causing failure and real soil samples agree well within the hexagonal shape of this Mohr-Coulomb failure contour. So, we can see the Mohr-Coulomb failure contour here, so, this is in the form of a hexagon. So, these are basically the principal stress space, sigma 1, sigma 2 and sigma 3.

So, what he says is that? The real soil samples, be it any soil sample the failure model agrees well with the Mohr-Coulomb failure criterion. So, it is well suitable for analysis of stability of dams, slopes, embankments and other geotechnical structures. So, it is very, very important to note that for getting the preliminary estimates, we can very well go for this simplistic, as well as more popular linear elastic perfectly plastic Mohr-Coulomb model.

And we have to be cautious on one sign that a constant stiffness is assumed in the model so which we will discuss about the subsequent slides in the form of limitations, whereas the other advantages, the computations the time. The time that is required to run your analysis is very, very fast. Why it is fast? Why? Because the stiffness is assumed to be constant in the model.

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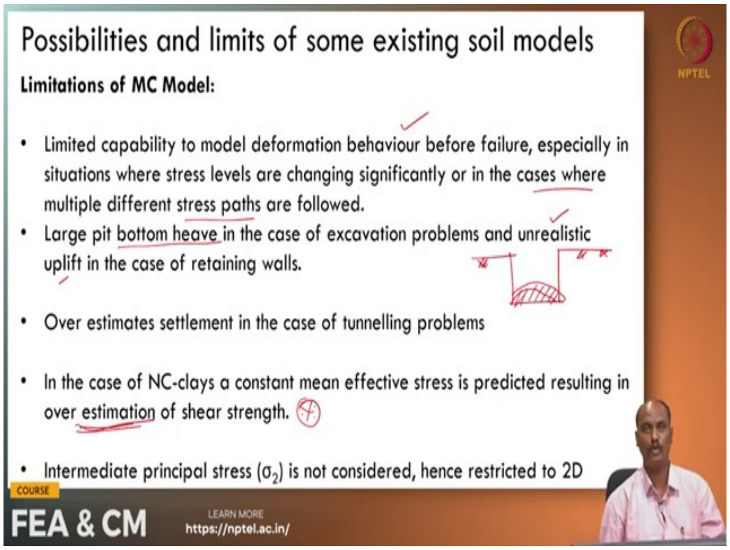

So, as I said earlier, when it comes to the limitations of this Mohr-Coulomb model so, the limited capability of the model to model the defamation behaviour before failure is one of the serious limitations. Especially, in situations where stress levels are changing so, most frequently, if you encounter stress level changes this model is not so, good. And whereas, in cases where multiple different stress paths are followed this again is a problem.

And especially when it comes to excavation problems or retaining walls. So, this particular Mohr-Coulomb model it gives a large pit bottom heave, so, you can see that this is a pit. You have this heaving problem that happens which is an unacceptable heaves especially in the case of excavation problems. So and then you also encounter unrealistic uplift in the case of retaining walls. So, this is again a problem with Mohr-Coulomb model.

So, when it comes to tunneling problems, we have unusual settlements, overestimating settlements. And then when it comes to normally consolidated clay, the very important aspect is a constant mean effective stress is being predicted. So, this is actually not the case when it comes to normally consolidated clay, especially in the case of undrained behaviour. So, as a result, we, the shear strength is overestimated.

So, this is quite dangerous when it comes to modelling of normally consolidated clay, especially underneath conditions. So, it is again an important point to consider or important limitations to consider before modelling a soil with Mohr-Coulomb criterion especially the soft clay. And when it comes to the intermediate principle stress sigma 2 this has not been considered in the formulations of the Mohr-Coulomb criterion directly.

And hence usually what we can say that? We can restrict it to 2D type of simulations or axis symmetric type of simulations.

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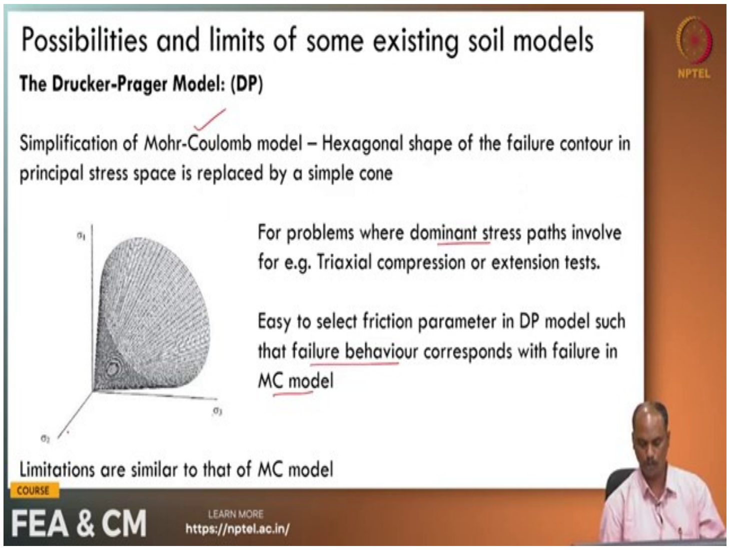

So, the next model is your Drucker-Prager model or a DP model, wherein what we see is? This is again a simplification of Mohr-Coulomb model the hexagonal shape of the failure contour is just replaced with the simple cone. So, we can see that in this stress space, namely sigma 1, sigma 2 and sigma 3. We have this simple conical shaped failure contour a real surface.

So, this is more suitable where dominant stress paths are involved, especially in the case of triaxial compression tests or extension tests, we can try to use this Drucker-Prager model DP model. And it is also relatively easy to select the friction parameter in this particular model, wherein the failure behaviour. Usually that is corresponding to the Mohr-Coulomb failure criterion.

So, the ligain, the limitations of the Mohr-Coulomb model are again relevant to this particular Drucker-Prager model also.

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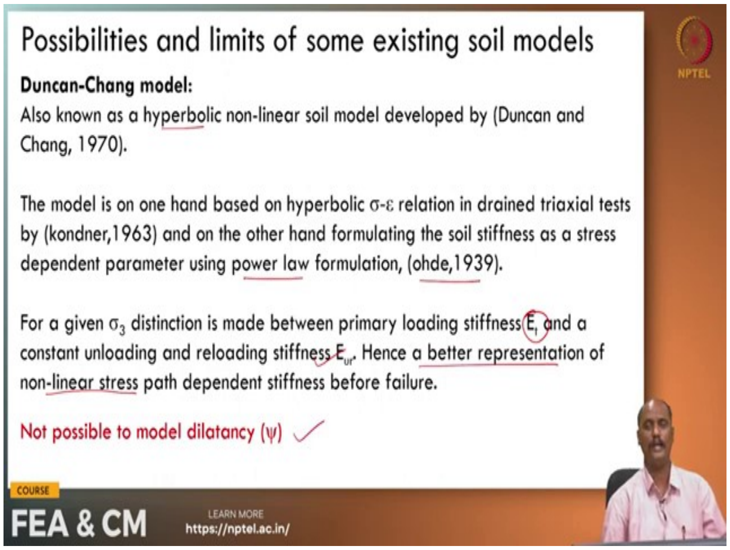

So, the next model is called as the Duncan-Chang model. So, this also called as a hyperbolic non-linear soil model developed by Duncan-Chang in 1970. So, I would just like to underline the word hyperbolic. So, this particular model was developed based on two philosophies. Number 1 the ideology behind Kondner in 1963, based on hyperbolic stress strain relationship of the drained triaxial tests which he has performed on one hand.

And on the other hand based on Ohde power law formulation wherein the soil stiffness can be addressed as a stress dependent parameter using this power law formulation by Ohde, Ohde in 1939. So, combined based on these formulations we have this Duncan-Chang model and the specialty is for a given sigma 3 for given confining pressure. So, distinction is made between the primary loading stiffness E t.

And then the constant unloading and reloading stiffness E ur. And if you can remember just before in the previous models, we did not have this unloading reloading stiffness. But here we have this unloading reloading stiffness E ur. And is a better representation of non-linear stress path dependent stiffness before failure. So, all that we are trying to do is to just see how far the real soil behaviour or the real situation that is prevailing there.

And the field can be captured with these stress strength relationship or the constitutive models and the limitation is it is not helpful to model the dilatancy parameter maybe psi.

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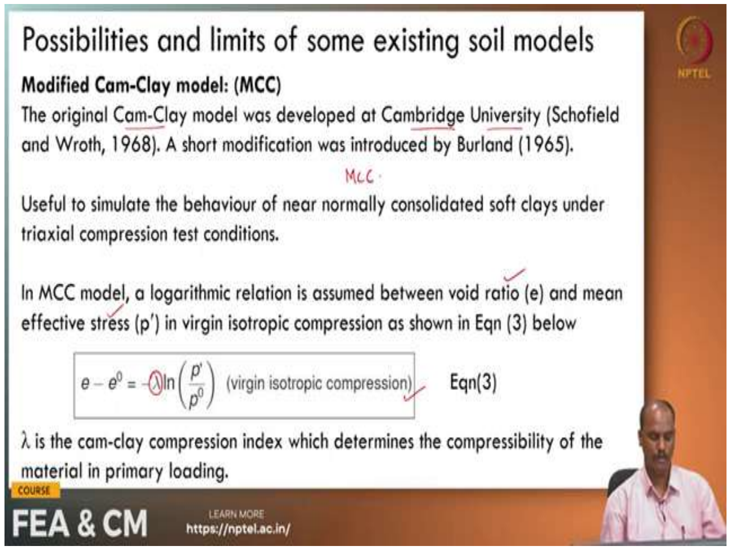

And next comes the modified Cam-Clay model MCC. So, this modified Cam-Clay is not a type of clay. It is basically setup based on or an ideology based on critical state framework. This was originally developed at Cambridge University by Schofield and Wroth in 1968 the original Cam-Clay model. So, a very short modification was given by Burland in 1965 with that modification it was called as the modified Cam-Clay model MCC model.

So, it is useful to simulate the behaviour of near normally consolidated soft clays or normally consolidated soft clays under triaxial compression test conditions. So, in this particular model there is an assumption that there is a logarithmic relationship that is existing between the void ratio e and then the mean effective stress p dash. So, this p dash is nothing but sigma 1 + sigma 2 + sigma 3 divided by 3 are the mean effective stress and then the void ratio.

$$\boxed{e - e^0 = -\lambda \ln\left(\frac{p'}{p^0}\right) \quad \text{(virgin isotropic compression)}} \qquad \text{Eqn(3)}$$

In virgin isotropic compression state, as shown in the equation so, e – e naught = – lambda ln p dash by p naught in version isotropic compression. So, this particular parameter lambda is called as the Cam-Clay compression index which determines the compressibility of the material in the primary loading. So, this lambda is very, very important in this particular soil model.

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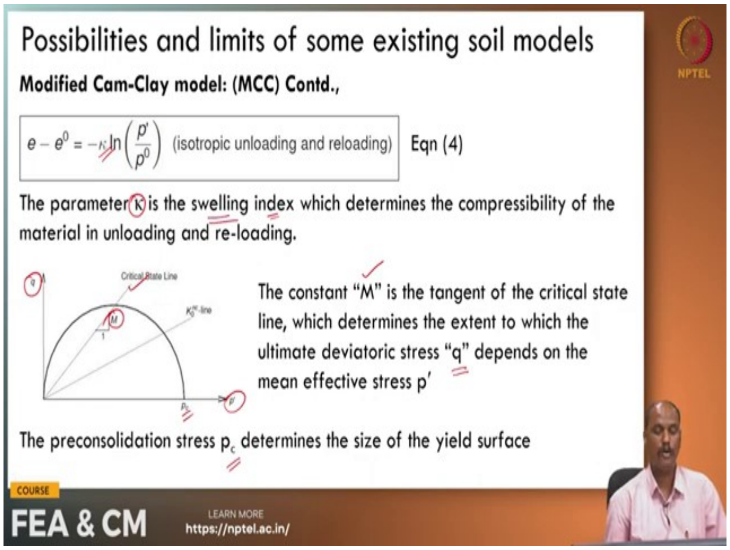

When it comes to unloading and reloading, we have this kappa parameter, kappa which is called as the swelling index which determines the compressibility of the material in unloading and reloading. So, this kappa and lambda they form the major parameter or the crucial parameter when it comes to modified Cam-Clay model. So, again, when it comes to the yield surface here we have this p dash and q.

Modified Cam-Clay model: (MCC) Contd.,

$$e - e^0 = -\kappa \ln\left(\frac{p'}{p^0}\right) \quad \text{(isotropic unloading and reloading)} \qquad \text{Eqn (4)}$$

Apart from lambda and kappa, there is a very important parameter called as your M parameter called as the friction constant. So, this friction constant M that is here is basically the tangent of the critical state line which determines the extent to which the ultimate deviatoric stress q depends, on the mean effective stress p dash. So, M is also an important factor to consider.

So, we have a specific set of equations to find out M. So, if time permits, we will discuss these equations. So, this pre consolidation stress p c determines the size of the yield surface, this is very, very important.

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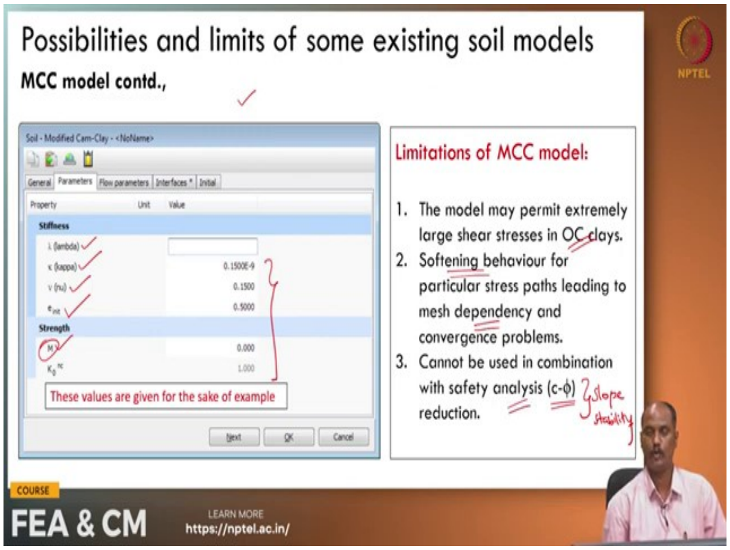

So, when it comes to modified Cam-Clay model, we have five important parameters that is lambda, kappa, nu and then the initial void ratio. So, this window, I am projecting from the finite program commercial finite program. And whatever values which I have indicated here in this particular slide are given only for the sake of example or not the exact values. So, this lambda and kappa are the inputs of this model.

And then new the Poisson's ratio and then the initial void ratio e, e naught and then the friction constant, namely M. When it comes to the limitations of this particular model so, this particular model may permit extremely large shear stresses when it comes to over consolidated clays. So, this is not at all useful for OC clays, it is suitable only for near normally consolidated, soft clays or NC clays.

So, this softening behaviour when it comes to particular stress paths or mesh convergence problems is also one of the limitations of MCC model or modified Cam-Clay model. So, you have to be pretty much careful about the mesh size and then the softening behaviour that happens occasionally. And most importantly, this particular model cannot be used in combination with C phi reduction. So, C phi reduction is what we do in your slope stability analysis?

So then C and phi are not the direct inputs of this particular model and the shear strength is also a reflection of the friction constant parameter, namely M. So, we cannot actually use this particular model in combination with slope stability.

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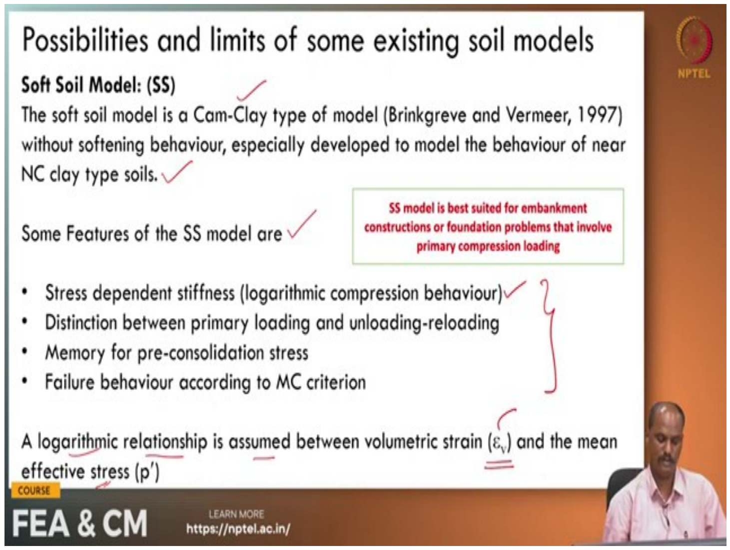

So, with this we are stepping to the next model called as the soft soil model. So, this particular soft soil model is again a modified version of a Cam-Clay type or modified Cam-Clay type of model again discussed by Brinkgreve Vermeer 1997 without softening behaviour. And I especially developed to model the behaviour of near normally consolidated type of clay soils near normally consolidated clay type of soils.

So, which is similar to modified Cam-Clay and there are some special features attached to this soft soil model which is nothing but the stress dependent stiffness or the logarithmic compression behaviour. Distinction between primary loading, unloading and reloading. The memory for pre-consolidation stress PC the failure behaviour according to Mohr-Coulomb criterion.

So, these are all the special features when compared to that of the modified Cam-Clay model. So, if we keenly notice in the case of modified Cam-Clay model, an assumption of a logarithmic relationship, existing between void ratio and effective stress was assumed in the modified Cam-Clay model. Whereas in this particular case, we have a logarithmic relationship that is assumed between the volumetric strain epsilon v and the mean effect of stress p dash.

So, if it is with void ratio and the effective stress of p dash, it is a modified Cam-Clay and when it comes to volumetric strain and then effective stress p dash mean effective stress p dash it is your soft soil model.

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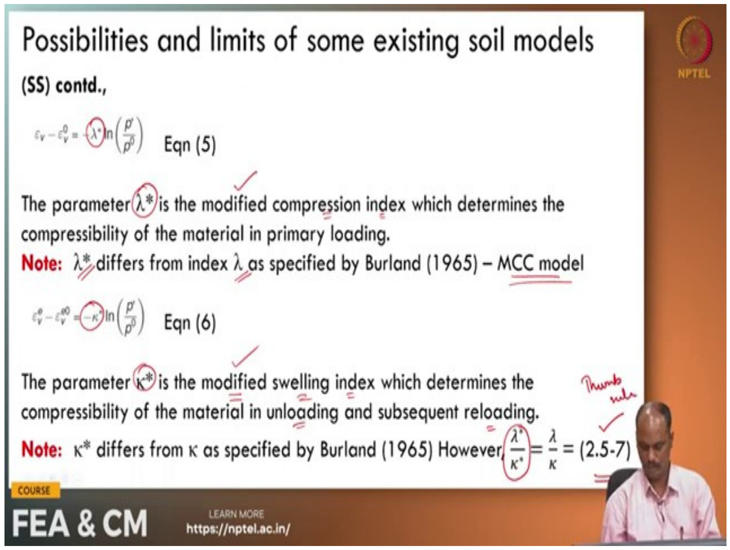
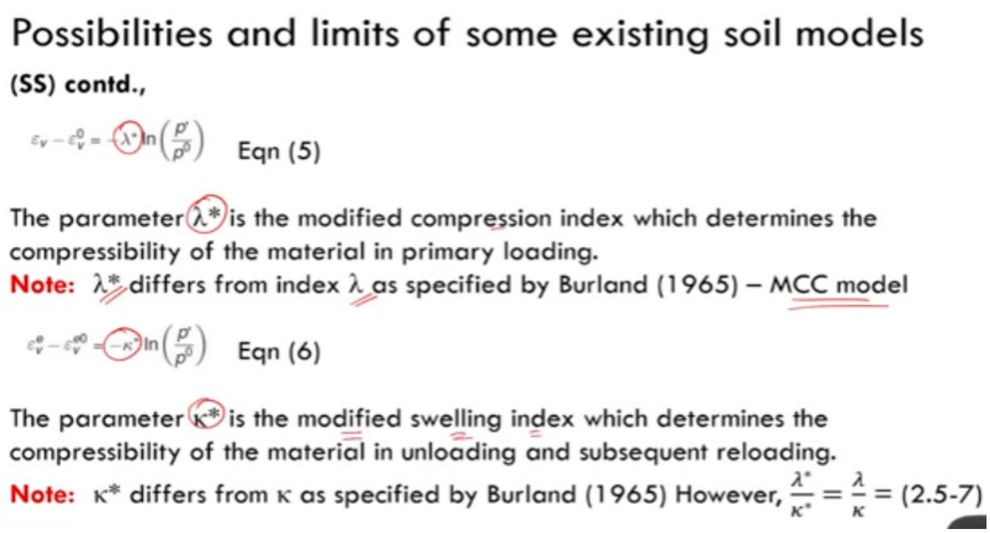


So, we can see that again. We have this lambda star which is nothing but the modified compression index which determines the compressibility of the material in primary loading. And this lambda star is different from lambda as specified by Burland in 1965 in the case of modified Cam-Clay model. And again, we have this kappa star as before, as indicated in equation 6 here as the modified swelling index.



So, earlier it was compression index and swelling index now, we have an additional term called as modified swelling index and modified compression index especially in the case of unloading and subsequent reloading. And most importantly, what we have to notice that the ratio of lambda star and kappa star is equal to lambda and kappa? But lambda star and lambda are different kappa star and kappa are different.

So, usually the range is between 2.52 to 7. So, this is a thumb rule or usually the range is 2.52 to 7.

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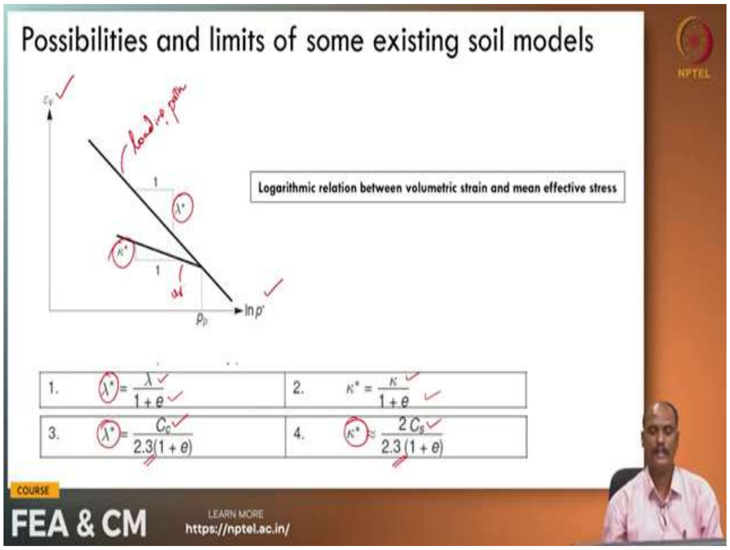
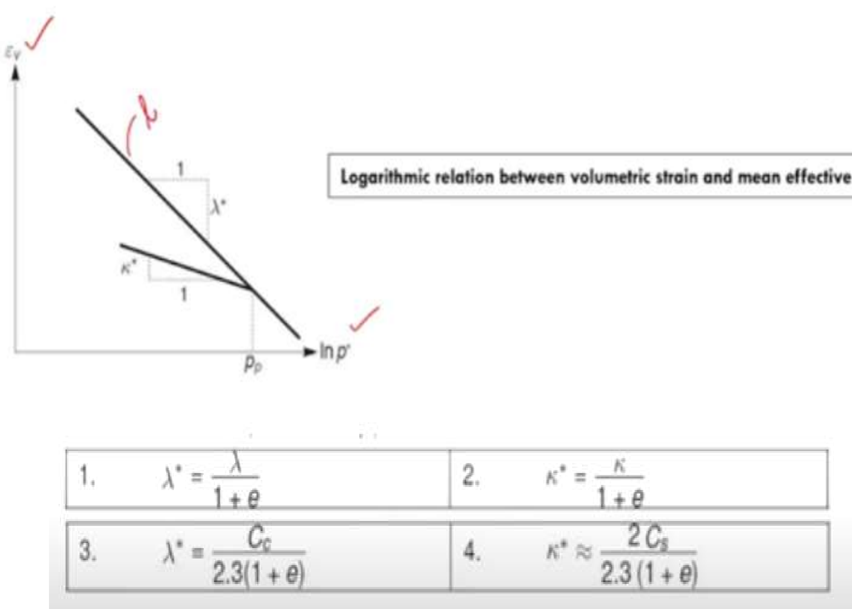


So, here I have just explained. I have just displayed the volumetric strain epsilon v which is ln p. So, this ln p dash we can see that this is the loading path and then this is the unloading and reloading path. So, this is we have this lambda star and kappa star. So, how do we compute this lambda star and kappa star? So, lambda star is nothing but lambda divided by 1 + e and kappa star is nothing but kappa divided by 1 + e.



And we can easily compute this lambda star and kappa star if we know the compression index C c and then the swelling index C s. So, this compression induction swelling index can be easily found out by from the consolidation test which we are actually performing in the soil laboratory. And then we have this 2.3 constant here which is nothing but the conversion between the two logarithms C c and C s.

So, this lambda star and kappa star are very, very important which are serving as an input for this subsoil model SSM.

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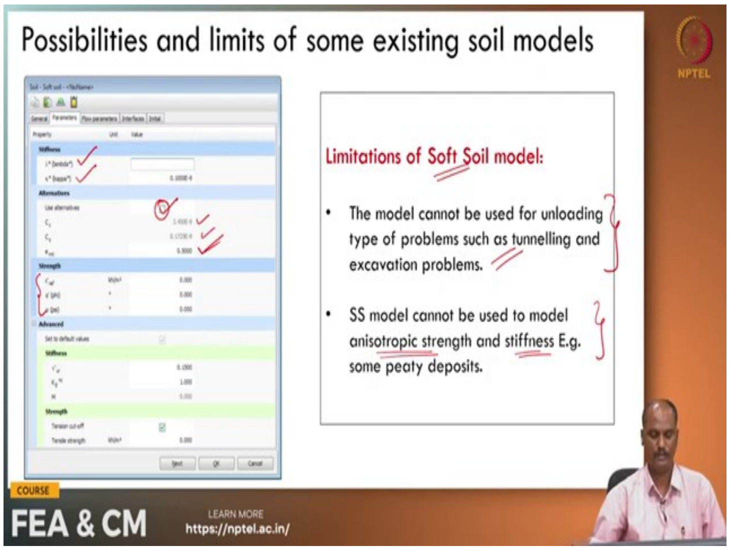

So, here I have shown the input parameters, namely the lambda star and kappa star or we can use in this particular finite element software, the alternatives. Either you can feed lambda star or kappa star or you can use C c and C s as input. Of course, e naught is not that crucial initial that crucial parameter but however, e initial has to be taken into account. And then we have this strength parameters, namely C dash y dash and then psi.

So, the only advantage are a bit of advantage when compared to that of the modified Cam-Clay model when compared to this soft soil model is that. In the soft soil model you can have a C phi reduction in combined with the other type of modelling and other things. So, when it comes to modified Cam-Clay model, your C phi reduction cannot be possible as C n phi are not the real input parameters.

So, when it comes to the limitations of the soft soil model. So, this model cannot be used for unloading type of problems, especially in the case of excavation or tunnel type of problems. So, we should not use subsoil model, especially for excavation type of problems. And when it comes to anisotropic strength and stiffness this particular model should not be used, especially when it comes to peat deposits PT deposits, wherein anisotropic strength and stiffness.

If it plays a crucial role, we should not use this particular soil model or this particular soil model does not have a capability of capturing the features of the anisotropic strength and stiffness.

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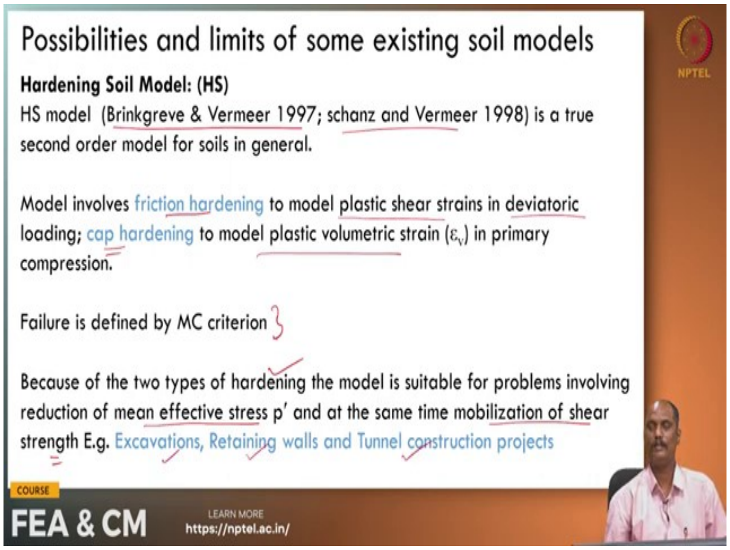

So, the very next model is called as your hardening soil model HS model. So, this was discussed in Brinkgreve and Vermeer in 1997 and also in Schanz Vermeer in 1998. So, this is basically a two second order type of model when it comes to soils. So, as far as this particular model is concerned, all the other type of models especially when it comes to the Mohr-Coulomb model that is the first order type of model.

Whereas this particular model is a bit advanced model called as your hardening soil model, this is a second order type of model. So, this involves two types of hardening, namely the friction hardening and then the cap hardening. So, this friction hardening helps to model the passive shear strains in the deviatoric loading. And when it comes to cap hardening, so, it is helpful to model the plastic volumetric strains epsilon v in primary compression.

So, with these two types of hardening, the model tries to capture the plastic shear strains as well as the polymer plastic volumetric strain namely epsilon v. So, again, in most of the soil models or in almost all the soil models, the failure is defined by Mohr-Coulomb criterion MC criterion. And because of two types of hardening, namely the friction hardening and then the cap hardening.

So, this particular model is suitable for problems involving the reduction in mean effective p dash. And at the same time, mobilization of shear strength, especially in the case of unloading problems or excavation type of problems. So, excavations, retaining walls and then tunneling construction projects so, this particular model is very, very useful.

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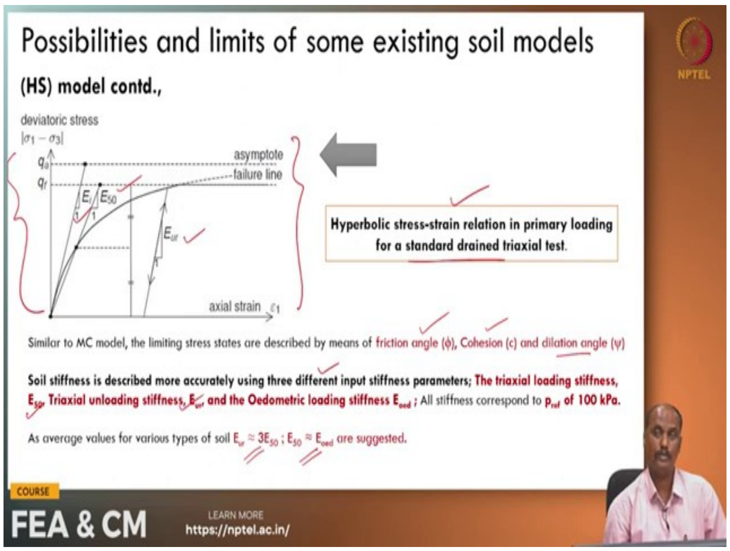
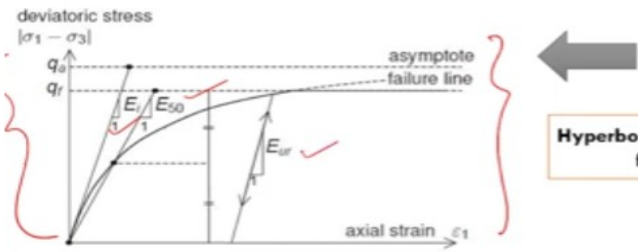


So, again, as I said that this is basically a hyperbolic stress strain relation in primary loading for a standard drain triaxial test based on which the input parameters we are going to select. So, here we have this deviatoric stress and the axial strain plot. And we have this initial modulus E i, E 50 and then the unloading and reloading modulus. So, similar to this Mohr-Coulomb type of model the limiting stress states are described by means of friction angle, cohesion and then the dilation angle psi.

Possibilities and limits of some existing soil models

(HS) model contd.,



Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test.

Similar to MC model, the limiting stress states are described by means of friction angle (ϕ), Cohesion (c) and dilation angle (ψ)

Soil stiffness is described more accurately using three different input stiffness parameters; The triaxial loading stiffness, E_{50}, Triaxial unloading stiffness, E_{ur}, and the Oedometric loading stiffness E_{oed} ; All stiffness correspond to **p_{ref} of 100 kPa.**

As average values for various types of soil $E_{ur} \approx 3E_{50}$; $E_{50} \approx E_{oed}$ are suggested.

And soil stiffness is described more precisely or, I would say, relatively accurately, in three different input parameters. So, what are they? The triaxial loading stiffness E 50, the triaxial unloading stiffness E ur. So, you are subscript, you are relates, unloading and reloading and then the oedometric loading stiffness e odometer. So, all the stiffness corresponds to the reference pressure of 100 kPa.

So, if you can see that the stiffness is more accurately captured in this particular type of model when comparing with other models, all other models, in the other hand and having the hardening soil model in one hand. The stiffness is captured in a much better fashion when compared to that of the other soil models, as a rule of thumb or for a general guideline. What we can say is that?

We can say that the unloading, reloading models can be assumed to be approximately equal to 3 times of E 50 or this E 50 can be assumed close to the automatic modulus.

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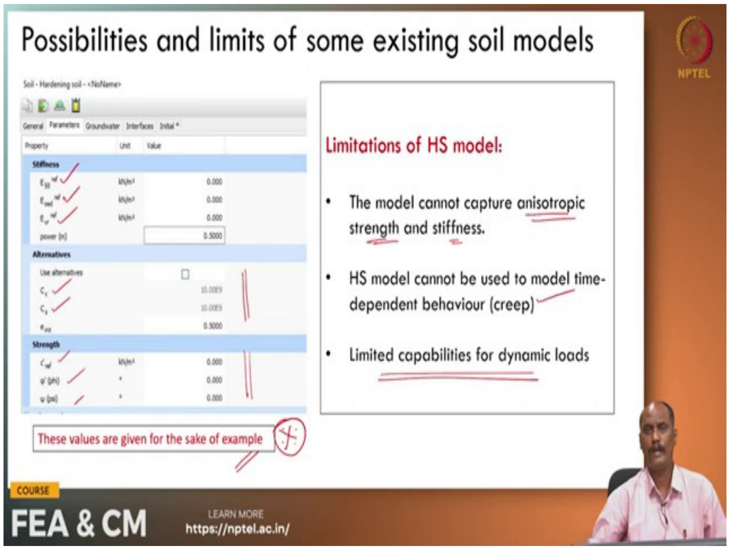

And these are all the input parameters E 50 E odometer. And then this unloading, reloading stiffness and then you can also use alternatives in the form of C c and C s when it comes to Soft-Soils. And you can have this strength parameters also C dash phi dash and psi and whatever values that are displayed here in the windows are very importantly, shown only for the sake of example and not the real-time values.

So, when it comes to the limitations of this particular soil model, so, this model cannot capture the again the anisotropic strength and stiffness. So, this model cannot be used to model time dependent behaviour, namely the creep and especially when it comes to dynamic loads. It has a limited capability. So, what we have to understand is that? So, every other soil model right from the Mohr-Coulomb soil model until the most advanced hardening soil model has it is own capabilities.

It is own limitations. So, it is up to the discretion of the designer or the user to understand the real stressed and relationship of that particular soil which he or she would like to model or try to analyse. And based on the available framework he has to intelligently choose the soil model before modelling the real behaviour or before trying to capture the real behavioural soils.

So, again, I would like to remind you that all these input parameters should be taken from laboratories and if you want field test also. And the more complex the soil model more the number of parameters that is required from either the laboratory or the field to capture the real scenario.

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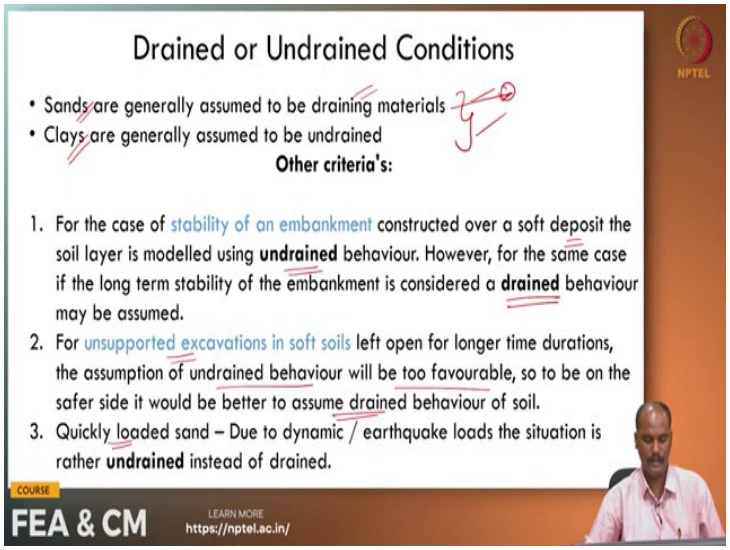

So, apart from all these things, I have just included some important aspects when it comes to modelling, especially in the case of undrained or drained behaviour. So, we all know that sands are generally assumed to be draining materials, free draining materials. So, we can usually we can assume a drain behaviour of soils sands, especially. And when it comes to clays, we generally assume the clay be here would be undrained.

And what I would like to say is? Strictly, we cannot say that sand should be modulus drained and clay should be modulus undrained. There are situations there could be situations wherein where our thought process should vary from one point to the other five that is from either drained or undrained. We have to intelligently choose or we have to intelligently decide how to model the particular situation are the real field condition?

Say, for example, when it comes to. the stability of an embankment constructed over a soft soil deposit or a soft deposit. The soil layer is usually modulus in undrained behaviour. However, for the same case, if you are interested in the long term stability of the embankment, you can assume a drain behaviour. So, it is again left with the discretion of the designer or the person who is trying to analyse it.

So, we have to think about the real scenario or the fundamentals based on which these finite element models they work. So, for which a strong knowledge in geotechnical engineering is required and especially in the case of unsupported excavations in Soft-Soils, left open for a longer time durations. So, undrained behaviour will be very too favourable. So, to be on the safer side it would be better to assume a drained behaviour also.

And when it comes to quickly loaded sand as I said that here we can say that sands, usually they are free running materials assumed to be drained. But in this particular case, especially in the case of earthquake loads or dynamic loads, what we can see is that? It is rather intelligent or it is rather apt to model the soil as undrained instead of a drained behaviour.

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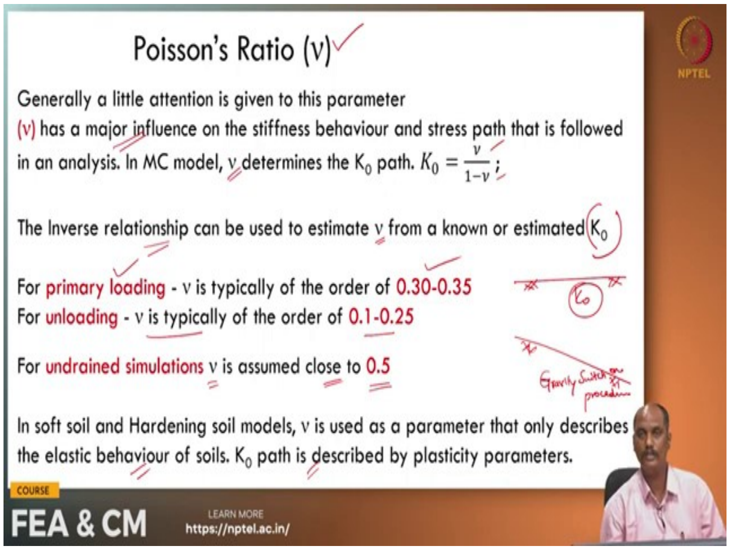

So, the next important parameter which is given least important is the Poisson's ratio. So, this has very major influence on the stiffness behaviour or the stress path that is followed on the analysis. So, especially in the case of Mohr-Coulomb model this Poisson station new determines the cannot path, so, this K naught does nothing but nu by 1 – nu given by Yakkis relationship.

The inverse relationship can also be used to estimate nu from a known or an estimated K naught. So, this K naught is also a very important factor to consider when you do a finite element analysis. So, this K naught procedure, this K naught procedure can be set up in two different ways, especially for horizontal grounds when you have this stresses or homogeneous you can activate K naught directly.

Especially, when it comes to sloping grounds, so, gravity switch on procedure is usually followed. So, we have this kind of options in the commercial finite element programs which we normally adopt for modelling. And for primary loading the nu is usually in the range of

0.30 to 0.35. For unloading nu is typically, in the order of 0.1 to 0.25. For undrained simulations in reality, nu should be assumed close to 0.5, not exactly 0.5.

I am again stressing it should be close to 0.5 and not exactly 0.5. So, when it comes to soft soil model and hardening soil models, the parameter nu is usually used as a parameter that only describes elastic behaviour of soils. Whereas K naught is described by plasticity parameters. So, the same parameter nu in one type of model it does one job when it comes to the other type of model the job is different.

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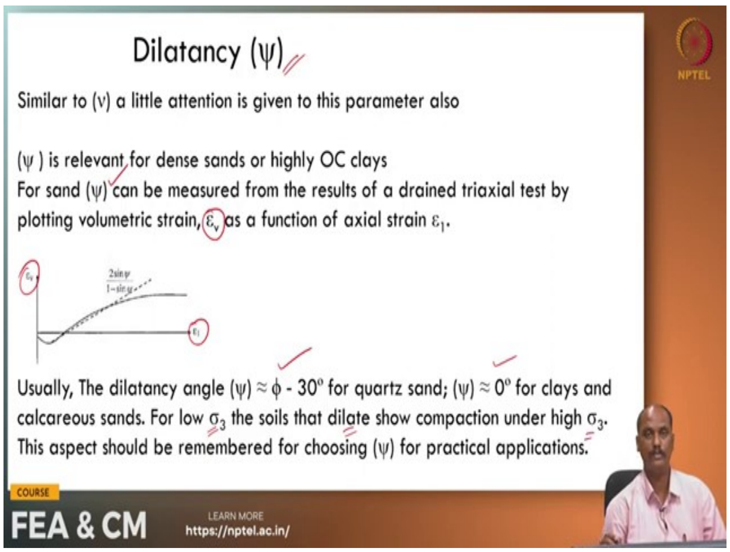

So, the last and the final thing which I would like to say is the dilatancy parameter, namely psi. Similar to nu only a little attention is given to this particular parameter also. So, this psi is relevant for dense sands and highly over consolidated clays. For sand the psi can be assumed or measured from the results of drained triaxial tests by plotting the volumetric strain epsilon v against the axial strain epsilon 1.

So, we can measure the dilation angle psi from the triaxial test by plotting epsilon v against epsilon 1. So, usually, the dilatancy angle is taken as phi – 30 degrees for quartz sand. And usually, psi = 0 for clays and then calcareous sands. When it comes to soils with low confining pressure low sigma 3 the soils tend to dilate and show a compaction under higher sigma 3 that is under higher confining pressures.

So, this particular aspect has to be remembered when it comes to practical applications of psi values.

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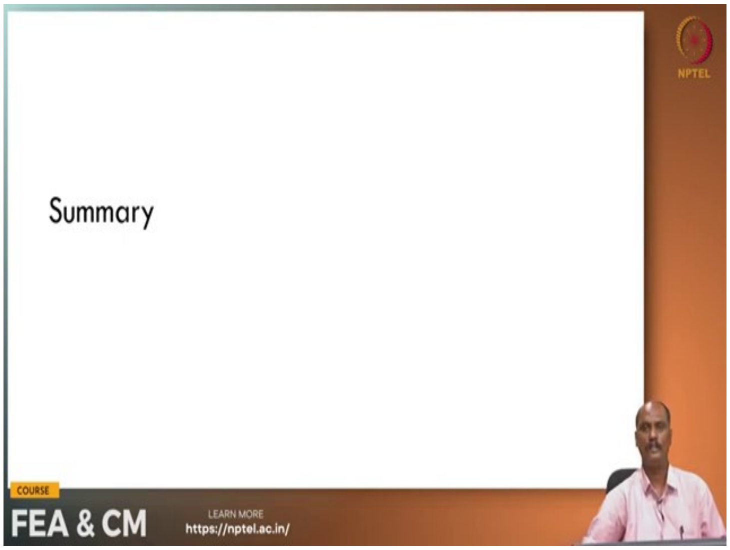

So, with this lecture ends so, as a summary or a recapitulation. What I would like to say is? We have discussed the real soil behaviour or the soil behaviour in the practical sense. And how the soil behaviour is getting captured in the stress drained relationships that different applications which we have in the real time scenario. Especially, when it comes to a embankments, tunnels, excavation type of problems, foundations, etcetera.

And in this particular lecture, we have discussed the different types of soil models right from the simplistic, Hooke's Law, Mohr-Coulomb Model, Drucker-Prager Model, Duncan-Chang Model and then the modified Cam-Clay Model, Soft Soil Model, hardening soil Model, etcetera. And then some aspects of the modelling when it comes to Poisson's ratio drained and undrained behaviour or the K naught procedure has been discussed in this particular lecture.

And what has been not covered is that the dynamic aspects of the soil behaviour has not been covered? And most importantly, when it comes to creep behaviour, there are some other type of soil models wherein creep coefficient namely mu, has to be measured and then incorporated. And then when it comes to small strength stiffness, we have some other advanced constitutive models which include some small strain stiffness also.

And that has not been covered in this particular lecture. So, another important thing to note, finally, is that whatever the values or the range of numbers which we have discussed in this particular lecture are given for the sake of guidance. When it comes to real time scenario, we will have to test the soils and find out the real numbers. And then go ahead with the modelling.

So, with this particular concepts and recapsulation. I would like to end this lecture and I would like to thank the particular organizer for giving me an opportunity to share the very little information which I was able to gain during my research tenure. Thank you very much.