Advanced Soil Mechanics
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Lecture - 20
Shear Strength Granular Soil - II

Welcome back. So, this is the continuation of the previous lecture, shear strength of granular soils. In this we will see first what is the concept of intrinsic friction? We have already discussed about dilation and how it gets suppressed at high normal stress, so something in continuation with that.

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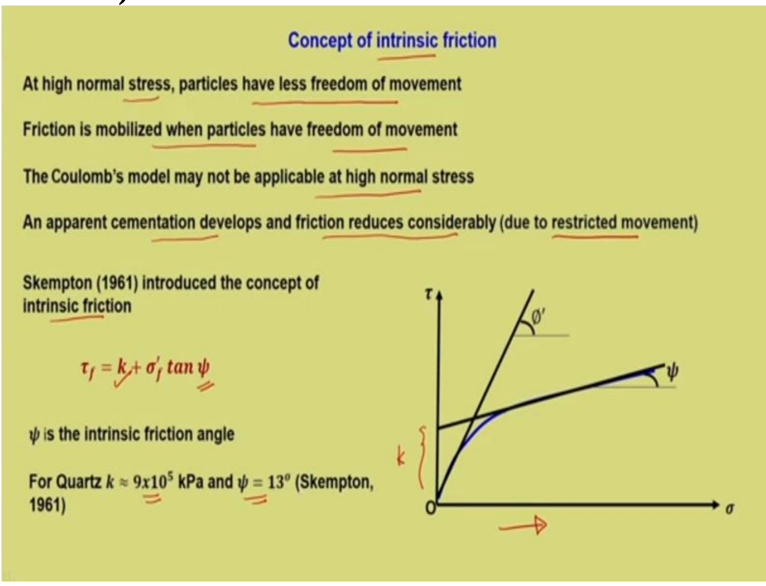

This is what is known as the concept of intrinsic friction. At high normal stress particles have less freedom of movement, we have seen that the particles are tightly packed and hence it has less freedom of movement. Now friction is mobilized only when particles have freedom of movement. In order to mobilize friction there should be movement, if it is tightly packed then the surface forces activates. So, we need some sort of movement for friction to mobilize. So, the Coulomb's model may not be applicable at high normal stress.

A kind of apparent cementation develops and the friction reduces considerably due to the restricted movement. So, Skempton in 1961 introduced the concept of intrinsic friction, let us see what is intrinsic friction. So, this is what it is, as the normal stress increases, it restricts the freedom of movement and hence the mobilized friction drastically reduces which is indicated by Ψ. So, now, it is considered different from the Coulomb's model wherein we discussed in terms of Ψ’.

So, at a high normal stress, there is some deviation in its behavior and there is a kind of apparent cohesion we cannot call it as real cohesion, a kind of apparent cementation which is represented by the term k. So, this basically is k and the angle is Ψ which is considerably less than φ'. So, this equation $\tau_f = k + \sigma_f \tan \Psi$ where Ψ is called the intrinsic friction angle. And for quartz k is found to be close to 9 x 10^5 kilo Pascal and Ψ = 13 degrees as per Skempton, 1961.

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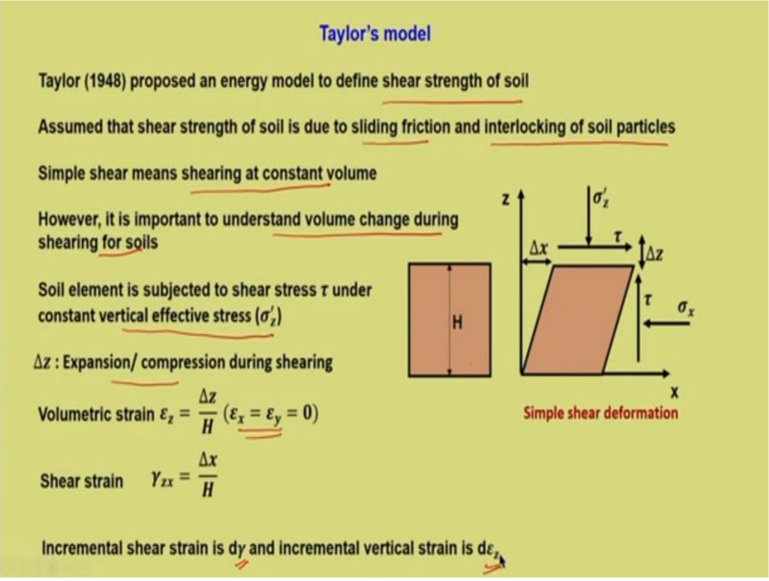

So that is all about intrinsic friction, just to add to the discussion of dilation and the suppression of dilation. Next we will see Taylors model. Taylor in 1948 proposed an energy model to define the shear strength of soil. Now this model is very much similar to that of Coulomb's model but marginally different in some assumptions. We will see what actually it is? Assumed that shear strength of soil is due to sliding friction and interlocking of soil particles, in Coulomb's model also it is due to sliding friction.

But the sliding happens along a failure plane whereas, in the case of Taylors model, there is no assumption of failure plane but the shear strength has 2 contributions, one is due to the sliding friction of the soil particles in movement and the other one is due to interlocking of soil particles. When I say particles in movement it is on a micro scale. So, it is not actually moving there is a sort of relative movement because obviously the friction gets mobilized.

So, what has been done in Taylor's model is? It has splitted the 2 effects whereas, in the case of Coulomb's model, this was embedded in the friction angle itself, it was an additive component. Now simple shear means shearing at constant volume. So, Taylors model actually refers to simple shear. Now what do you mean by simple shear? Simple shear means those shearing that happens at constant volume.

You remember when we discussed the first module, we said that the effects are decoupled. We are studying a material by decoupling the volume change and shearing or the deviatory component separately. But this will not work all the time fine for soils because we need to understand what is the sort of volume change that happens during shearing, otherwise it is not meaningful. So, there is a sort of deviation from whatever we have learnt initially in module 1.

We need to discuss about pore pressure or volume change during shearing and this is very specific for cohesive soils. So, it is important to understand the volume change during shearing for soils. So, there is a sort of deviation from our simple shear concept, the soil element is subject to shear stress τ under constant vertical stress σ'_z and this is the soil element it is subjected to τ which is the shear stress and σ'_z which is the constant confining stress or the constant confining vertical effective stress σ'_z.

And there is a kind of rotation and it is not actually rotation there is a displacement y Δx in the x direction and the compression expansion which is denoted by Δz in the z direction. So that is the expansion compression during shearing. And volumetric strain ε_z can be written as Δz / H. This H is the initial height and this is close to a plane strain condition where $\varepsilon_x = \varepsilon_y = 0$.

So, shear strain $\gamma_{zx} = \Delta x / H$. So, this is the concept of a linear strain and the shear strain is discussed in module 1, so that it is easy for you to understand. So, $\Delta x / H$ is the shear strain, so incremental shear strain is $d\sigma$ and $d\varepsilon_z$.

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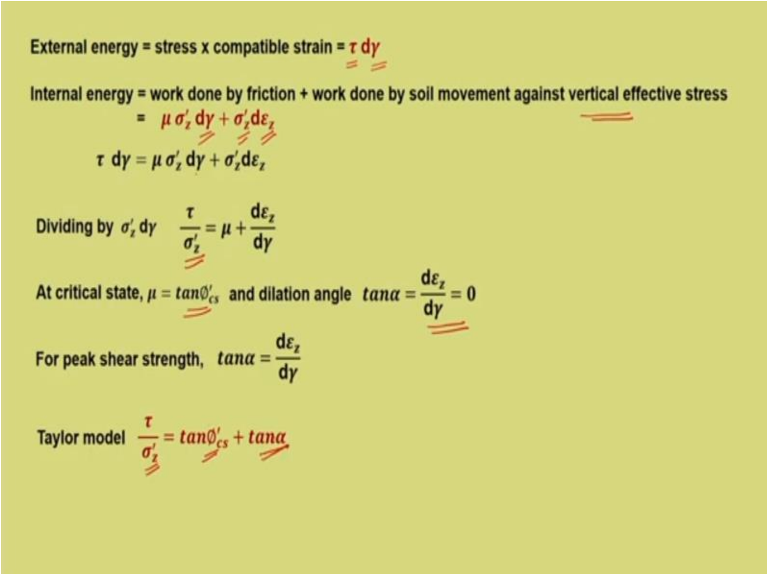

According to Taylors model external energy because it is a energy based concept is stress into compatible strain. Now what causes shear that is $\tau * d\gamma$. So, this is the compatible strain or the shear strain. An internal energy is essentially associated with work done by friction specifically sliding friction and work done by soil movement against the vertical effective stress. So, we can write $\mu * \sigma'_z$ becomes the shear or the sliding friction into $d\gamma$.

So, this becomes the shear strain plus $\sigma'_z * d\varepsilon_z$ that is σ'_z is the vertical stress $d\varepsilon_z$ is the vertical strain. So, for equilibrium we can equate internal and external energy as shown here. Now dividing by $\sigma'_z\, d\gamma$ we will get $\tau / \sigma'_z = \mu + d\varepsilon_z / d\gamma$ or shear strain. Now we know that at critical state μ becomes equal to tan φ'_{cs}.

That is, there is no dilation, whole of the sliding friction component is associated with the critical state friction angle and dilation angle $\tan\alpha = 0$. But for a peak shear strength condition $\tan\alpha = d\varepsilon_z / d\gamma$ or vertical strain to shear strain. So, Taylors model is then the summation of these 2 components. So, τ / σ'_z which is written here is equal to tan φ'_{cs} which is the component of critical state or the fundamental aspect plus the dilation aspect. Now what has happened? We have splitted these 2 instead of $\varphi'_{cs} + \alpha$ in the Coulomb's model.

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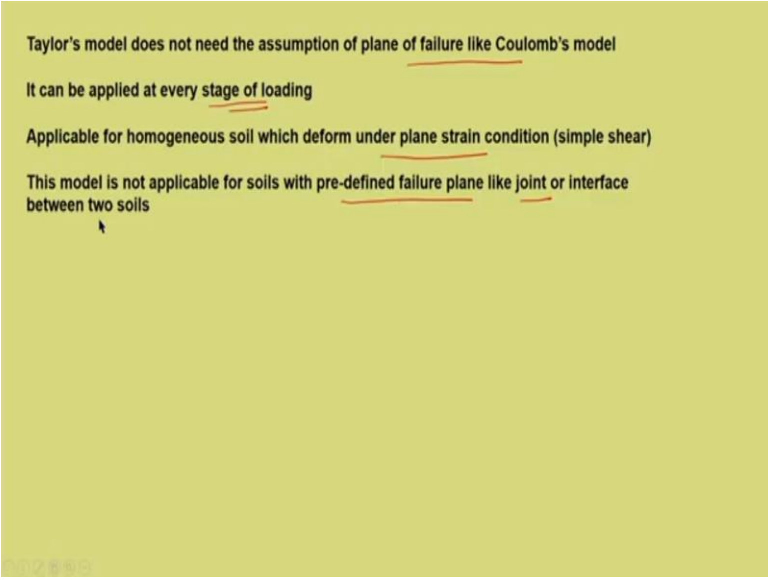

So, Taylor model does not need the assumption of plane of failure like Coulomb's model. It can be applied at every stage of loading because the relative aspect of dilation and the critical, the basic friction angle is taken care of by the summation. Whereas, in the case of Coulomb's model it is within the friction angle together, the total friction angle. So, it is applicable for homogeneous soil which deform under plane strain condition or a simple shear. The model is not applicable for soils with predefined failure plane like joints or interface between 2 soils. So, here there is no consideration of plain of failure.

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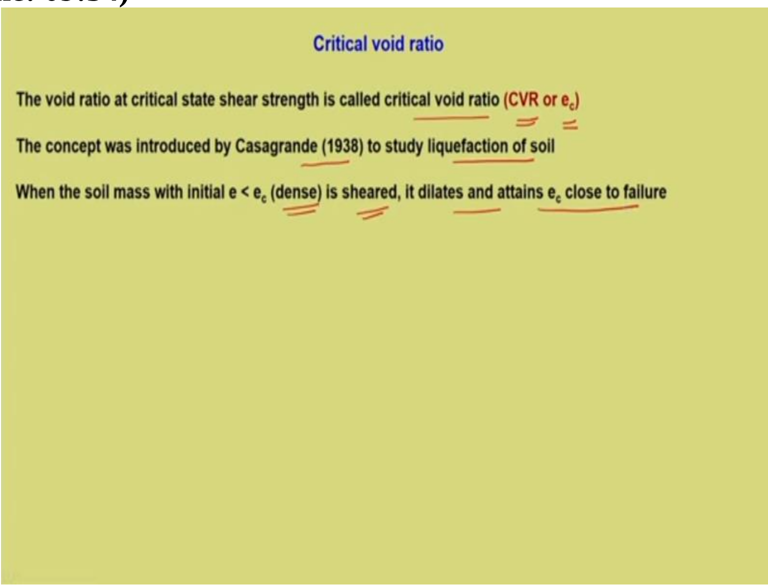

So, now, we will come to the next concept what is known as critical void ratio. It is the void ratio at critical state shear strength and it is called critical void ratio CVR or e_c. Now what is this critical void ratio? This concept was introduced by Casagrande in 1938 to study liquefaction of soil. What it states is that when the soil mass with an initial void ratio less than e_c, e_c is the critical void ratio that is a dense state when it is sheared, it dilates and attains e_c close to failure.

So, when a dense state of the soil if it is sheared then the void ratio will increase because of the dilation and finally it reaches what is known as a constant volume or a constant void ratio at close to critical state. So, it is called critical void ratio or constant volume void ratio.

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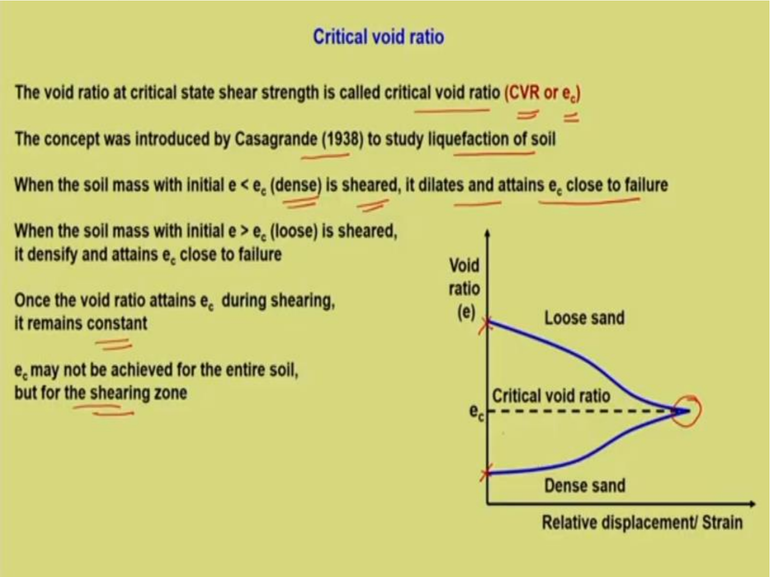

So, this is what it is, this void ratio on y axis relative displacement or strain on x axis, this is the point. So, initial void ratio is less than e_c and this is a dense state of the sand or soil and when it is sheared it dilates and it reaches a constant void ratio which is known as critical void ratio at this point. Similarly when a soil mass with initial e which is greater than e_c that is for a loose state when it is sheared it densify and attains easy close to failure, failure means here we refer to critical state or critical void ratio where the volume becomes constant.

Once the void ratio attains e_c during shearing, it remains constant. So, for all practical purpose critical void ratio remains constant, once it reaches that particular state. e_c may not be achieved for the entire soil but it may be for the shearing zone which means the entire soil may not reach the state of e_c basically where the failure zone initiates that particular zone e_c will be achieved.

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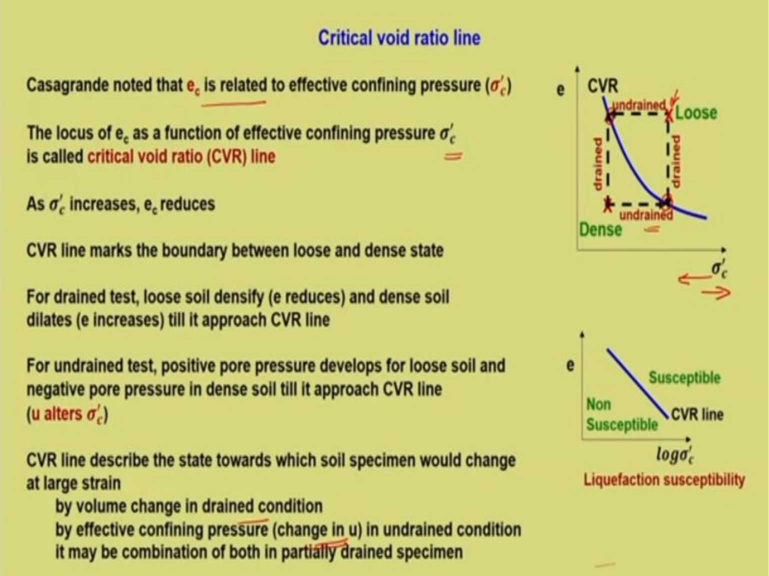

So, Casagrande noted that e_c is related to effective confining pressure σ'_c. So, depending upon the confining pressure, e_c will change, why? The amount of dilation effect also changes depending upon the confining pressure. So, accordingly whatever e_c it achieves that also changes. The locus of e_c as a function of effective confining pressure σ'_c is called critical void ratio line, CVR line, as can be seen here. It is e versus σ'_c.

Now this blue line represents the locus of critical void ratio, we can see that as σ'_c increases, e_c reduces. Now this blue line is specifically e_c, variation with σ'_c. CVR line, this particular line marks the boundary between loose state and dense state. Let us consider a drained test now, we are discussing this in general. For a drained test a loose sample, this is the loose sample, densify that is e reduces for a drained test. What is meant by drained test?

There is no pore water pressure and hence the volume changes. For a loose state it reduces till it reaches the critical void ratio and dense soil, this is the dense soil, as the shearing happens, it increases in a drained test because drained test volume change happens. So, it increases and reaches the critical void ratio. Now for an undrained test for the same case, let us say the case of loose state. What happens in the undrained test?

In the undrained test, there will be development of positive pore water pressure. We will be discussing this in detail in the next lecture on shear strength of cohesive soil. So, for the time being, we will understand that for undrained test there will be development of pore water pressure. Now in loose material, it will be positive pore water pressure. Now because of positive pore water pressure, the confining pressure that is, σ_c that keeps on reducing because the effective confining pressure keeps reducing.

So, σ'_c keeps reducing till it reaches the critical void ratio line or till it achieves critical void ratio, similarly in the case of dense state, we can see that as shearing happens under undrained condition, there will be negative pore water pressure and till it approaches the critical void ratio at close to failure. Now what is happening here? In the case of dense material which has a tendency to dilate the pore water pressure development will be essentially negative, keeps on reducing, so it becomes negative.

Now because of this negative pore water pressure, σ'_c that is the effective confining pressure keeps increasing, this we know effective stress keeps increasing for negative pore water pressure till it reaches the constant void ratio line.

So, CVR line describe the state towards which soil specimen would change at large strain. Now by the time it reaches the critical void ratio line, it would have undergone large strain. So, we are talking about a critical state of that particular material which mostly happens at 10 to 20% strain, so that is what it means. So, CVR line is the line towards which soil state moves during shearing and finally it reaches that particular constant volume and then remains constant.

So, CVR line it is the locus of the line which towards which the soil specimen would change at large strain, by volume change in drain condition which we have seen, by effective confining pressure in undrained condition which is the manifestation of pore water pressure change, this is decreasing or this is increasing and it may be a combination of both in a partially drained specimen where there will be an interplay of both pore water pressure and volume change. So that is the concept of critical void ratio.

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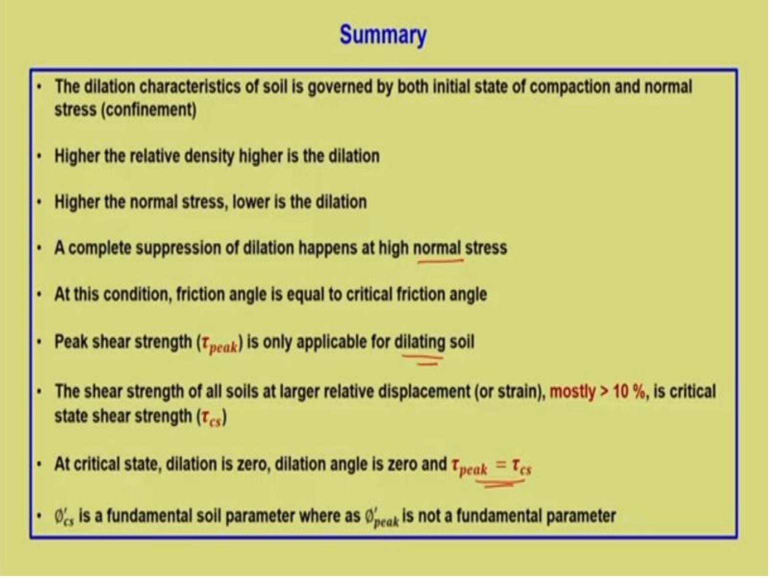

Now we will summarize the shear strength of granular soil where the dilation characteristics of soil is governed by both initial state of compaction and normal stress which is the confinement that we have discussed. Higher the relative density, I mean the initial compaction state, higher is the dilation because interlocking is more and hence the dilation will be more. Higher the normal stress, lower is the dilation because there is a kind of suppression that happens.

A complete suppression of dilation happens at high normal stress. At this condition, friction angle is equal to critical friction angle. Peak shear strength, τ_{peak}, is only applicable for dilating soil. The shear strength of all soils at larger relative displacement or strain mostly greater than 10% is critical shear strength τ_{cs}. So, this is the kind of minimum guarantee shear strength for a given material.

So, whatever be the state we will see that this soil of a particular type possess critical shear strength which is τ_{cs}. At critical state dilation 0, dilation angle is 0 and hence $\tau_{peak} = \tau_{cs}$. φ'_{cs} is a more like a fundamental soil parameter whereas, phi dash peak is not a fundamental parameter because φ'_{peak} depends on whether dilation happens or not. Whereas, φ'_{cs} is not a function of that.

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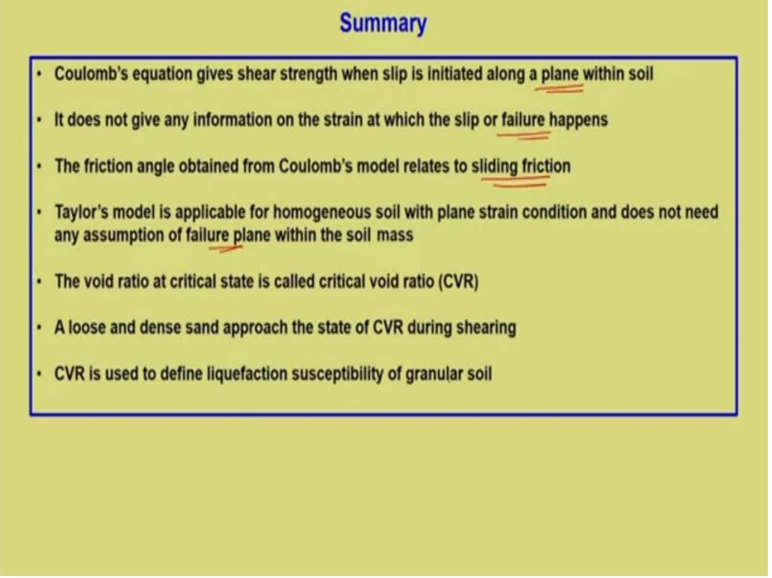

Coulomb's equation gives shear strength when slip is initiated along a plane within soils. So, there is an inherent assumption of plane of failure developed within the soil for Coulomb's equation to be valid. It does not give any information on the strain at which that slip or failure happens. We have discussed this aspect before also that is, all these models does not talk about the strain at which the slip or failure happens.

The friction angle obtained from Coulomb's model relates to sliding friction. This is true for Taylors model as well. And Taylors model is applicable for homogeneous soil with plane strain condition and does not need any assumption of failure plane within the soil mass. The void ratio at critical state is called critical void ratio that is the constant void ratio which a soil mass attains in towards failure when it is sheared.

A loose and dense sand approach the state of CVR during shearing. So, CVR is used to define liquefaction susceptibility of granular soil. So that is all about shear strength of granular soils. In the next lecture we will see shear strength of cohesive soil which is even more complex due to the fact that, it is highly influenced by pore water pressure and drainage condition, so that we will see in the next lecture. That is all for now. Thank you.