Advanced Soil Mechanics
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Lecture - 31
Some Additional Aspects of Shear Strength

Welcome back to all of you. In the last lecture, we have completed the interpretation of triaxial tests, the last one being the consolidated drain test. In today's lecture, we will see some additional aspects which has not been covered till now related to shear strength of soils. And with this lecture, we will be winding up this particular module. So, we will get on to some left out aspects which we will discuss just now.

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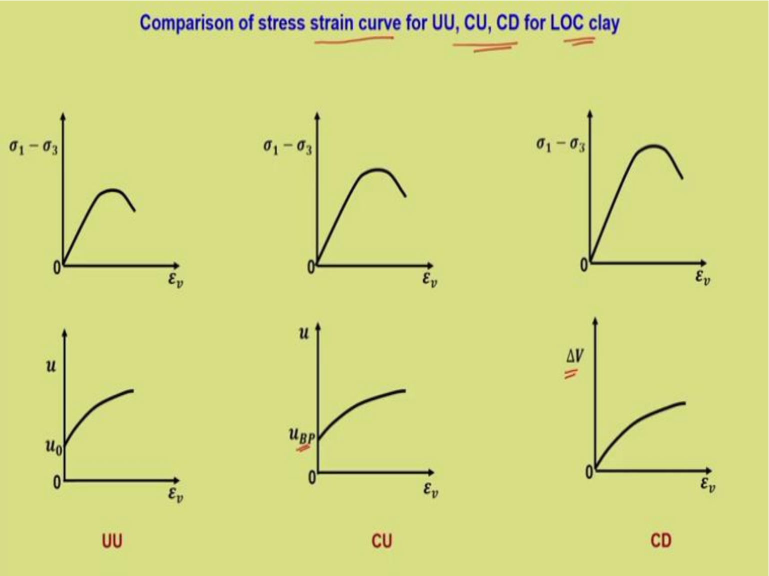

So, we will see the first aspect is the comparison of stress strain curve when you have all the 3 tests done on a particular soil. All the other conditions remaining same, how will these results compare, I mean to say UU, CU and CD. This comparison is not conclusive but a conceptual comparison we can say. So, the first one is UU test, the stress strain response and the pore water pressure generation as u nought is the initial pore water pressure before shearing.

That is may be at the consolidation stage. And then during shearing the pore water pressure goes up. CU, it is also the same. The stress strain response more or less is identical and in the event of back pressure application, the pore water pressure corresponding to consolidation stages u back pressure. And when it comes to consolidated drain instead of pore water pressure it is volume change. Now, the quick comparison it is only in terms of stress strain response.

You can see that the final pore the failure stress corresponding to all the 3 is progressively increasing as it comes from UU to CU, CU to CD. Don't take this comparison as conclusive. Because there are various other factors that can alter these results. So, that is why I told all other factors remaining same, a more or less comparison will be in this form in terms of its strength. So, that is all about the comparison of the 3 types of tests that you perform in triaxial testing.

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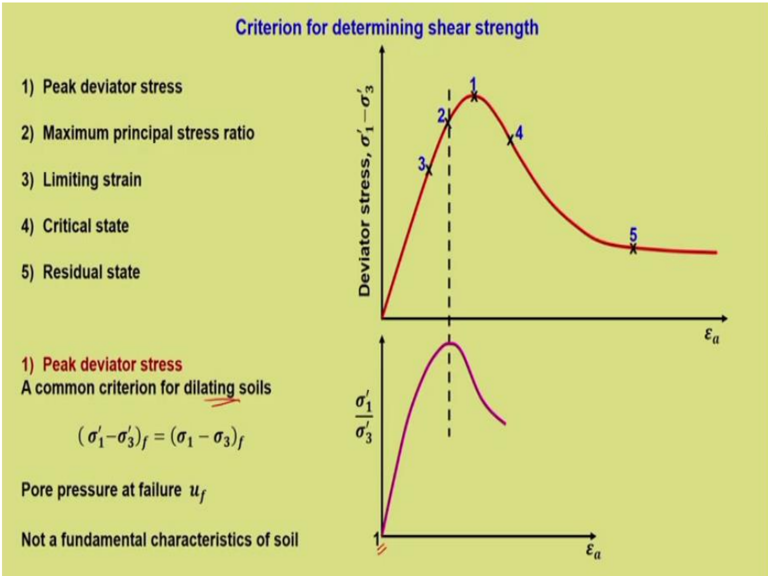

Now, the next one is the criteria for determining the shear strength. Now, why should we have this question at the first place? We conduct triaxial tests, any of these 3 tests. Plot the Mohr circle. Now, where is the criteria coming into picture. So, when we have conducted the test we would have either taken the peak stress at failure for plotting the Mohr circle. It will not be done such that in one of the tests we will take peak the other one will take ultimate or critical. It will not happen like that. So, there will be some criteria based on which the failure state of the soil is defined. And we are we have already seen that the peak stress is only possible in the case of dilating soils. So, always it may not be possible. In the case of normally consolidated it progressively densifies. So, there is no particular peak or particular way of defining the failure state.

So, what should be done in those cases? Having known the different type of tests and its interpretation now the job is how to define the failure envelope and for that what criteria need to be taken. So, that is why we will discuss, what are the common criteria which are adopted for defining the failure envelope or shear strength of the soil? Now, this deviator stress versus axial strain this can be volumetric strain as well.

Now what are the different criteria? It is very it is quite explicit that in such a case we can always take peak as the criteria provided we can identify the peak. So, the first one is peak deviator stress. So, 3 samples we conduct in the triaxial testing. In all the 3 samples the identical samples we will consider the peak deviator stress. Now, this is possible only if the soil that is tested is either dense state or it is an over consolidated state.

So, this is the first criterion for determining shear strength. The second one is maximum principal stress ratio or more specifically maximum effective principal stress ratio. And we have seen this and we have discussed this before also maximum stress obliquity. So, that is what we have already seen. The same thing it becomes a very useful criteria for defining the shear strength of the shear envelope. And that is very specific to may be un-drained soil.

So, in the figure you can see that the plot between σ_1'/σ_3' versus ϵ_a at the isotropic consolidation it is 1 because $\sigma_1' = \sigma_3'$. So, it starts at 1. So, this is the second criterion which is possible. Now, second this corresponds to this peak and we can see that the peak of deviator stress. It is not matching with the peak of σ_1'/σ_3'.

The next one is limiting strain that is the third point we will discuss each of these separately. So, we will first see what are the available criterion. Then, comes the critical state. So, this also to some extent we have seen while discussing the stress strain response. And the fifth one is residual state which is at a very low stress condition. So, we will see each of these one by one.

So, peak deviator stress does not need much of explanation because we have seen this all throughout the discussion of this particular module. There is a common criterion for dilating soils, $\sigma_1' - \sigma_3' = \sigma_1 - \sigma_{3f}$. Pore pressure at failure is u_f. It is not a fundamental characteristic of soil. Now I think this becomes again a testing or revision of whatever we have explained before.

Why it is told that it is not a fundamental characteristic of soil. It is because it is dependent on the initial stress history and the confinement. Both is going to affect or determine how much dilation it will undergo. So, it depends basically on how much the soil will dilate. So, hence it cannot be considered as a fundamental property because it keeps changing. That is why most of the time for the design there is a tendency to neglect the cohesive component.

The reason is you might have obtained cohesive component while doing the test in the lab but what is the guarantee that the same cohesion will be available in the field. I am not talking about the cohesion that is related to the actual cementation. That will be there. But if it is not because of actual cementation and it is exclusively due to the kind of over consolidation it has undergone then probably the dilation may not happen in the field while loading it is not clear.

So, there is a general tendency that we tend to neglect cohesion for design. So, what I mean to say is peak characteristics and the shear strength associated with peak cannot be considered as a fundamental property. To be very specific from whatever discussion we had earlier the dilation angle which is over and above the critical angle how much it would be it is condition specific.

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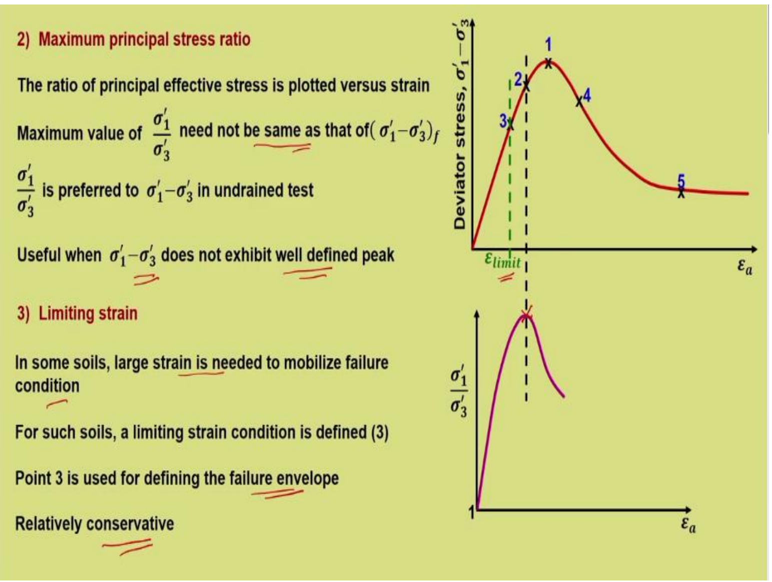

Now, the maximum principal stress ratio, a very useful way of representing stress strain behavior and obtaining the failure envelope for especially for un-drained case. The ratio of principle effective stress is plotted versus strain. You have seen this in the previous slide. Maximum value of σ_1'/σ_3' need to be same as need not be same as that of $(\sigma_1' - \sigma_3')_f$.

What it means is that the peak value that you obtain from deviatoric stress which is this one may not be same as in the case of σ_1'/σ_3' and this is very true for un-drained case. Why? So, this is what it is. Here the peak happens to be at this particular point that is corresponding to point 2. And it is different from that of point 1. Now, this is preferred to σ_1'- σ_3' in un-drained test.

If you have noted deviatoric stress σ_1- σ_3 = σ_1'-σ_3'. So, it is not going to make much difference even if you measure in terms of pore water pressure the un-drained testing. But σ_1' and σ_3' will be exclusively different. So, that is the reason why σ_1' /σ_3' will be more reliable in the case of un-drained testing.

It is also useful when the deviatoric stress does not exhibit a well-defined peak. So, next one is limiting strain. So, what happens is in some soils large strain is needed to mobilize the failure condition or rather it is very difficult to identify the failure condition. For example, in the case of normally consolidated soil, it goes on increasing. So, it goes on densifying. So, where to stop the test during triaxial testing? So, we need to have some criteria.

And these are also very well explained in the code of practice as well. So, there is a limiting strain based on which we can terminate the testing. So, that is what is meant by limiting string. So, that is the point 3. For such soil, the limiting strain is defined by point 3. And this can be any specific value or the value which is coated in the code. For example, in IS code also there is limiting strain specified. In ASTM also, it is specified. Both are different.

One is close to 10 %, the other one is 20 %. So, it is it or it can be any sort of limiting strain which the engineer finds it appropriate for that particular condition. Whatever be the testing is stopped corresponding to a limiting value of strain. Point 3 is then used for defining the failure envelope. Now, if it happens to be slightly lower then what we are ensuring is whatever stress that is developed within the soil will be close to the elastic limit.

Depending upon what elastics what limiting strain that we are fixing that will depend that will determine what sort of failure envelope that we are taking. Because now all these stresses will be plotted with respect to this particular point 3. What does that mean? If this stress what we are considering is fairly less then it means that we are shifting our failure envelope downwards. That means we are limiting we are becoming more and more conservative.

We are not allowing the soil to fail beyond a certain point. So, that means that all our design will also get influenced by this. Yes, there is a trade-off between the conservative result and the economical part of it. But this particular aspect when you consider limiting strain is if it is too conservative with the limiting strain then the failure envelope that we get also will be fairly conservative. That is what it is written.

It is relatively conservative. And it can it will be more conservative depending upon the limiting starin.

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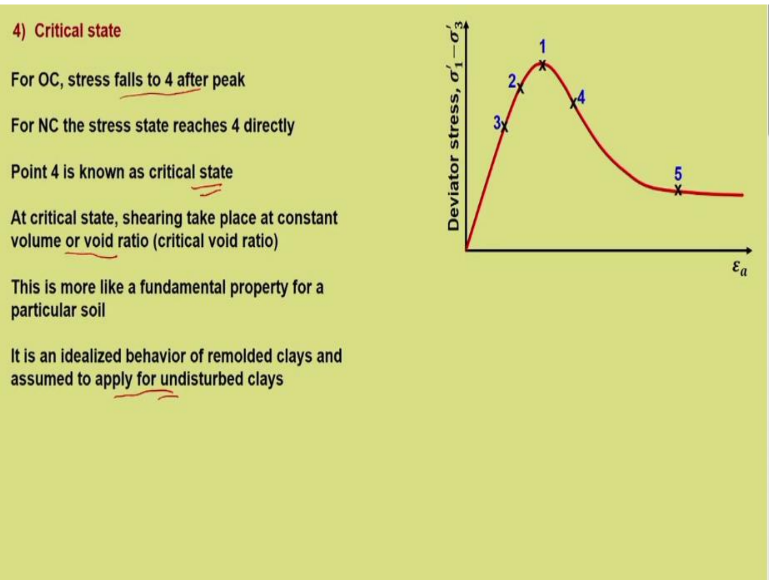

Critical state is the fourth criterion and this we have seen quite a number of times as we discussed in our previous lecture. For OC, the stress falls to 4 after peak. That is strain softening behavior. For NC, it reaches 4 directly. And point 4 is known as the critical state. Now, we will be discussing at length this concept of critical state in the fourth module. But for our quick understanding, at critical state, shearing takes place at constant volume or void ratio.

So, it is the state which the soil achieves during shearing such that the volume does not change further but deformation happens. Deformation happens at constant volume. So, that state is known as critical state. It is more like a concept. And hence it is also more like a fundamental property for a particular soil. This we have seen a bit of it in our earlier discussion. We have drawn the failure envelope and the peak is over and above this critical.

So, for any soil, this particular inclination that is 5 critical angle of internal friction corresponding to critical state becomes more or less a kind of fundamental characteristics. It is an idealized behavior of remolded clays. And it is assumed to apply for undisturbed clays as well. In short it applies to most of the kind of soils but it was mostly idealized for remolded clays. And it is assumed that it behaves well even for undisturbed clays as well.

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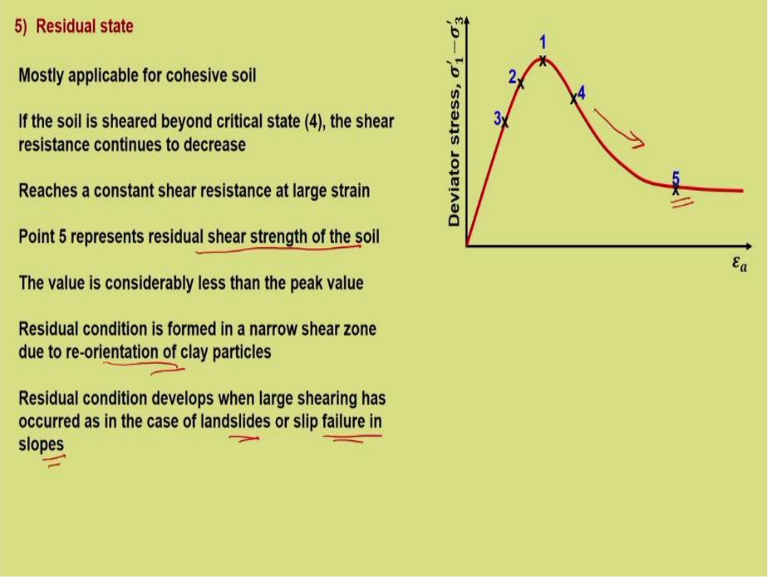

Now, the fifth one which we have not discussed till now is the residual state. You can see that the residual state that is the fifth one is fairly different from all other 1, 2, 3 and 4. It is mostly applicable for cohesive soils only. The reason is you can see here residual state corresponds to very large amount of strain. That means the soil has already undergone a large amount of strain and it is almost at the verge of failure.

So, the amount of shear strength which the soil mobilizes at a very high strain is what is known as residual strength. And how much of it, it mobilizes? It is a very tricky and a complex situation. But what it means is that it is a very low shear strength which is mobilized in a soil corresponding to a large strain or large shear displacement. That is what is meant by residual state.

If the soil is sheared beyond critical state the shear resistance continues to decrease. So, that is what it is shown here. It reaches a constant shear resistance. So, at 5, it more or less becomes a stable value at large strain beyond which it is more or less going to be either fail at that particular point. So, it almost reaches a constant value. Point 5 represents residual shear strength of the soil.

This value is considerably less than the peak value. Now, residual condition is formed in a narrow shear zone due to reorientation of clay particles. So, it mostly it is associated with clays and as example of such residual state is in the case of slope stability where there is failure planes which are formed or slip lines which are formed. So, the residual condition develops when large shearing has occurred as in the case of landslides or slip failure in slopes.

In the case of landslides, there is a continuous movement before the actual failure happens. So, it has already undergone sufficiently large strain but it has not failed. So, whatever is the shear strength that the soil possess during that particular time. That is not time that particular strain. That is what is meant by residual strength. Now, determining residual shear strength is totally different from what we do in the conventional triaxial testing.

It is very difficult to withstand the failure of a soil in conventional triaxial testing. So, there are some special instrument which are used for determining residual shear strength like a ring shear test where a large strain can be induced into the soil without failure. So, those type of testing will help us to obtain the residual shear strength and which corresponds to large movements like that in landslides and in slope failure.

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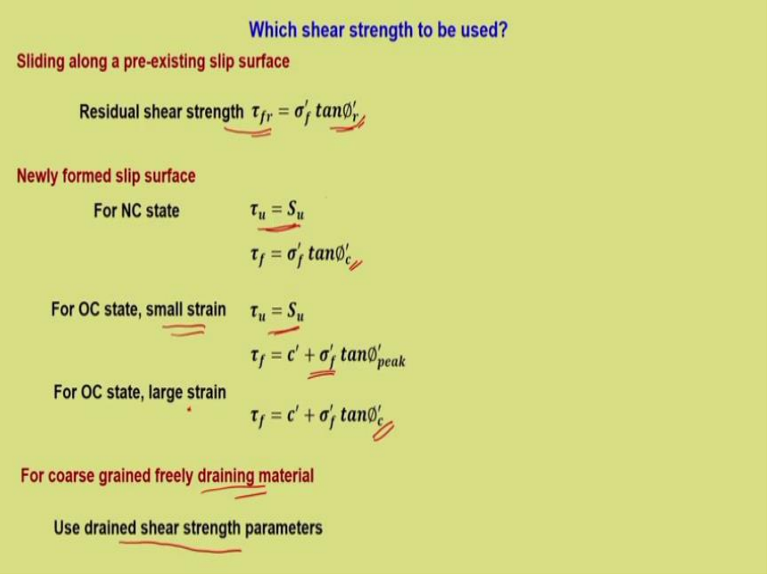

So, now we have more or less finished like the whatever shear strength aspects which we need to discuss. Now, after knowing all these things, what to apply where is an important question. Again this discussion is not conclusive because the amount or the kind of possibilities that we have in the field is innumerous. It is very difficult to pinpoint each of these possibilities in a course.

So, a broad understanding of what to use, where, I will discuss in brief. Sliding along a pre-existing slip surface. So, it is very clear like already existing failure surface or slip surface and the soil is moving. In that case, what shear strength to use? So, based on our discussion today we can easily say that it will be residual shear strength. That is what residual shear strength $\tau_{fr} = \sigma'_f \tan \phi'_r$. r corresponds to residual strength

See please note here, r stands for residual. Now, how do you get ϕ'_r? We get ϕ'_r by considering the deviatoric stress corresponding to residual state from the previous stress strain diagram. That deviatoric stress value is used for plotting the Mohr circle and obtaining the ϕ value. Then that ϕ value corresponds to residual strength. Newly formed slip surface that is it is just about began to slip or the failure plane is just forming.

So, in that particular situation, which value we need to adopt? Now, this will depend on what type of stress history or the initial state the soil has. So, for a normally consolidated state and for short term strength we will use the un-drained shear strength S_u. Please note we are not using ϕ_u. For any un-drained case, for all practical purpose, we use S_u, un-drained shear strength for NC, for short term strength. And for long term, it will be $\tau_{fr} = \sigma'_f \tan \phi'_c$.

Please note here, it is critical state value because for normally consolidated, we do not have the kind of peak. For OC, if it is small strain then for un-drained it remains the same whether it is large or small $\tau_u = S_u$. And it is $\tau_f = c' + \sigma'_f \tan \phi'_{peak}$. Please note here it is a small strain problem. We know that the peak condition is always with respect to smaller strain.

And at larger strain, its strain softens and reaches the critical state. So, this is what we have already seen. So, that information is very vital in appreciating this particular point. For long term if you have a small strain problem it is conveniently we can use the concept of peak shear strength, a ϕ'_{peak}. And that is what is written here. And in the case of long strain, S_u remains the same. It is not sensitive to this.

So, in the case of large strain, we can see that it is convenient to use ϕ'_c because by the time it reaches a large strain it would have crossed the peak and strain softened to the critical state. And for coarse grained soil is a freely draining material it is always we are using drained shear strength parameters that is ϕ', ϕ'_{peak} or $\phi'_{critical}$. So, it depends upon the denseness of the soil.

So, that is all about which shear strength to be used. Now, let us summarize. Like we have now gone through most of the important aspects of shear strength. What are the different factors? Now, when you start any lecture normally these factors are discussed in the beginning like what are the different factors affecting the shear strength. But here I have chosen this to be discussed in the end. Because many of these factors if discussed in the beginning it is very difficult to appreciate.

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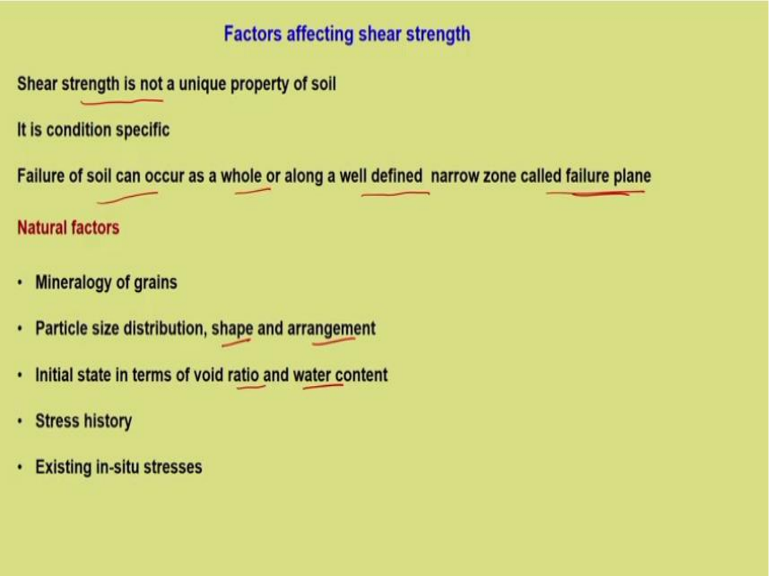

So, here what are the different factors? There are 2 sets of factors. First of all before discussing the factor let us make it clear that shear strength is not a unique property of soil it is condition specific. And this we have discussed at length throughout this module. And it is very clear now. Failure of soil can occur as a whole or along a well-defined narrow zone called failure plane. Now, the factors are classified as natural factors and induced factors.

So, let us first see the natural factors, mineralogy of grains because the clay behavior itself is determined by mineralogy. And it is very easy to understand also. For example, an expansive clay behaves different from a non-expensive kaolinite kind of clays. Particle size distribution, shape and arrangement. So, shape of the particle very easy to understand. Particle size distribution how distributed it is and that distribution will determine how it is arranged when it is compacted.

So, all these are going to determine what is the shear strength. Now, this is a property which is very relevant for soil type of materials because you cannot say that shear strength of soil is this.

Because it is also dependent on how you are packing it a loose soil or a dense soil. So, for the same soil you have different strength depending upon particle size and its arrangement.

Initial state in terms of void ratio and water content very important because that determines again how compacted the soil is and compaction, it induces strength into the soil. Stress history, we have seen this very clearly. So, if this I have told in the beginning it is very difficult for you to understand. Now, since we have gone through the entire aspects of interpretation in terms of normally consolidated, over consolidated, it is easy to appreciate how stress history would affect the strength.

Existing in situ stresses, so, when there is a kind of undisturbed sample what kind of stresses it was already subjected to that also it is a part of a bit of stress history itself. But, what is the in situ stress condition that available in the field? That also determine how much strength it can mobilize.

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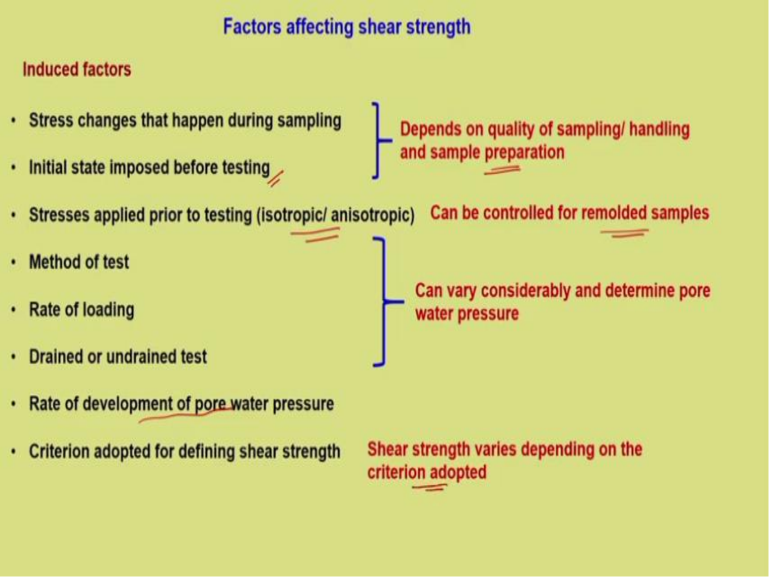

Induced factors. So, what the whatever we have discussed is by its own we are not doing any additional aspect for that will affect the shear strength. So, induced factors includes stress changes that happens during sampling. We have again seen this with an example what causes during sampling. Initial state imposed before testing, this also it is governed by us what type of initial state do we tend to give.

May be it is an isotropic, anisotropic consolidation. So, all these possibilities are there. So, this depends on the quality of sampling, handling and how we are preparing the sample for the test.

So, this is exclusively dependent upon how we do the test. Stresses applied prior to testing, it is more or less similar to that of initial state and whether it is isotropic, anisotropic, it can be controlled for remolded samples.

Method of test, what type of test and sequence we have seen all these factors indirectly throughout the lecture. And we have seen how that is going to influence the final strength. Method of test, rate of loading, drained or un-drained test, these need does not need any sort of discussion at this stage but we are just summarizing it. Can vary considerably and determine the pore water pressure in the case of un-drained test.

What is the rate of development of pore water pressure? The criterion adopted for defining shear strength. So, that is what we have just discussed. So, that also will determine what type of failure strength and failure envelope that we are referring to. Shear strength varies depending on the criterion adopted.

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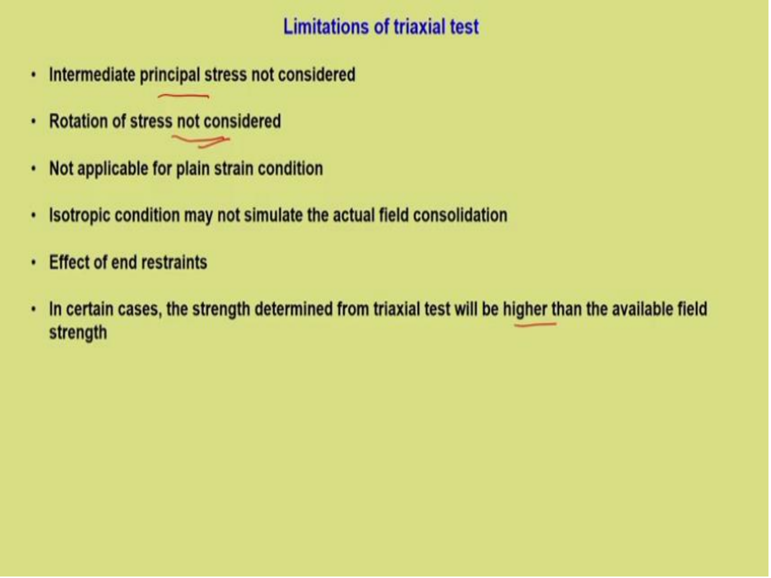

So, that is all related to what are the factors affecting the shear strength. Next is limitations of triaxial test. In triaxial test, we do not consider the effect of intermediate principal stress quite obvious. Rotation of stress is not considered which is a common phenomenon. If you consider the slope and the slip line you can see that the principal stress rotation happens. Now, this is not considered in triaxial testing rather it is not possible.

Not applicable for plane strain condition, we have already seen that plane strain is a very common condition while dealing with problems related to soils but triaxial testing conditions are different from that of plane strain. So, but we still tend to apply ϕ or c that we determine from triaxial to any condition. But we need to keep in mind that it is different and the kind of strength development also will change.

A very good example is the kind of pore water pressure equation that we obtained. For plane strain, it is different from that of axisymmetric triaxial condition. Isotropic condition may not simulate the actual field consolidation. Now, this is with respect to the conventional test that we conveniently do. We do always isotropic testing. Now, there are ways of doing anisotropic testing as well but not very common.

We need to keep in mind that this may not actually simulate the field condition. Effect of end restraints, this we have also discussed in the previous one of the lectures where the pore water pressure development may not be uniform within the sample because of this cap end restraints. Now, for that we need to be careful while loading. So, that we have already seen. So, the effect of end restraints remains in the case of triaxial testing.

In certain cases, the strength determined from triaxial test will be higher than the available field strength. A very good example is the isotropic consolidation. So, there it will be different. Strength that we get finally will be different.

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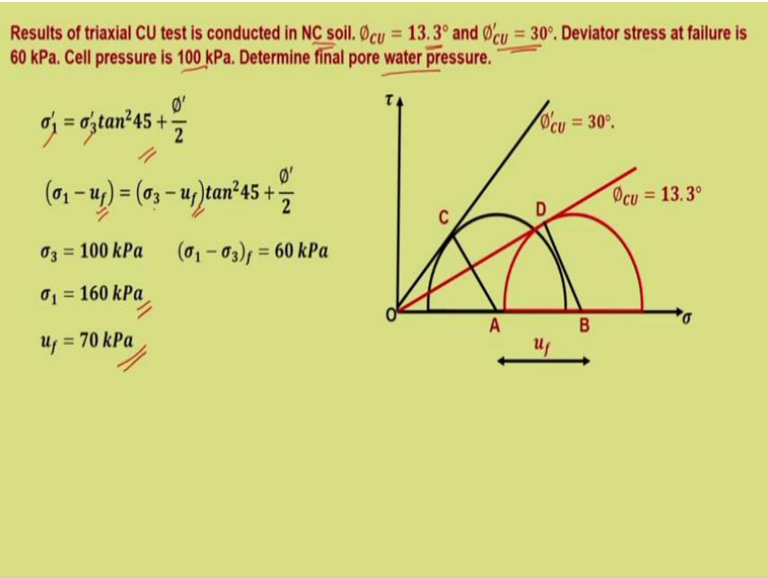

With that we finish the discussion and the additional aspects. We will just see a very small brief problems just to make sure that what is the influence of pore water pressure on the shear strength. So, the first problem is results of triaxial CU test is conducted in NC soil. $\Phi_{CU} = 13.3$

degrees and ϕ'$_{CU}$ = 30 degrees. Deviatoric stress at failure is given 60 kPa. Cell pressure is given as 100 kPa.

We need to find out what is the final pore water pressure. Now, if you don't understand that for an NC soil it passes through origin then we will struggle at this point thinking what will be the cohesive component. So, that is very clear like for an NC soil, we do not have cohesion. So, this is the kind of problem which has been given. We are asked to find out what is the final pore water pressure. So, it is a very simple problem.

We just need to substitute in the Mohr Coulomb failure envelope. This is not given. σ_1' , σ_3' is not given but σ_1 and σ_3 can be determined. σ_3 = 100 kilopascal. So, σ_1 - σ_3= 60 kPa. $\sigma_1$can be determined which is 160 kPa. Now, substituting that into the equation, only unknown is u_f.

$$(\sigma_1 - u_f) = (\sigma_3 - u_f)\tan^2(45 + \frac{\phi'}{2})$$

So, ϕ' is also given which is 30 degrees. So, if you substitute that you get u_f=70 kPa.

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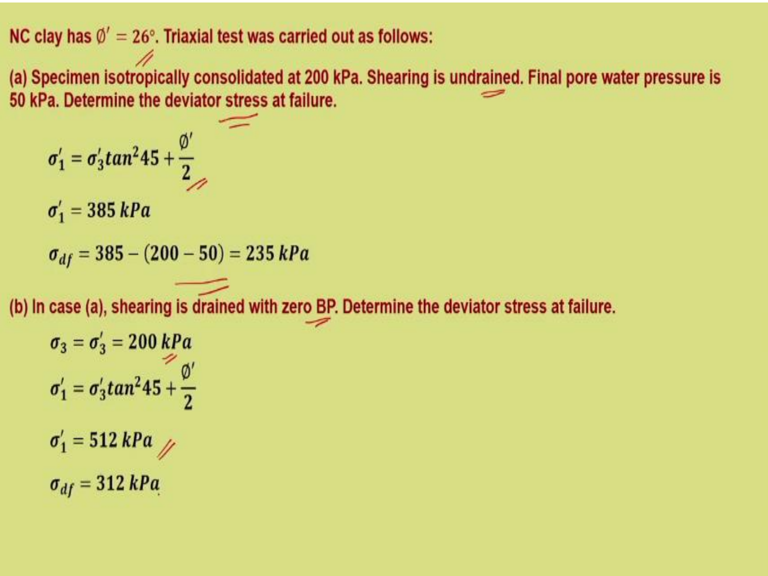

Another problem that is normally consolidated clay has $\phi' = 26^0$. Triaxial test was carried out as follows. We are discussing 3 different cases. The first case is specimen is isotropically consolidated at 200 kPa. Shearing is conducted as un-drained. Final pore water pressure is 50 kPa. Finally, deviatoric stress at failure, it is a straightforward question.

$\sigma_1' = 385\ kPa$. We have 200 kPa and the final pore water pressure is given. So, we can always determine what is the kind of the deviatoric stress at failure.

$$\sigma_1' = \sigma_3{}'\tan^2\left(45 + \frac{\phi'}{2}\right)$$

So, $\sigma_1{}' = \ 385\ kPa$ and $\sigma_{df} = 385 - (200 - 50) = 235\ kPa.$

So, ϕ' is given. Substitute it, get the answer for $\sigma_1{}'$ then you can directly get what is σ_{df}.

Now, in the same case a, let us say that shearing is drained with zero back pressure. So, what will be the deviatoric stress at failure? So, again it is $\sigma_3 = \sigma_3$' = 200 kPa because now pore water pressure is not there and there is no back pressure as well. So, $\sigma_1{}' = \sigma_3{}'$. Same equation we will get $\sigma_1{}' = 512 kPa$. And $\sigma_{df} = 312\ kPa$.

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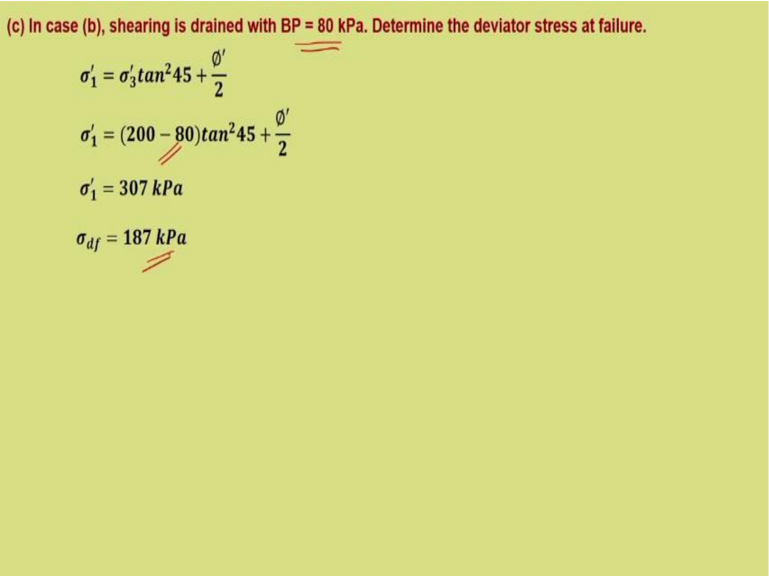

In the case 3, shearing is done but there is a back pressure of 80 kPa. So, what will be the deviatoric stress? Same procedure, only thing is here $\sigma_3{}'$ changes. You need to find out what is $\sigma_3{}'$. And $\sigma_1{}' = 307\ kPa$ and $\sigma_{df} = 187\ kPa$.

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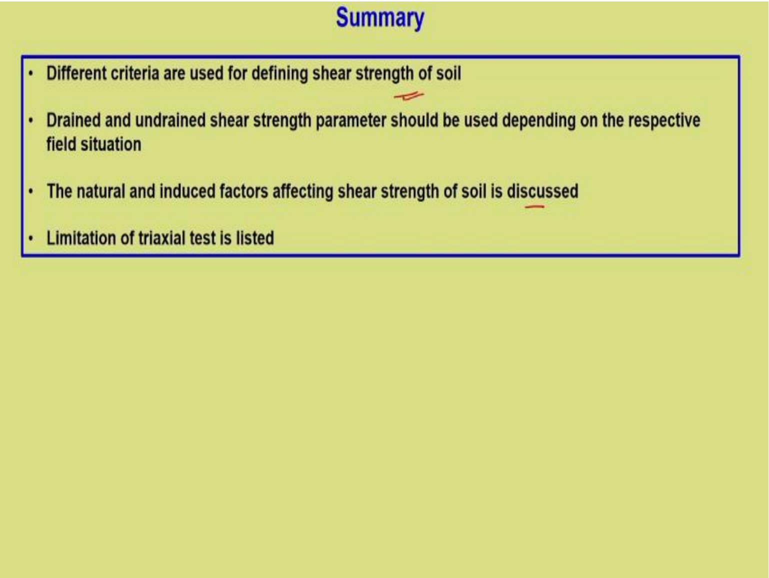

So, that is all with this particular lecture. Let us summarize what we have seen in today's lecture. Different criteria that are used for defining the shear strength of the soil is discussed. Drained and un-drained shear strength parameter should be used depending on the respective field condition. So, that also we have discussed briefly. The natural and induced factors affecting soil shear strength is discussed. And the limitation of triaxial test is listed.

So, that is all with this particular module. In the next lecture, we will summarize what all we have learned in Module 2. That is all for now. Thank you.