Soil Mechanics
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Lecture – 48
Shear Strength of Soils
Lecture No.6

Students we go forward now to the sixth lecture on the topic of shear strength of soils. As per my habit I will first try to recollect what we have discussed in the previous lectures, especially the last fifth lecture before we go on to the topic of today's lecture. So to start with, in the last lecture I had mentioned that we have divided this topic into 3 aspects.

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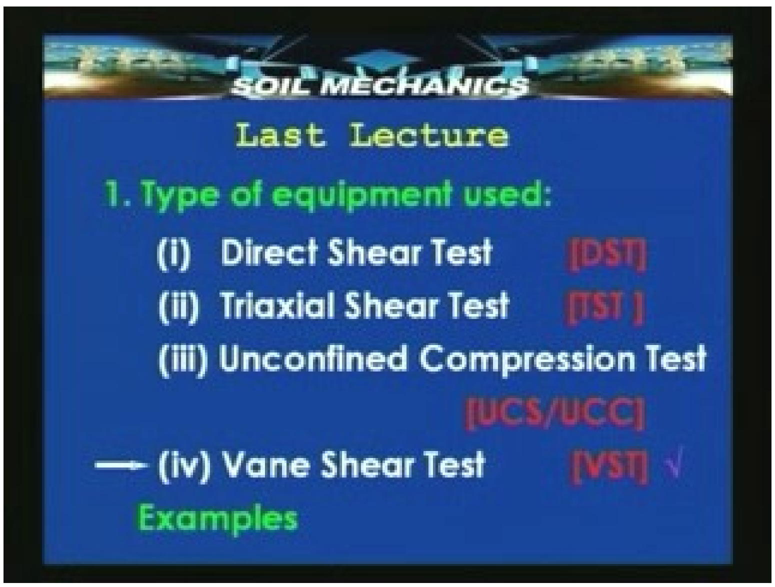

The aspect being the type of equipment used and the kind of test that are done to determine the shear strength of soils. Out of those we had discussed in detail the direct shear test, triaxial shear test, unconfined compressive strength test and in the just completed last lecture we saw in detail the vane shear test and then an example on how to determine the vane shear strength from the results of a typical vane shear test.

Then we went on to the second topic or second subtopic within this main topic of shear strength that is drained and undrained test. That is the importance of drainage and how the tests are conducted either with or without drainage and what is their significances. And let's recapitulate as usual very briefly what we did in this 2 sub topics.

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First the vane shear test, as I mentioned last time a vane is a device which has 2 vanes like this, two blades or vanes like this which are attach to a steel rod which can be rotated. There is a handle that is a torque can be applied. It has got dimensions of H and d as shown. The values of d and H usually vary in this range in the field device whereas in the miniature lab device the values are much smaller, d is only 12.5 mm and H the height is 25 mm.

This device is immersed or pushed into the soil gently and a torque is applied and the device is rotated. When the device is rotated it produces a cylindrical failure surface in the soil. Based on the torque required to produce this failure surface we are in a position to calculate from the geometry of the failure surface, the unit shear resistance offered by the soil which is nothing but the shear strength of the soil.

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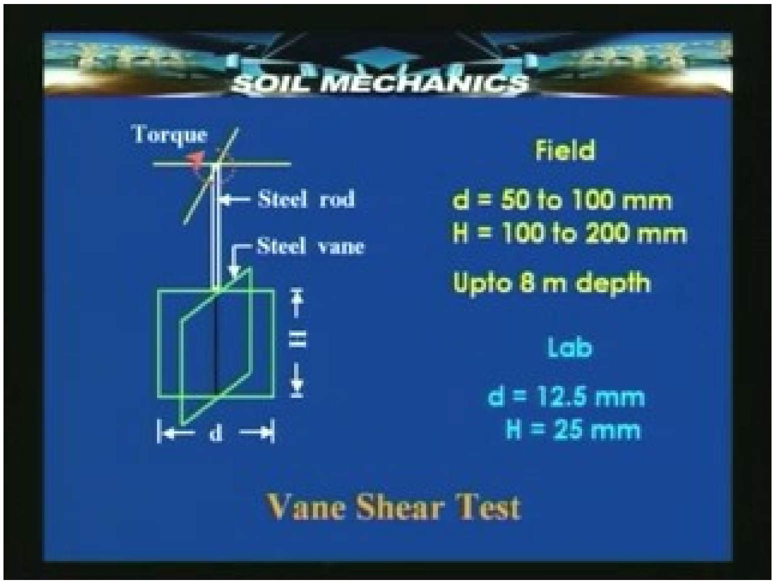

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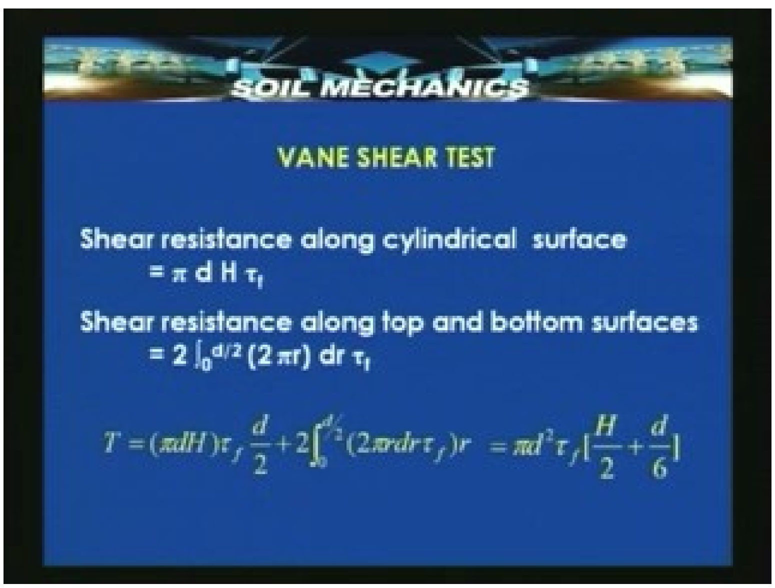

So we have a formula which computes the shear resistance along the cylindrical surface of the soil. The surface which has been cut by the vane and the resistance experienced by the vane at the top and the bottom surfaces. These two together contribute to the total shear strength which must be overcome by the torque so that failure can be produced inside the soil. So what is the moment of these forces and the sum of these moments and calculate that and put it equal to the torque, that will give you a relationship from which the unit shear resistance tow$_f$ at failure, shear strength at failure can be calculated and that's nothing but the shear strength. We had seen that there are some corrections that need to applied depending upon what kind of distribution of shear resistance we assume at the top and the bottom and also depending on whether the vane is fully immersed in the soil or not.

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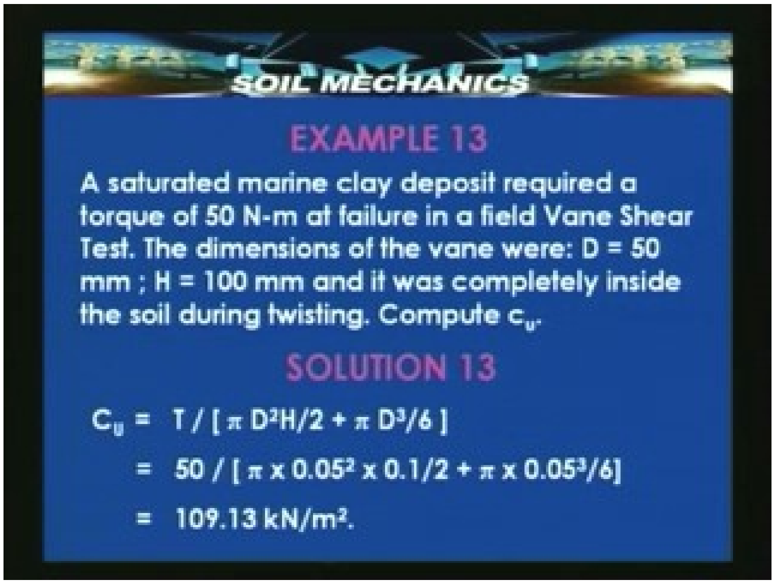

We saw a simple example where the undrained cohesion of the soil was computed from the torque and the geometrical dimensions which were available to us in this problem.

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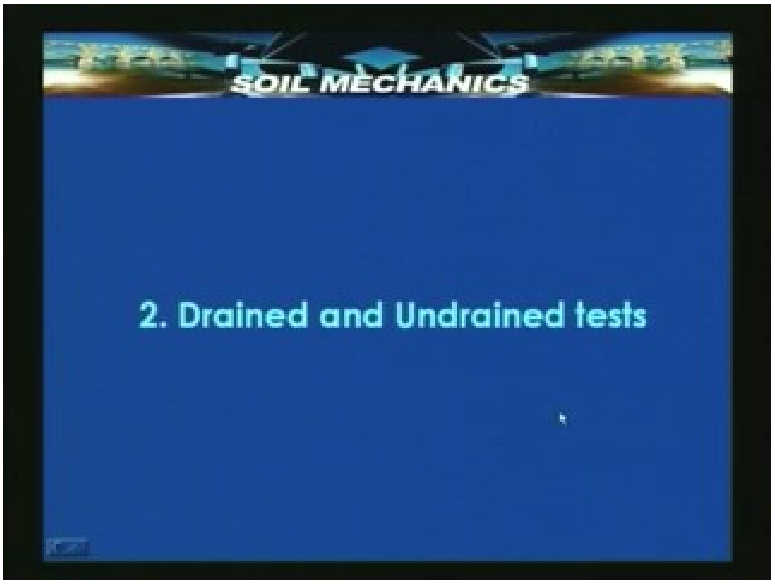

Now let us go to drained and undrained test. Let us take the basic scheme which we used last time to understand the importance of drainage. Here is a typical triaxial test specimen, it is subjected to two stages of loading as we know in the conventional triaxial test, I must mention here at this point of time, there are several different types of triaxial test.

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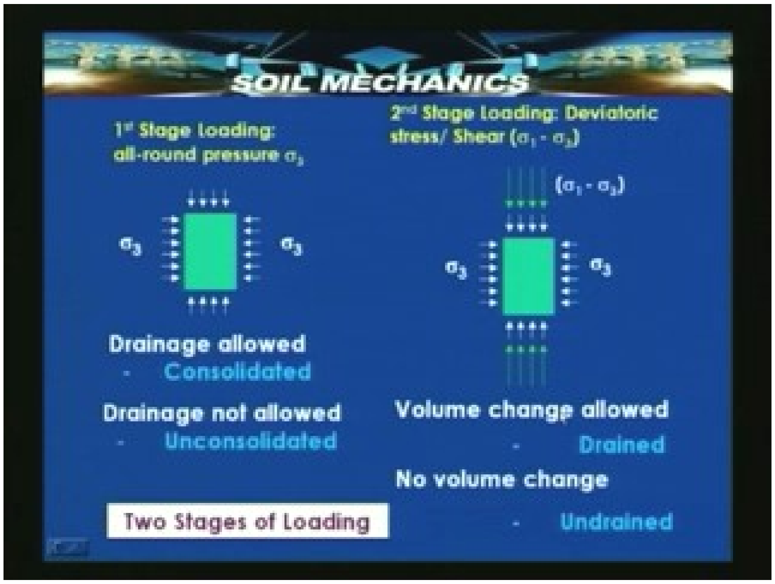

The one which we normally use is the one which is called the CTC or the conventional triaxial compression test. The CTC test for example involves applying a uniform lateral pressure sigma$_3$ in the water that surrounds the specimen in the test cylinder. And then this pressure is in the second stage of loading increased to a value sigma$_1$ by adding what is known as the deviatoric stress sigma$_1$ - sigma$_3$. Thus the second stage of loading leads to the specimen experiencing a lateral pressure sigma$_3$ in the horizontal direction and a net axial pressure or a total axial pressure sigma$_1$ in the vertical direction and while applying these stresses we can either allow drainage or prevent. If drainage is allowed there will be a volume change and consolidation will take place. That is we are assuming that the soil is saturated.

If on the other hand consolidation is not permitted that is drainage is not allowed then the soil remains as unconsolidated in the first stage of loading. And then when we go on to the second stage if we allow volume change through drainage then that’s known as drained test whereas if you do not allow any volume change, if you prevent the drainage then it is called the undrained test.

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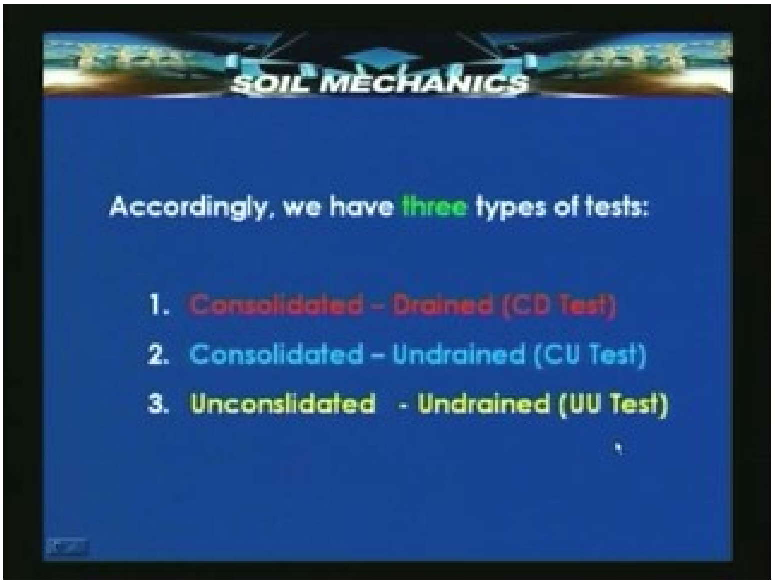

We have three combinations, well known combinations of test. The consolidated drained, consolidated undrained, unconsolidated undrained. The first is commonly called the CD test, the second one the CU test, the third the UU test or sometimes these are also known as the S slow test and the Q the quick time test.

Let us take a look at the mechanics of the consolidated drained test quickly. Here is this table which we discussed last time which shows the pressure variation with the loading. Here for example when the confining pressure is applied the total stress is equal to the confining pressure in both direction because this specimen is surrounded by water and water transmits pressure equally in all directions. Water does not take any pressure because drainage is permitted, water is allowed to be expelled, pore pressure or neutral stress is zero.

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Effective stress is simply equal to the applied stress sigma$_3$ in both directions. So let us take a look at the mechanics of the first type test that is the consolidated drained or CD test. Of course we had seen it last time also but let us quickly recapitulate. There are two stages of loading as we had seen last time and the stresses vary through out the test as shown in this table. First stage of loading involves application of the confining pressure sigma$_3$, when the confining pressure is applied the total stress becomes equal to the confining pressure both in the horizontal and the vertical direction because water transmits pressure equally in both directions.

On the other hand since drainage is permitted water itself does not take any pressure and the neutral stress u remains zero both in the horizontal and the vertical direction and the effective stress which is the difference of these two becomes equal to simply the applied sigma$_3$. On the other hand when the axial load is applied in the next stage, the applied pressure gets added to the total stress in the vertical direction because no pressure is applied in the horizontal direction. The neutral stress still remains zero because during shear drainage is not prevented. The effective stress therefore becomes the difference of these two equal to sigma$_1$ dash equal to sigma$_3$ plus delta sigma$_1$, sigma$_3$ dash equal to sigma$_3$.

Thus whatever pressure we apply initially in the first of stage loading in the lateral direction that remains effective even at the end of the test whereas the vertical effective stress becomes equal to the applied lateral pressure plus additional axial load delta sigma$_1$ which we apply. In other words all the load that is applied becomes effective in this test because it’s a test in which consolidation is permitted and drainage is permitted or volume change is permitted during shear.

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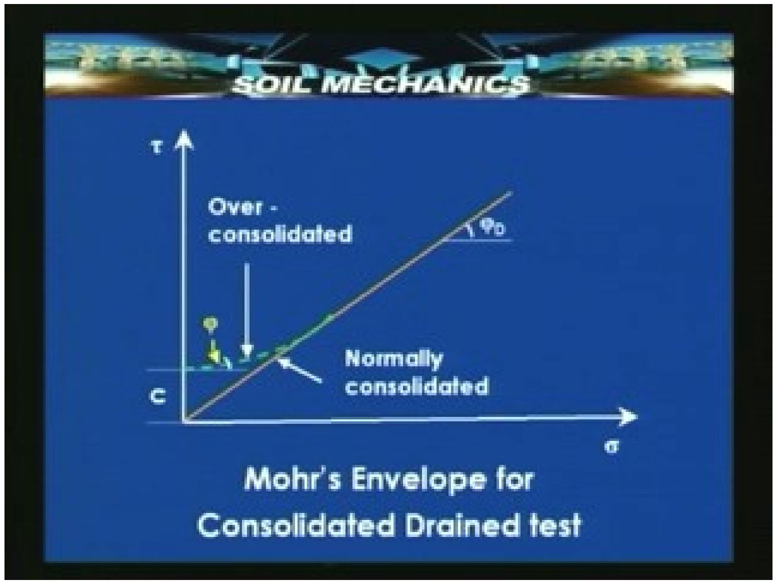

The Mohr diagram for this looks somewhat like this. This is the typical Mohr envelop which you see here being a drained test for reasons which I explained last time, the envelop will pass through the origin if the soil is normally consolidated. Because the very concept of normally consolidated soil implies that in its initial formation stage when there is no pressure sigma, it will have no strength, it will be a slurry or a fluid and it will have no shear strength but then as the stress sigma increases the slurry gets more and more consolidated leading to ultimately over a long period of time, a soil which is consolidated fully under the pressure that it has experienced.

If on the other hand the soil is overconsolidated that is it had experienced a pressure in the past which is larger than the pressure that is now applied then the envelop will not be passing through the origin because there will be a cohesion in such a soil at pressures which are less than the past overburden pressure but beyond that this soil is as good as normally consolidated soil for pressures above the old preconsolidation pressure and therefore it joins the envelop. And a drained test gives you c and phi_D, D standing for drain and these are effective stresses and therefore we can call them as c dash and phi dash.

The next type of test is the c_u test, here now if we want to study the mechanism first when confining pressure delta $sigma_3$ is applied, the total stress becomes $sigma_1$ equal to $sigma_3$, $sigma_3$ equal to $sigma_3$ just as in the previous test. Because here again consolidation is permitted in the first stage of loading. Pore pressure remains zero, effective stress becomes the applied stress, but the difference comes now in the second stage compared to the previous test. When the axial load is applied the total stress is of course becomes $sigma_3$ plus delta $sigma_1$ and $sigma_3$ in the lateral direction but since no drainage is permitted neutral stresses are non-zero. They are equal to the extra axial stress that we are applying that is delta sigma$_1$ in both directions because this pressure is taken up by water.

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And now the effective pressure is less than the effective pressure that existed before the application of this undrained load because during undrained load pore pressures are developed and therefore whatever preexisting effective stress was there at the end of the first stage loading, gets reduced due to this additional pore pressure. So sigma$_1$ dash become sigma$_3$, sigma$_3$ dash becomes sigma$_3$ minus delta sigma$_1$.

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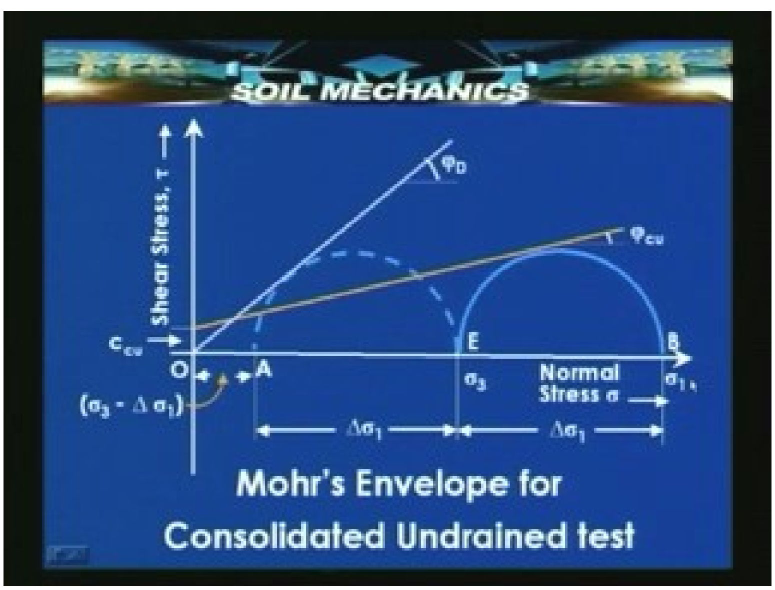

The corresponding Mohr's diagram if we take a look, the continuous line thick circle represents the total stress circle that is the total stress sigma$_3$ applied and the corresponding total stress sigma$_1$. If we know the pore pressure which incidentally is equal to delta sigma$_1$ that is the additional axial load applied that's which the pore pressure is known in this particular test.

If we subtract this pore pressure then we get the effective stress dotted line diagram, the Mohr circle represented in dotted lines which says that the effective stress at point A which is the minor principle stress is sigma$_3$ minus delta sigma$_1$ that at this point E, E is equal to sigma$_3$. Now if I draw an envelope to this drained Mohr's circle I will get a line an envelop passing through the origin with an angle phi$_D$ drainage, D standing for drains and cohesion equal to zero. But if I plot a tangent to a total stress Mohr's circle of course it is implied that we will be doing atleast three such test with a total stress measurements and plotting a common tangent. Here for convenience I have only shown one such Mohr's circle. If this were the common tangent to three such tests, this would pass through the y axis at a point giving an intercept equal to c$_{cu}$ where cu stands of consolidated undrained, the corresponding angle of friction would be phi c$_u$.

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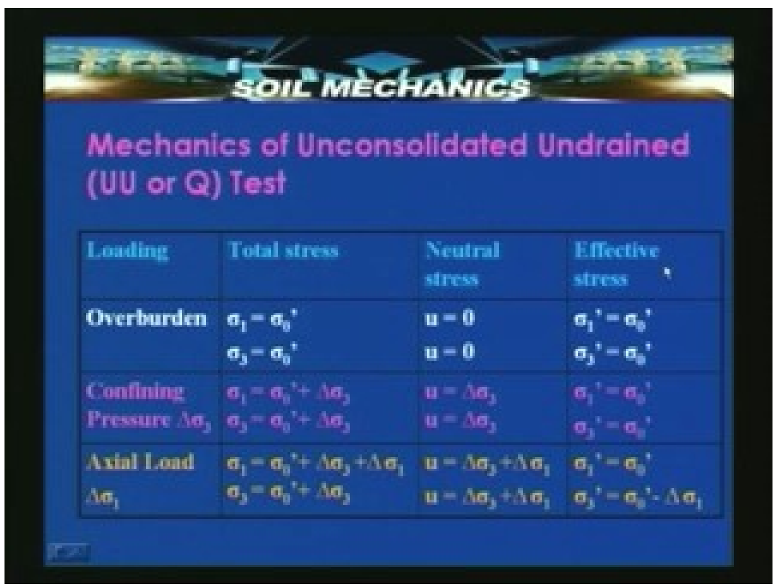

Next if we take the unconsolidated undrained test UU, here since even during the application of confining pressure, consolidation is not permitted. We are not allowing any effective stress to develop at all. Then what's the stress that's going to be there in the soil? It will obviously be the stress under which the soil has been formed that is the overburden stress, preexisting stress on the soil before the test, before the specimen is extracted and this test is conducted and therefore in this case in order to know the stress state at the end of the test. We have got to take the initial overburden effective stress that was present in the soil before the test was conducted. In fact the testing procedure is such that initially we subject the soil to a pressure equal to the overburden pressure and then we continue the test. So overburden pressure is let us say sigma$_0$ dash then that is the effective pressure sigma$_1$ and sigma$_3$, no neutral pressure initially and the effective stress is sigma$_0$ dash as shown here.

Now the moment we apply the confining pressure, total stresses become equal to this original sigma$_0$ dash plus the applied confining pressure. Pore pressure is the applied confining pressure obviously undrained conditions are prevailing. The effective stress therefore does not change; it is the same that was existing in the field. When you now apply the axial load delta sigma$_1$, it just gets added to the total stress in the vertical direction, no change in the total stress in the horizontal direction, pore pressure also increase because no volume change or undrained conditions are maintained. So u becomes delta sigma$_3$ plus delta sigma$_1$ in both directions. The effective stress in the vertical direction remains as sigma$_0$ dash therefore whereas in the horizontal direction it reduces by an amount equal to delta sigma$_1$ the pore pressure that is generated by the axial load.

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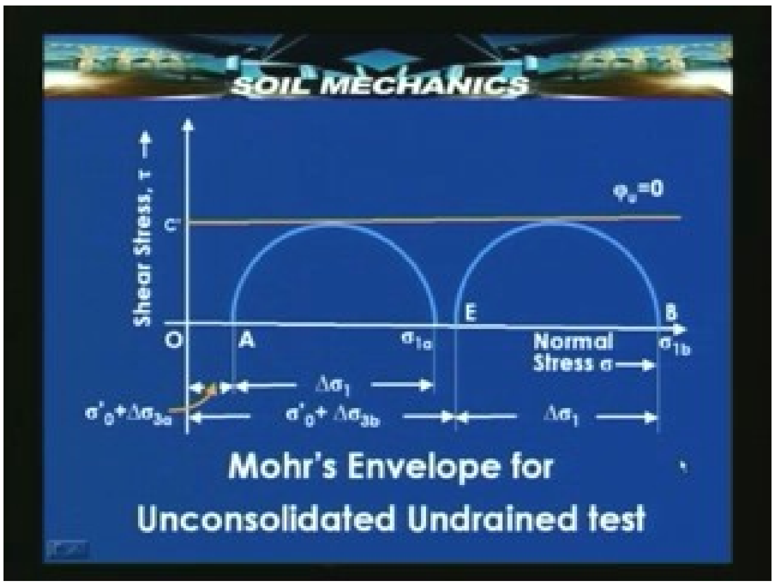

If we were to plot a Mohr's diagram based on this then the Mohr diagrams will be shifted from each other depending upon the sigma$_0$ dash value and delta sigma$_3$ that we use but the diameter of the Mohr circle will remain all the time same. That is equal to the axial load delta sigma$_1$ that is applied. Therefore here we have two circles corresponding to 2 different initial confining pressures applied, their common tangent will make an angle phi undrained equal to zero and giving a cohesion c undrained equal to the intercept on the y axis and this is what an unconsolidated undrained test and its Mohr diagram look like.

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Now let us go on to the subject matter of today's lecture. In today's lecture we shall be proceeding further and seeing some more details about the drained and undrained test and also take up a few examples. And we will also see the third aspect that I was planning to cover in this series of lectures. That is test on cohesion less and the cohesive soils. Now the next slide talks about the practical significance of the drained and undrained condition that are used in the test.

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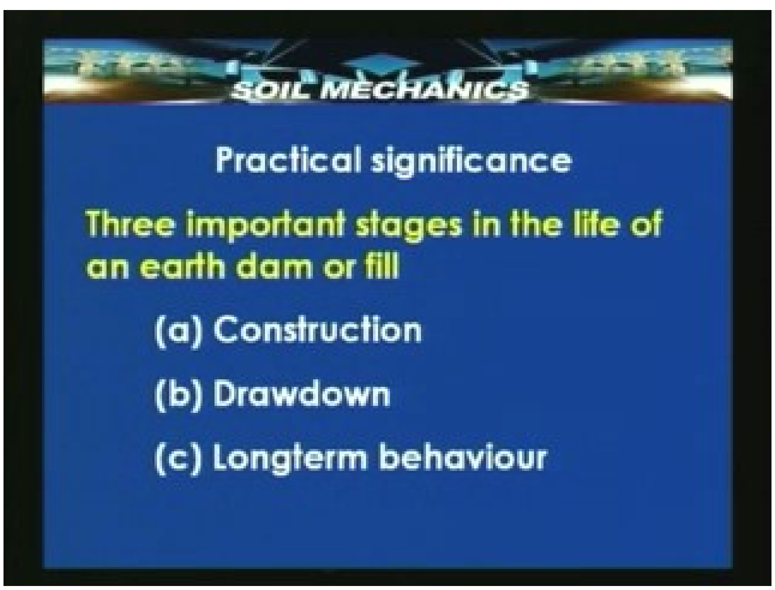

The practical significance of drainage can be very nicely understood from the example of the lifetime performance of an earth dam or a fill. So let us take the example of an earth dam. What are the major stages in its lifetime? First is the construction, as you know an earth dam is constructed by compaction layer by layer. And during this process of construction water is added to the soil and compaction is carried out and in this process some pore pressures are bound to be generated and therefore during the short term of construction period, if the pore pressures which are generated are high then even due to the self weight and its shear stress component in some specific particular surface of weakness could cause a slide or a failure of the earth dam. What I mean is suppose this is the earth dam which is being constructed layer by layer, if pore pressures are allowed to generate due to rapid construction and non-permission of water to expel then it quiet possible that the pore pressure so generated cause shear stresses along some potential failure plane which exceed the shear resistance of the soil and failure may take place.

Why does the shear resistance come down is because the pore pressure produce a net effective stress which is very small due to which the resistance that is created is also small. This kind of failure is a short term type of failure. On the other hand if you take a slightly longer duration of the earth dam, during its lifetime the reservoir gets filled or emptied depending upon whether it's a monsoon season or it's a reason for irrigation for cropping. So there is a seasonal variation of the water level behind the earth dam in the reservoir and if this water level goes down rapidly for any reason then again it is a converse of what happened in the during the construction. Again due to rapid drawdown there could be high pore pressure generated which might take time to dissipate and if they do not have the time to dissipate because of very low permeability of the soil then the effective stresses could considerably reduce.

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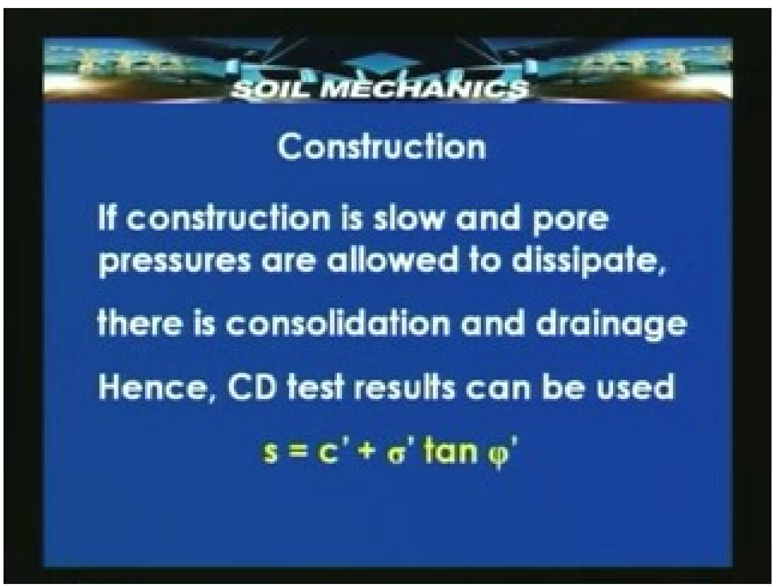

And the shear resistance of the soil may come down and there could be a plane, a potential failure plane along which a slide might occur. On the other hand the long term behavior stability and safety of the earth dam is perhaps the most important aspect and we have to ensure that the dam remains stable for the estimated or the design life of its span of performance. Let's take a look at what kind of shear strength test needs to be done, what shear strength parameters need to be used under these constructions, drawdown and long term conditions? It's interesting to note that each of these corresponds to a different set of drainage conditions and therefore a different test result has to be used. If construction is slow and pore pressures are allowed to dissipate then there is consolidation during construction and drainage also.

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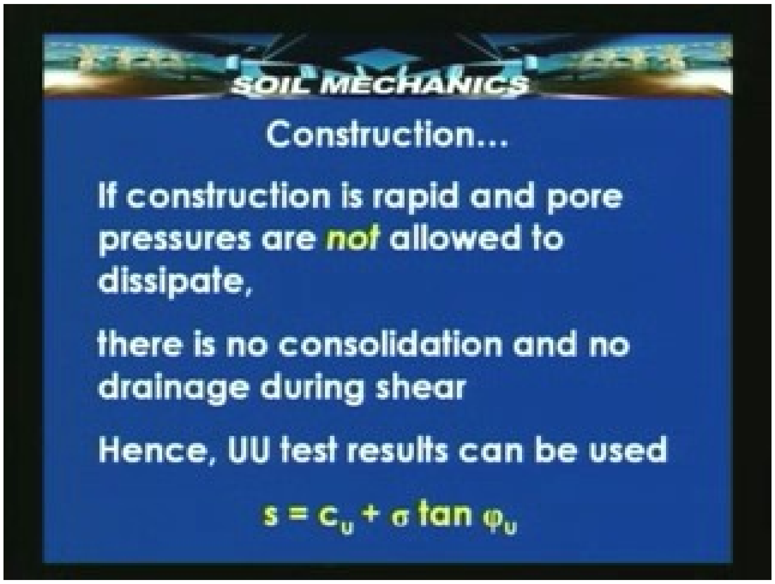

Hence it is a consolidated drained situation and CD test results can be used that is shear strength of the soil will be c dash plus sigma dash tan phi dash along any particular plane. If now consider the case of construction being done rapidly. Then pore pressures are generated and are not allowed to dissipate. There is therefore no consolidation and therefore no shear drainage during shear.

Ideal test for simulating this condition would be the unconsolidated undrained test. The results of this test are the most appropriate for use during evaluation of the safety of the dam during rapid construction. So s is equal to c_u plus sigma tan phi_u where sigma stands for the total stresses. Under the drawdown condition again pore pressure are not allowed to be dissipated.

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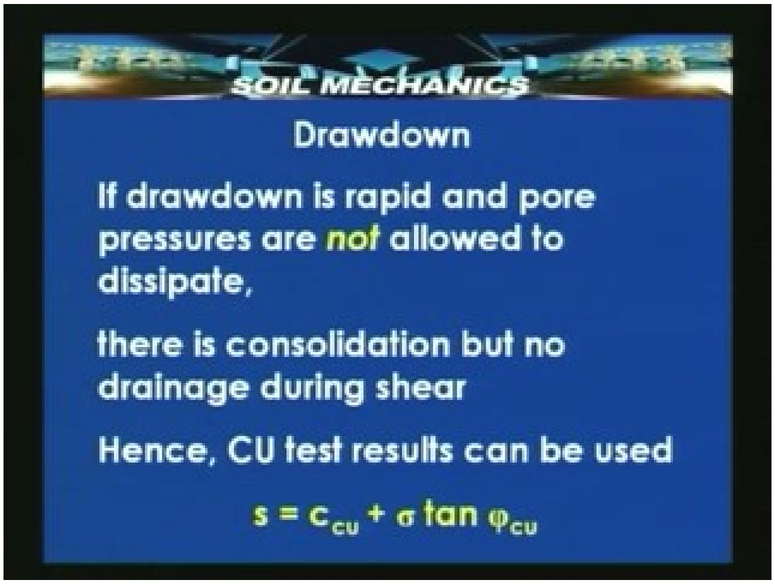

There is consolidation because sometime has passed since the dam has been constructed but there is no drainage during shear. Hence the consolidated undrained test would fit the requirement and shear strength will be c_{cu} plus sigma tan phi_{cu}.

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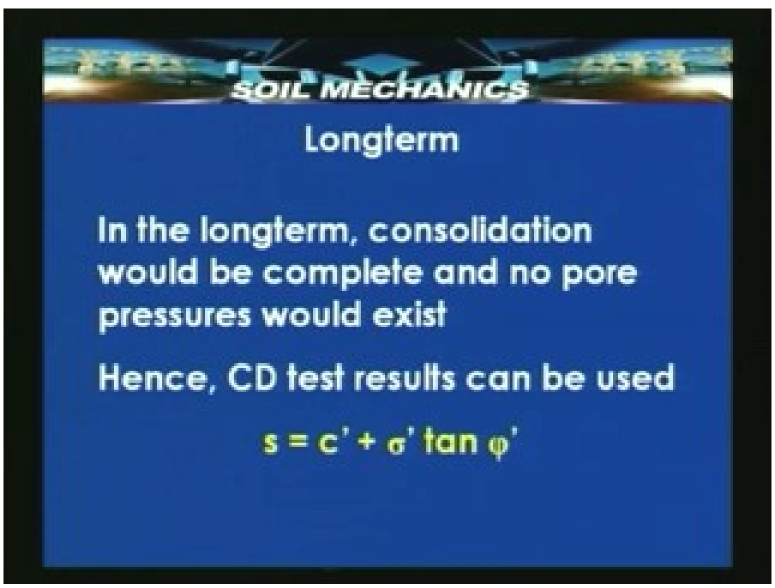

In this test as we had pointed out or seen a little earlier, the pore pressures are known. They are simply equal to the additional axial load applied during the undrained stage, the second stage of loading. Since the pore pressures are known although we get the total stresses and shear strength parameters corresponding to the total stresses, we can always easily the effective stresses and the effective stress parameters and use them. And in fact often this test is preferred to the simple consolidated undrained test. On the other hand the long-term behavior as I have mentioned already is a situation where the construction is complete. The pore pressures have been totally dissipated, the dam is now in a condition which can be equated to the totally drained condition that is consolidated drained and the CD test results again can be used as we did for construction period where slow construction was permitted, s is equal to c dash plus sigma dash tan phi dash.

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Therefore to summarize during the life of an earth dam, slow construction and long term stability would both involve determining drained shear strength parameters through the CD test. For sudden drawdown CU test and for rapid construction and the corresponding short term stability UU test would be appropriate. Let's see an additional comment on this aspects involving drainage. We have mentioned sometime back that undrained strength are usually determined only for clay soils which are low in permeability but actually in principle undrained test can be determined for all soils but they may not apply to sands they may not apply to all the soils though it is possible to determine that. What would apply perhaps to highly pervious soils like sands and gravels would be only the drained strength. So effective strength parameters can be used to check the stability at any time if the pore pressures are known. It is not necessary that we need to use only the total stress parameter's for a situation where the UU test results are required.

If we know the pore pressures at any point of time, the pore pressures are measured in the test then always the effective strength parameter can be calculated and they can be applied for computing the stability of the dam at any stage not necessarily at any particular stage. Now for sands and gravel because they are freely draining in nature, the effective strength parameters can be used even to check the short term stability because short term stability under rapid construction demands the UU test.

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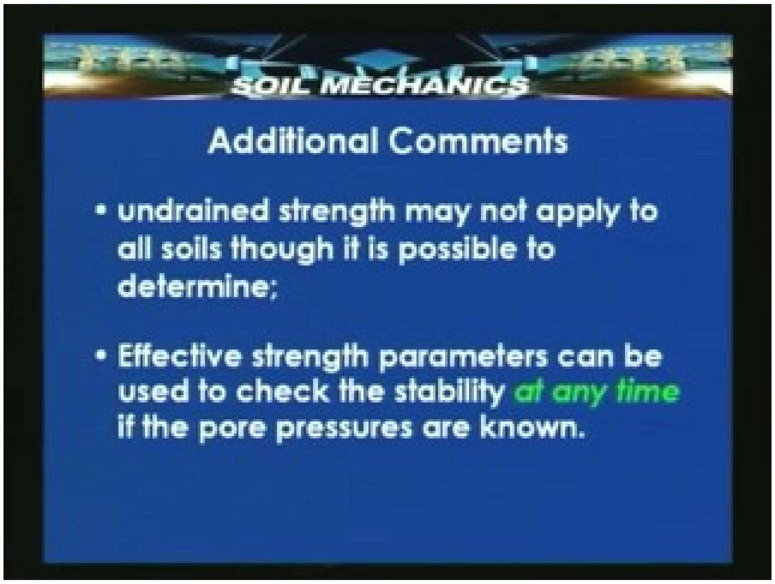

On the other hand if it is a pervious sand or gravel then effective strength parameters can still be used because pore pressures would not be allowed to generate. A total stresses analysis would not be therefore recommended for such soils.

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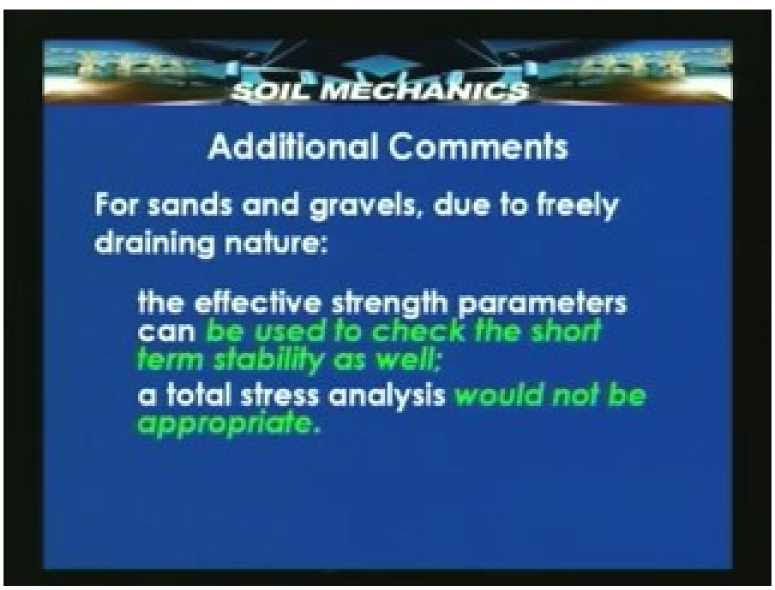

For clayey soils a total stress analysis is the only correct way to access the stability. Now there are a number of factors other than this drainage which affect the shear strength of a soil. Primarily we can identify 6 such factors.

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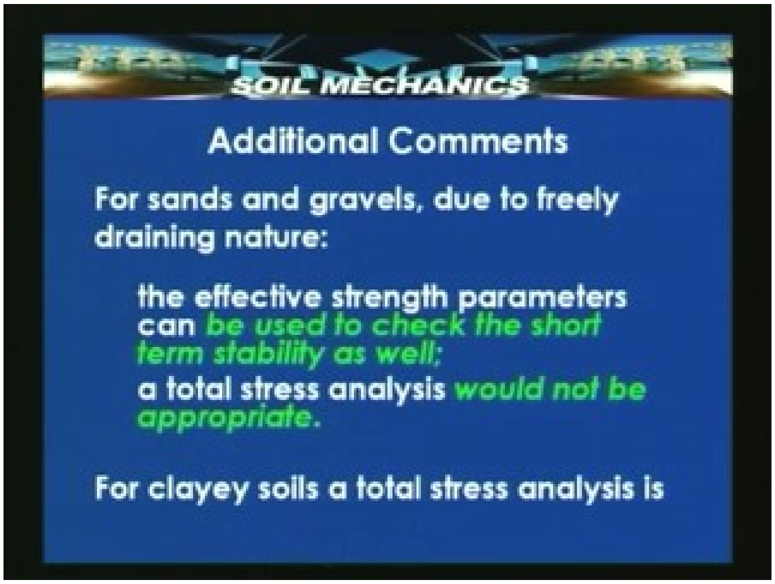

These six factors are listed in front of you here. The type of the particles are responsible for the kind of shear resistance that can be mobilized between the particles when they experience the interlocking pressure or the intergranular stresses. The density of packing up the soil also is a parameter to reckon with because the density of packing indicates how close the particles are and how much in contact they are. Confinement again is an indicator of the degree of contact between the particles.

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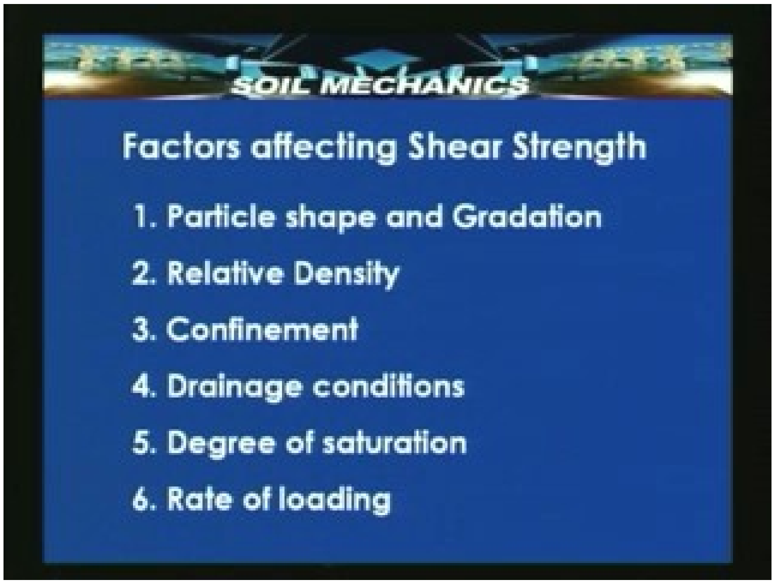

The first 3 factors therefore are best applied to coarse grain soils whereas the next three aspects dealing with drainage, the degree of saturation or relative rate of loading compared to the relative rate of drainage all these things are specifically applicable to fine grain soils. Let's take a look at what the effects of these parameters are.

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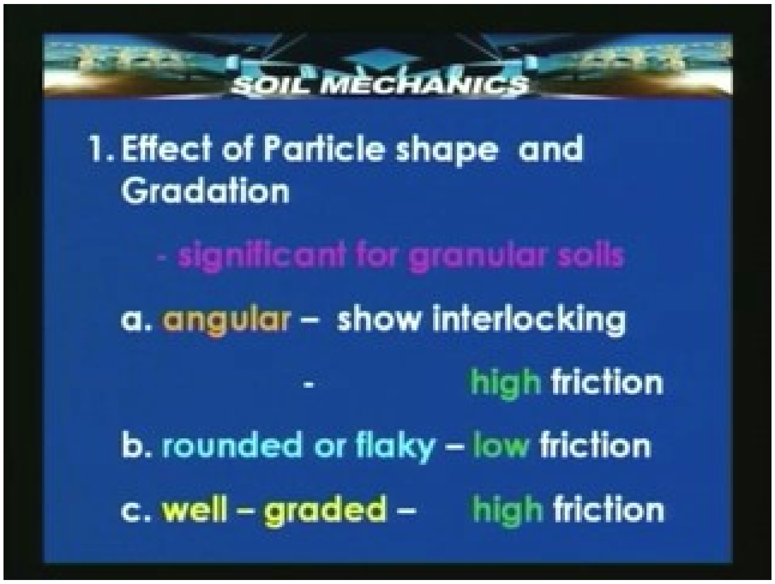

Effects of particle shape and gradation- significant only for granular soils. If the particles are angular if they have sharp edges then they exhibit lot of interlocking. Therefore the angle of friction would be high. If on the other hand they are rounded as in the case river bound soils or flaky as in the case of clayey soils, the friction is likely to be very low. And coming to gradation a soil which is well graded obviously will have the highest possible frictional resistance.

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Here is the table which gives the approximate value of friction for different degrees of density for different particle shapes, rounded and uniform have angle of friction varying from 30 to 37 as the density increases. Well graded soils can have very high friction angle even as high as 45 when they are densely packed.

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Shape and Gradation	φ degrees Loose	φ degrees Dense
Rounded Uniform	30	37
Rounded Well-graded	34	40
Angular Uniform	35	43
Angular Well-graded	39	45

Now effect of relative density is more or less rewording of what we saw just in the previous slide. When the relative density is zero, the soil is in its loosest stage, e the void ratio is minimum and friction is lowest possible. When the relative density is 100% which of course is difficult, the soil is in its densest state, e is maximum and the friction is high.

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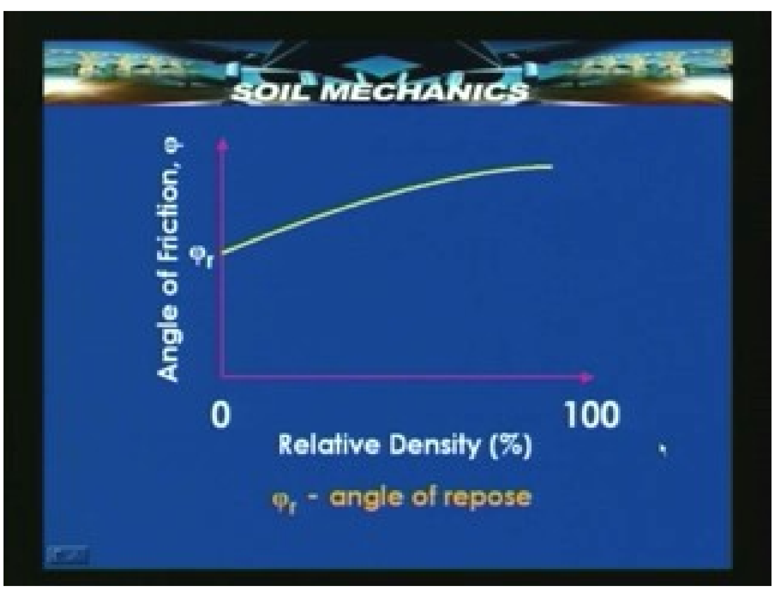

There is approximate relationship between the angle of friction and the relative density, as the relative density percentage increases from 0 to 100, the angle of friction also increases as shown in this figure.

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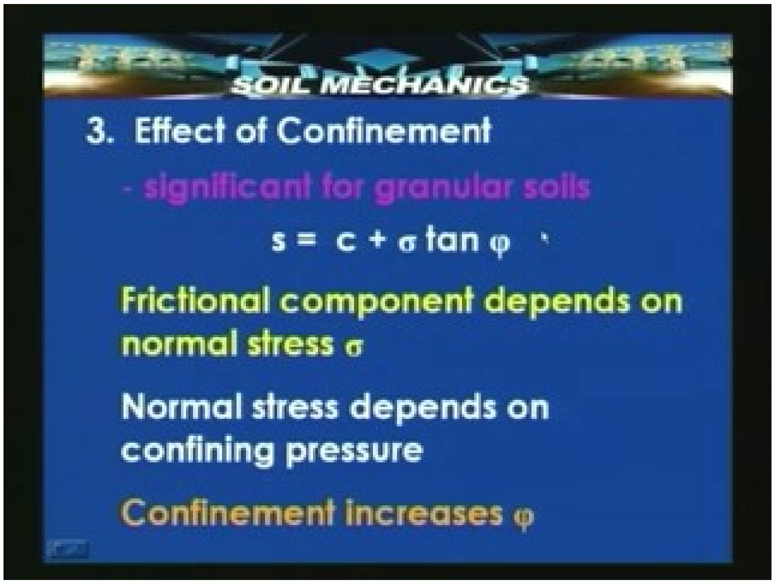

Now confinement is also again significant for granular soils, the frictional component depends upon the normal stress sigma and the normal stress sigma depends upon the confining pressure where sigma is the normal pressure on any potential plane of sliding. Therefore confinement increases phi.

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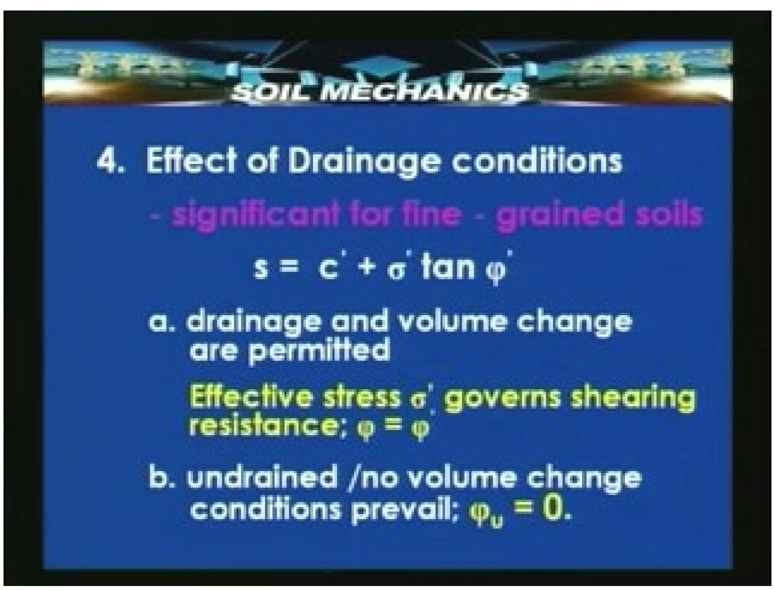

The effect of drainage has been seen by us rather in detail during the course of last lecture and todays lecture and it is significant for fine grain soils depending upon whether we have drainage and volume change permitted or not permitted, we will have effective stress sigma dash or total stress sigma and corresponding angles of friction and cohesion.

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Water content and degree of saturation will also play an important, when the water content increases cohesions decreases. When the degree of saturation increases also, the cohesion increases to an optimum value and then decreases.

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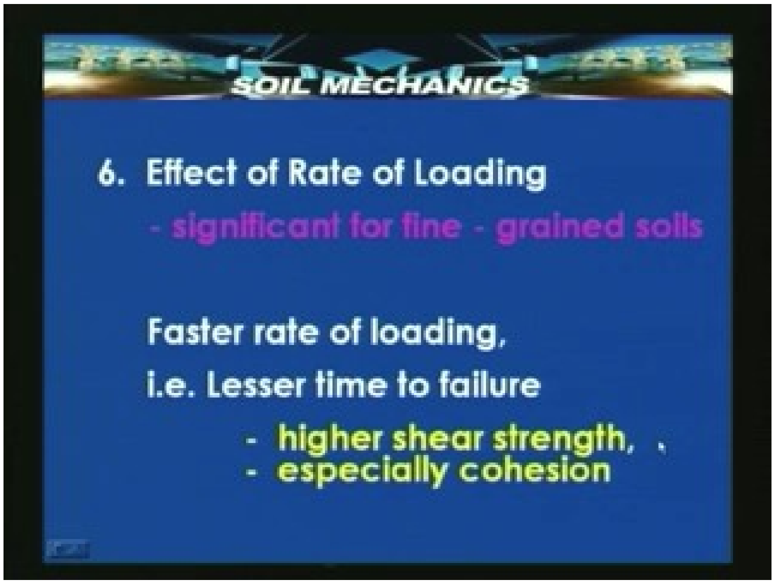

The rate of loading is extremely important because the so called drainage, so called drained or undrained condition is a relative term as I have stressed in the last lecture. The loading if it is rapid compared to the rate of dissipation of pore pressure then it is an undrained condition. So fast rate of loading, less time to failure indicates undrained conditions and therefore it gives a relatively higher shear strength corresponding to the total stress parameters especially for a value of cohesion.

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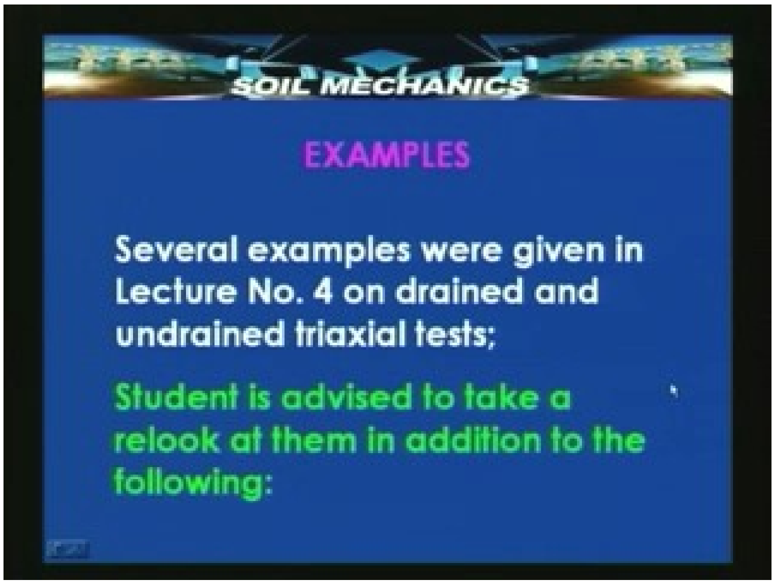

Now let us take a few examples on different types of triaxial tests. We have already seen a number of examples in lecture number 4. It will be recommended that you please go through the examples that I have discussed in lecture number 4. However I will quickly run through those examples for the sake of continuity.

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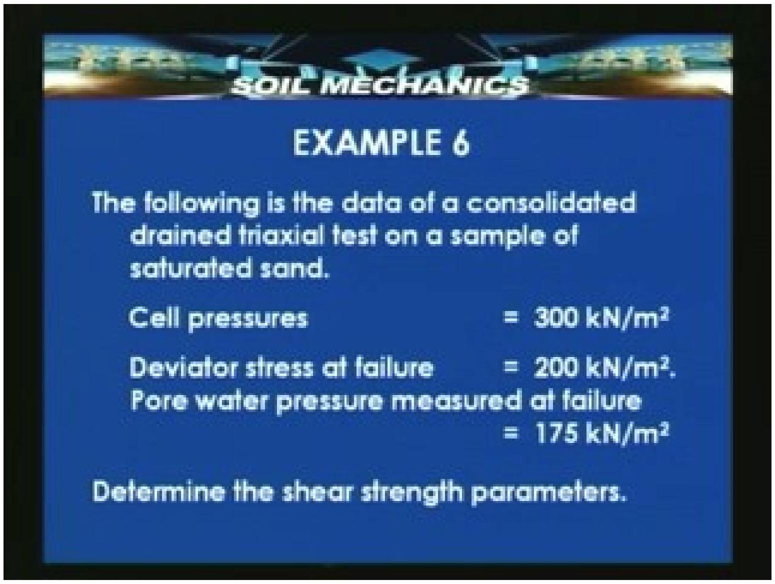

There was an example in which we had the cell pressure, the deviatoric stress at failure and the pore water pressure known and the shear strength parameters were to be determined.

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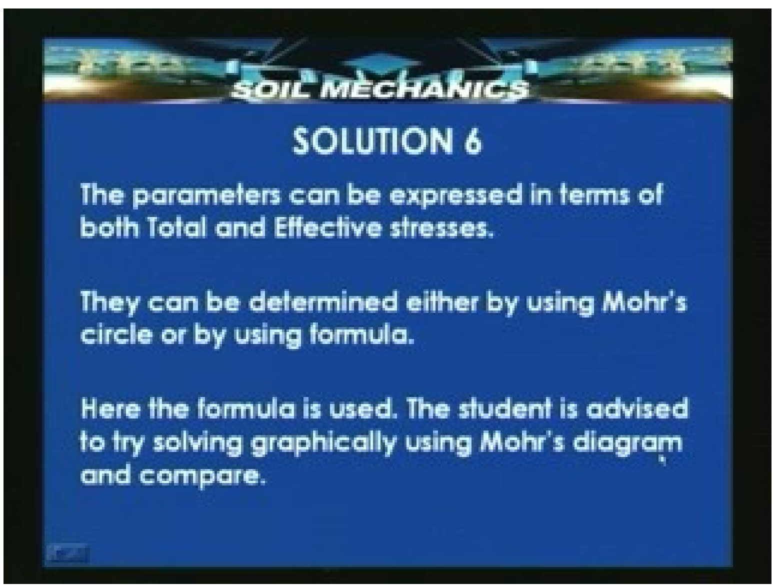

We could either determine the total stresses or the effective stresses and using either the Mohr circle or a formula we could get the cohesions and friction.

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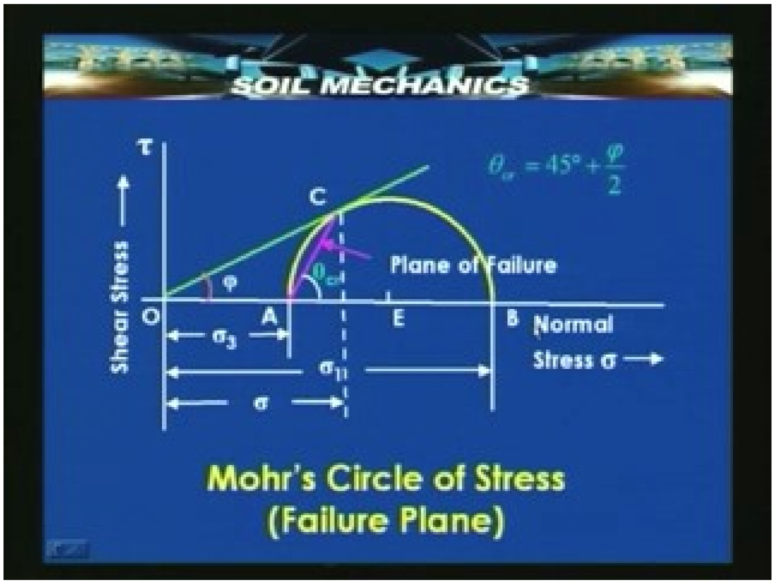

Here is the Mohr circle, by drawing the Mohr circle we can get the cohesion and friction corresponding to this envelop and if it is drained test, c will be zero.

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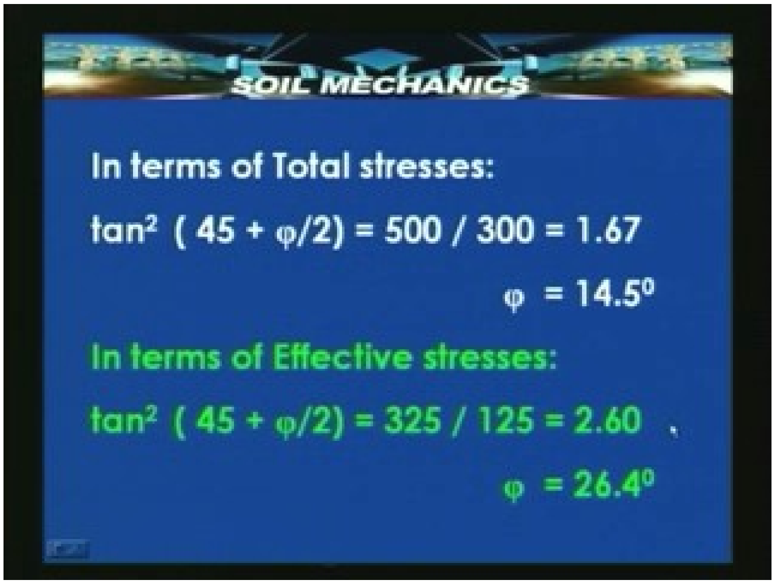

In terms of total stresses for this particular problem we get phi equal to 14.5 and in terms of effective stresses phi is equal to 26.4.

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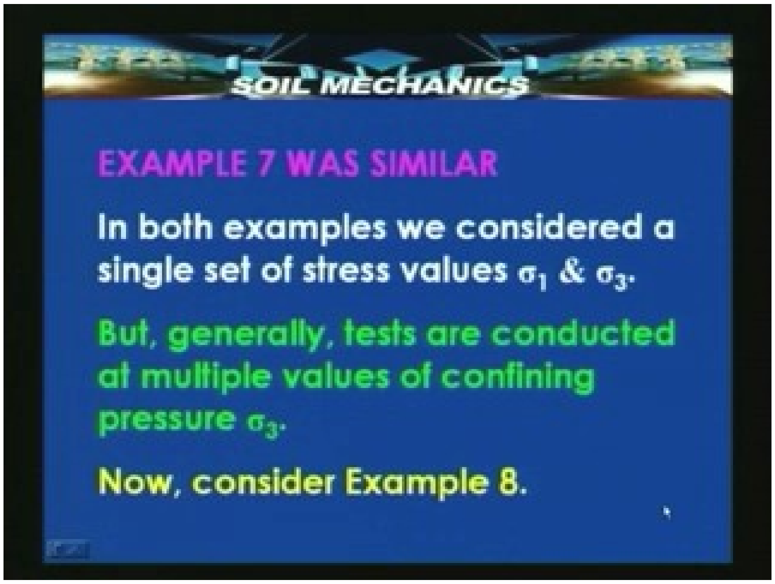

There was another example which was very similar to this.

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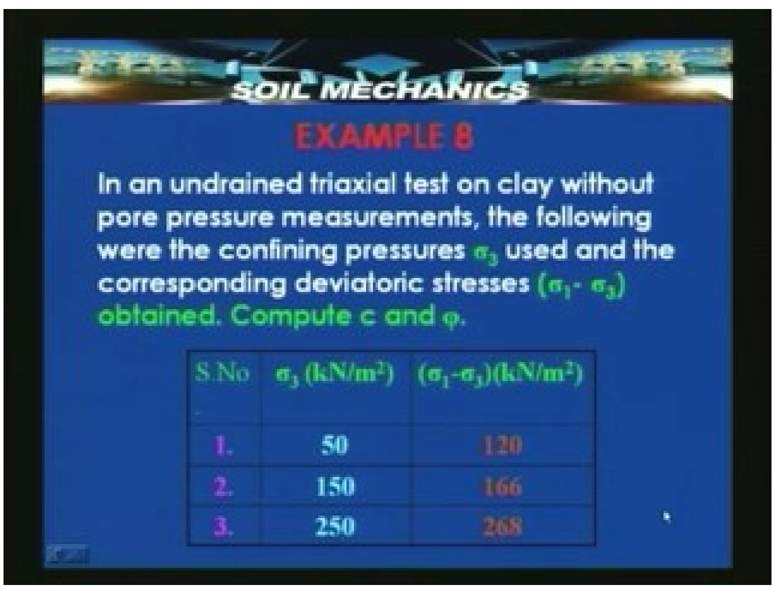

And another example 8 in which we had sigma$_3$ and the deviatoric stress sigma$_1$-sigma$_3$. We computed c and phi either using the Mohr's circle plot or both by using the Mohr's circle plot and by using the p q plot.

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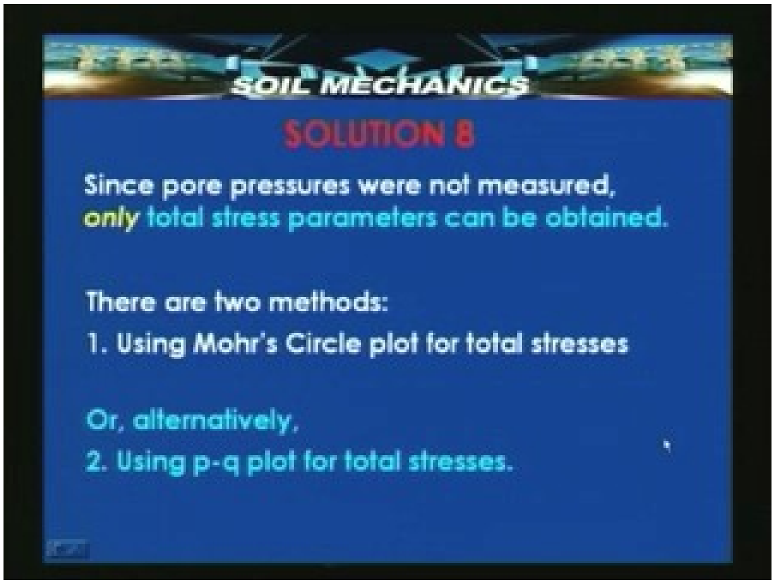

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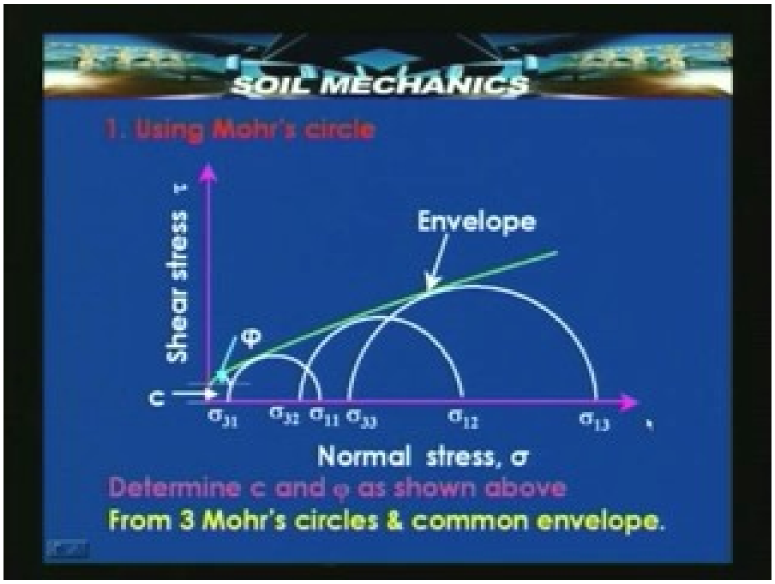

Here is the Mohr's circle plot and from the Mohr's circle plot and using the analytical relationship and also by using the p q plot, we got the values of cohesion and friction.

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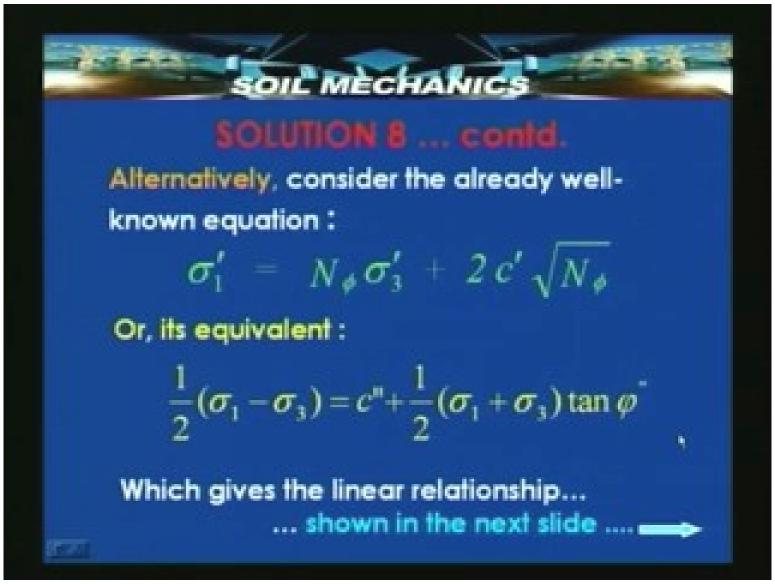

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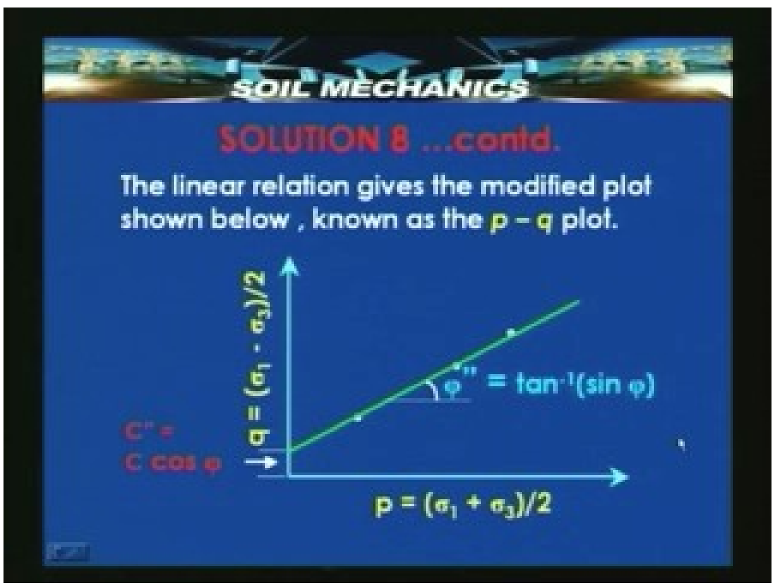

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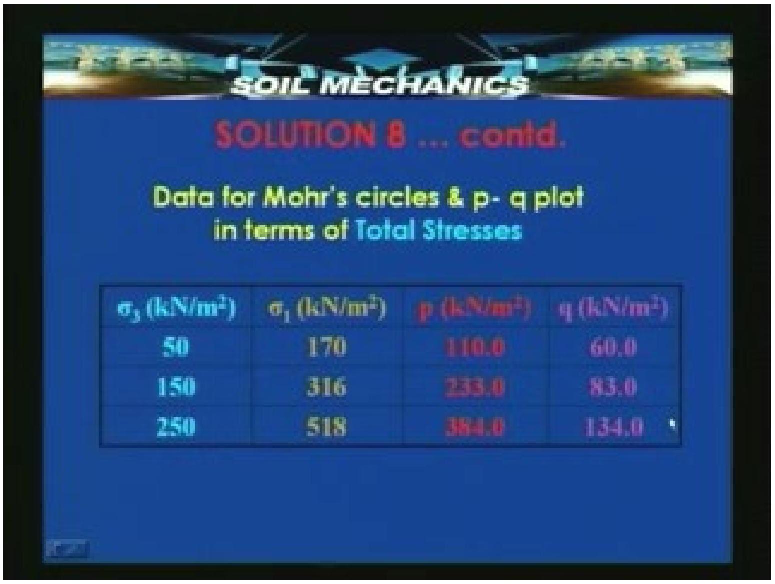

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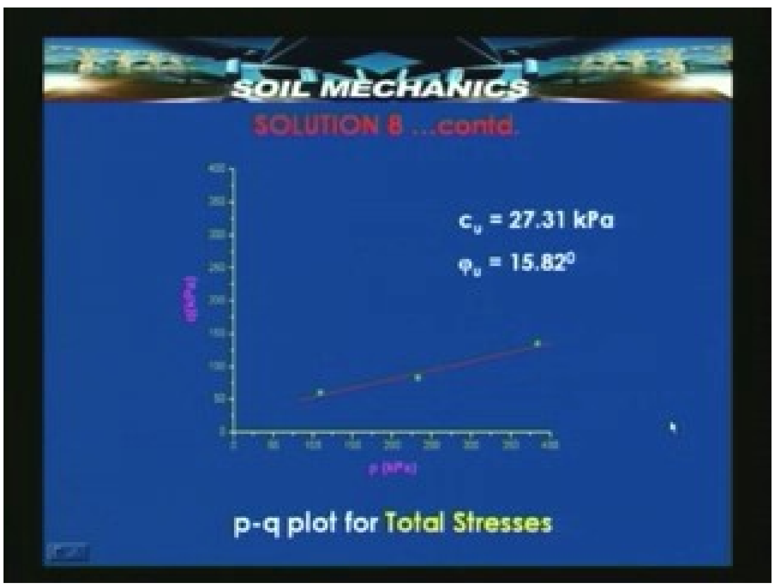

Here the p q plots are shown and the c_u and phi_u values corresponding to total stress p q plots are also displayed.

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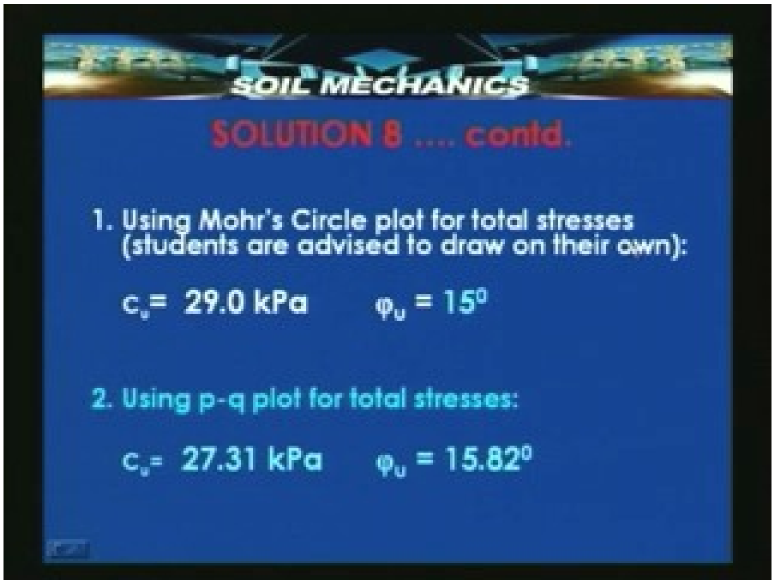

And the c_u and phi_u values from the Mohr's circle plot and the p q plot are shown here.

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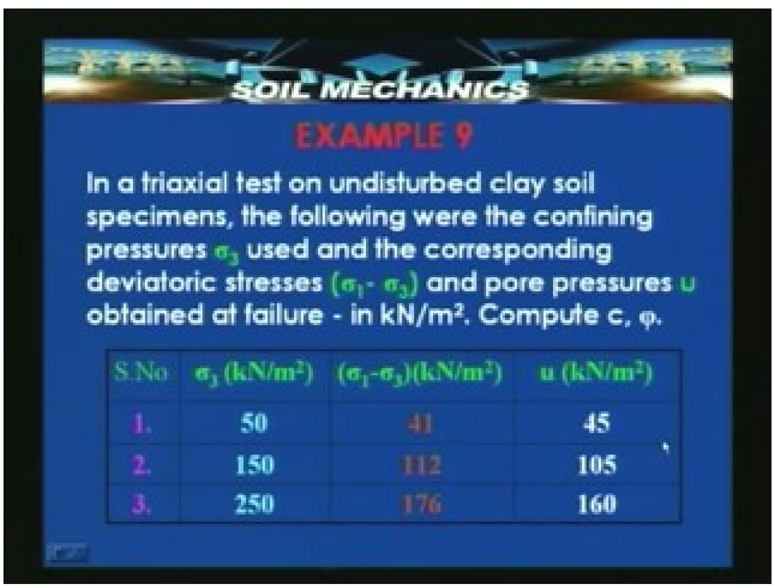

Example 9 again involved knowing $sigma_3$, $sigma_1$-$sigma_3$ and the pore pressure and the computation of the shear strength.

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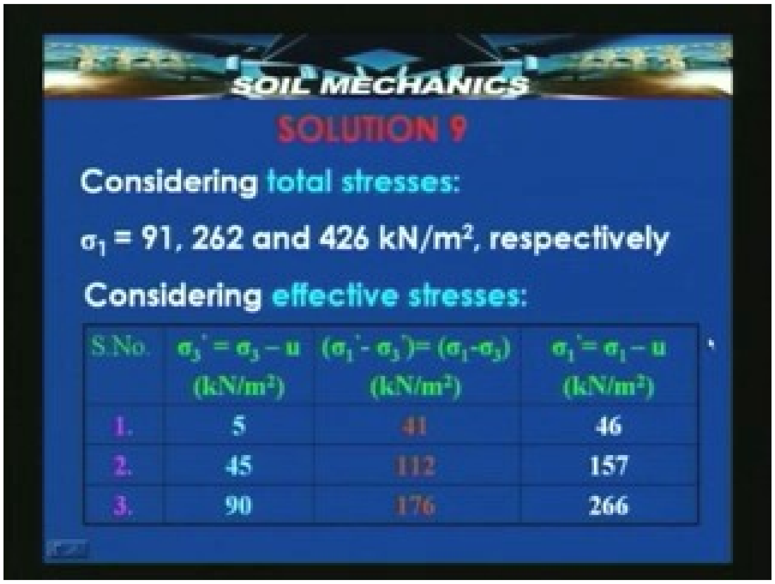

S.No.	$\sigma_3' = \sigma_3 - u$ (kN/m²)	$(\sigma_1' - \sigma_3') = (\sigma_1 - \sigma_3)$ (kN/m²)	$\sigma_1' = \sigma_1 - u$ (kN/m²)
1.	5	41	46
2.	45	112	157
3.	90	176	266

Here we have the pore pressure calculation and the effective stress calculation.

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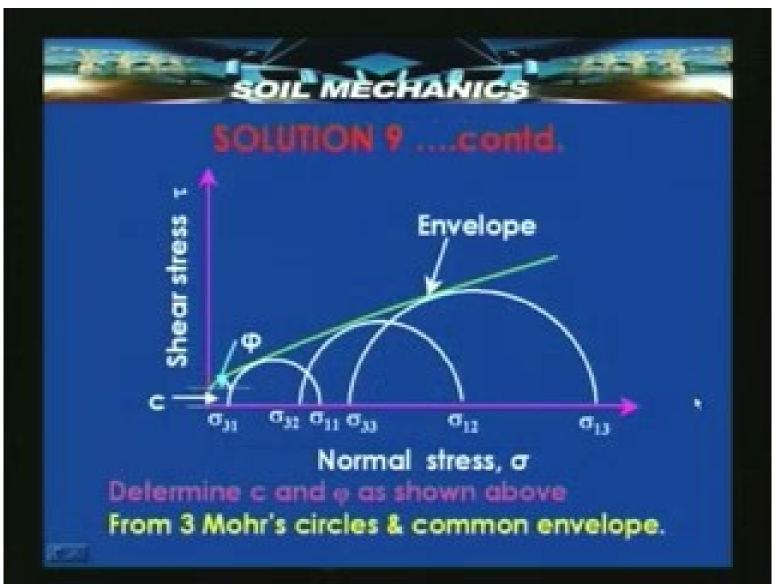

Then we plot a Mohr's circle with the above values of effective stresses.

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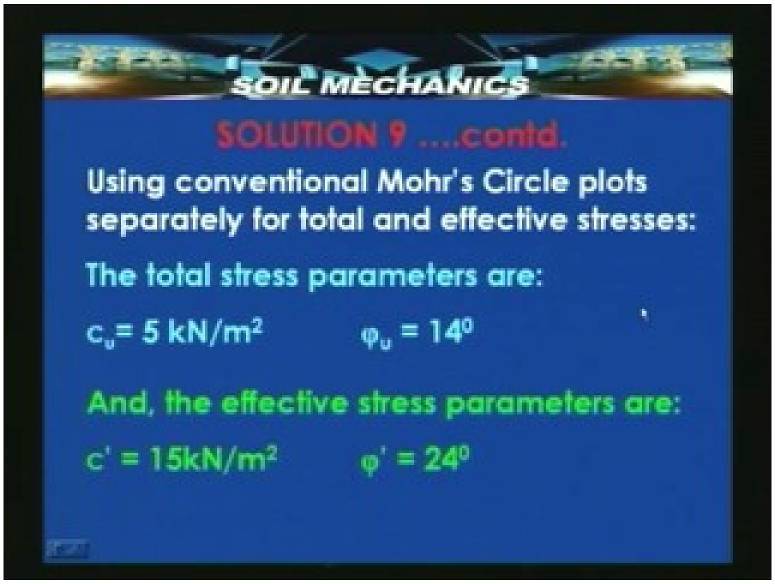

We get the values of c_u and phi u or c dash and phi dash depending upon whether we plot total stress diagram or effective stress diagram.

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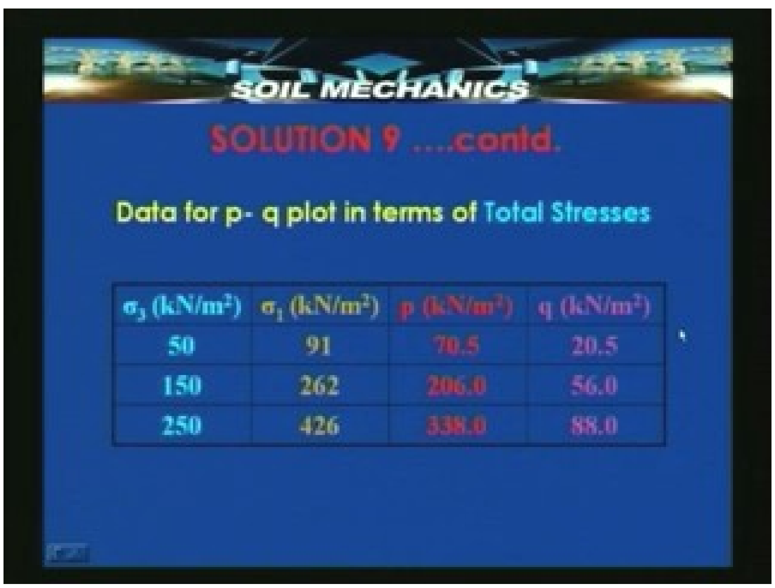

These are the data for the corresponding p q plot and the p q plot also gives you for total stress a value of c_U and a value of phi_U and for effective stresses also a value of c dash and phi dash.

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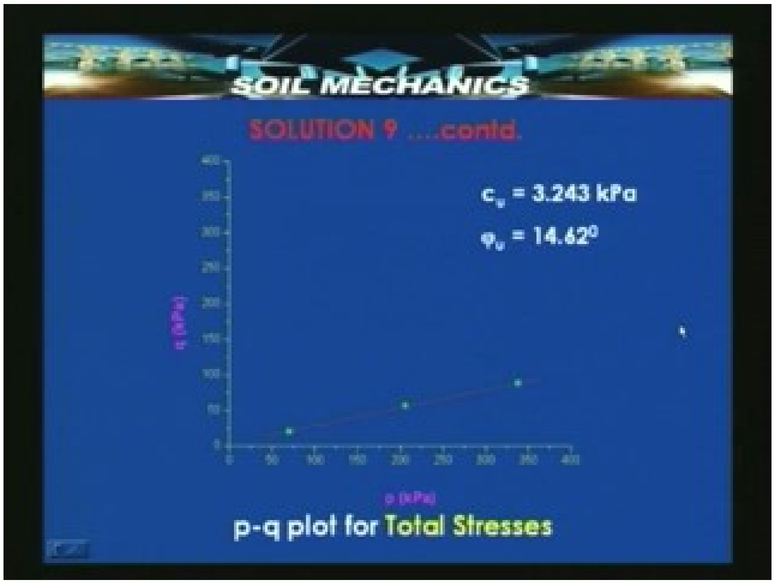

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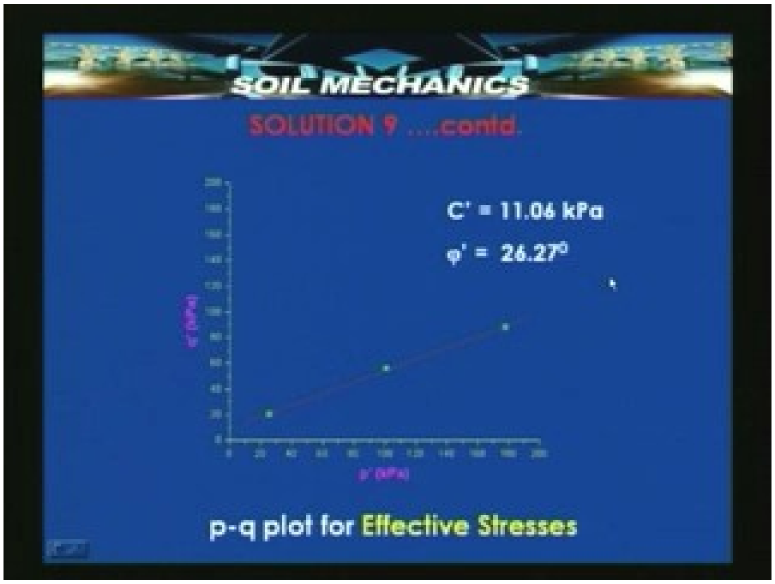

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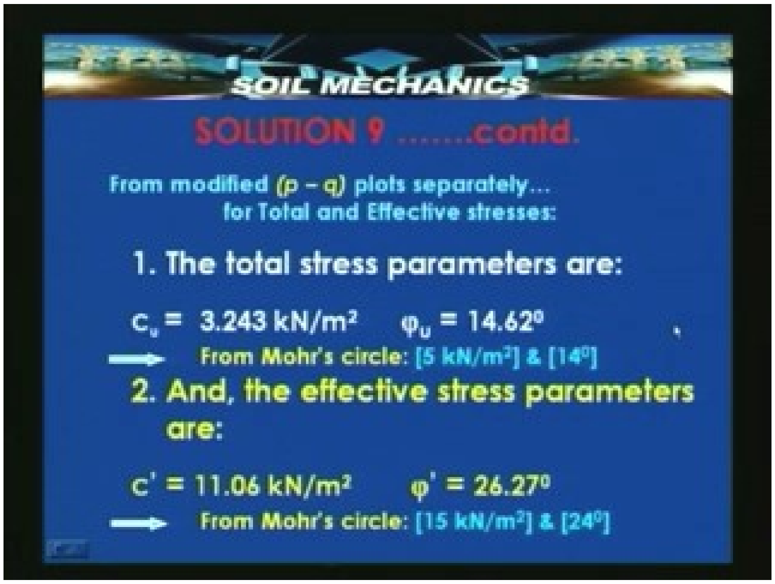

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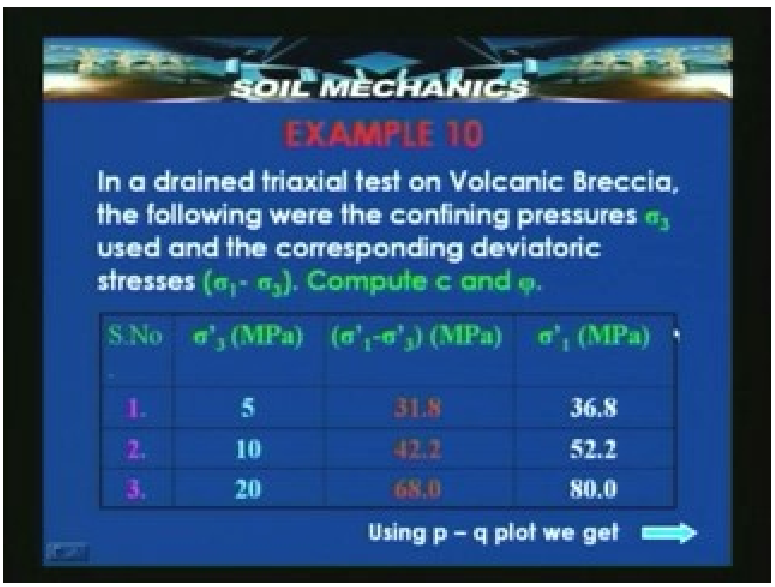

This is the summary of the results. This is again another example which we saw last time where sigma$_3$ dash and the deviatoric stress effective were both known from which we calculated the sigma$_1$ dash and computed c and phi by an effective stress p q plot.

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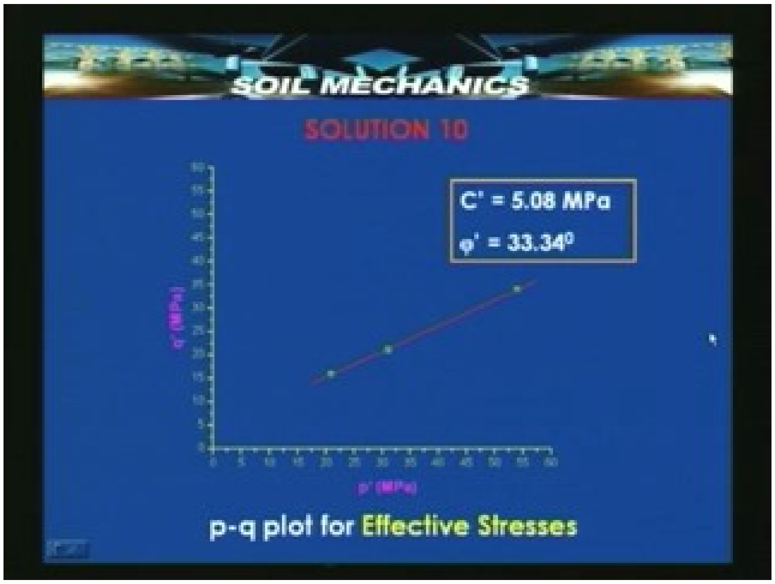

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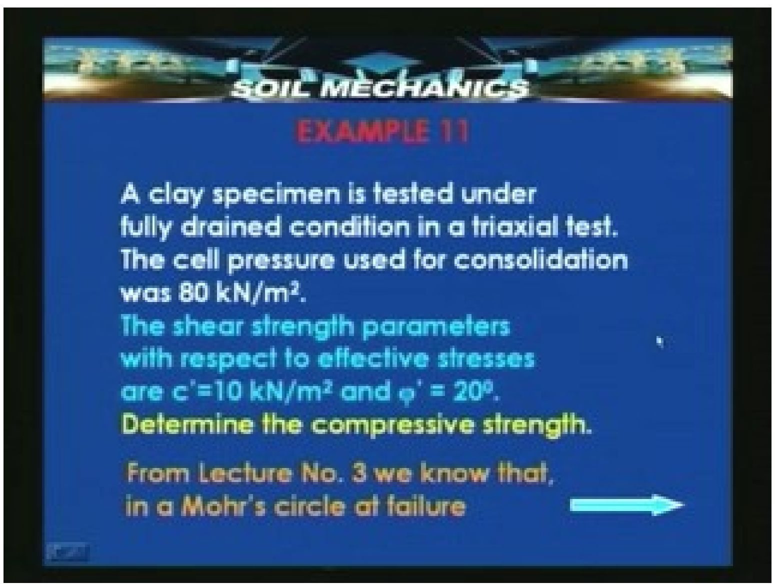

Now we go on to another example in which a clay specimen is tested under fully drained condition in a triaxial test. The cell pressure used was 80 kilo newton per meters, the shear strength parameters were known c dash and phi dash. What is the compressive strength? The compressive strength can be calculated as equal to the distance AB the deviatoric stress.

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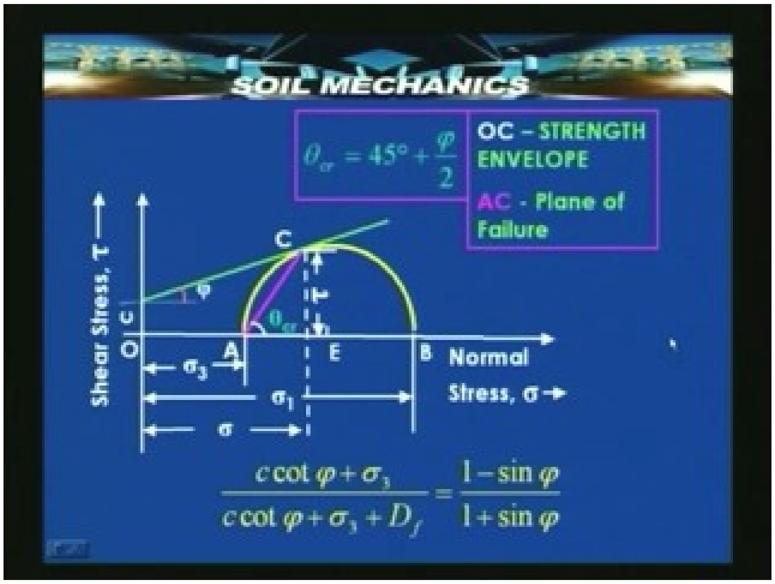

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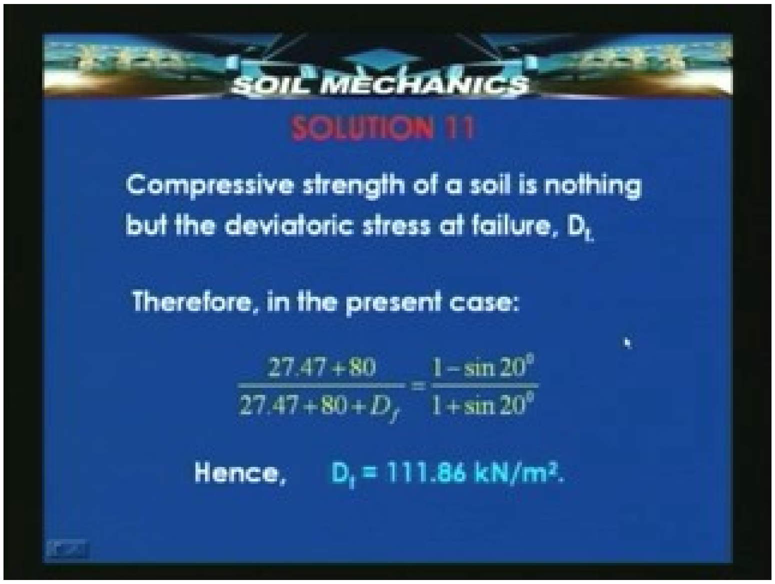

And so from the Mohr diagram and by using this relationship which can be obtained from geometry, we can get the deviatoric stress D_f which is nothing but the compressive strength. This is similar to a problem which we saw in the lecture number 3 and therefore I would recommend that you take a look at the formula and its application as it has already been discussed in lecture number 3.

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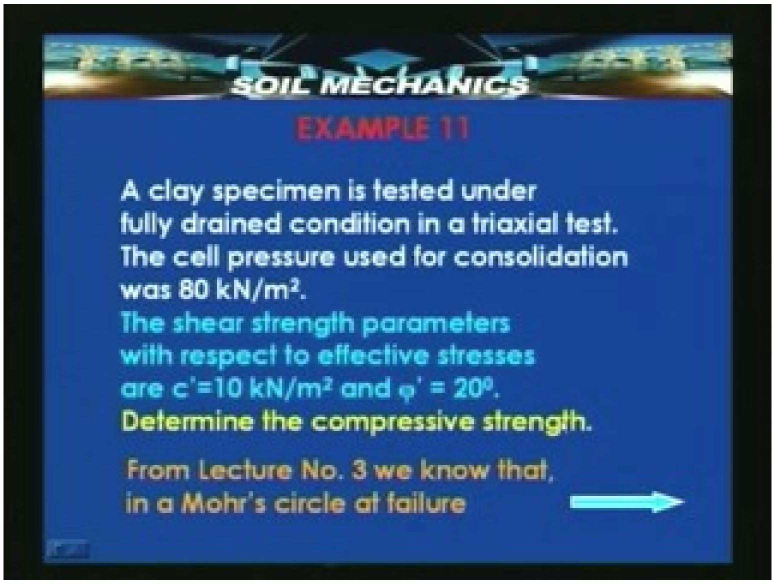

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Here we have now new examples introduced in this particular lecture. Here is an example in which sigma_3 and sigma_1 are given for 3 samples, c and phi are required. This is very similar to the example number 10 which we have just gone through and therefore using the p q plot for example we can get c and phi. Now what are these values of c and phi? Being a drained test the values which we obtained are nothing but the effective stress parameters but he result shows that there is a value of cohesion equal to 23.5 kilo newton per meter square. We had seen that in a CD test there is no cohesion at sigma equal to zero, for a normally consolidated soil. If there is cohesion then the soil is over consolidated soil. Therefore we conclude that this particular soil in this example is an overconsolidated soil because c dash is not equal to zero.

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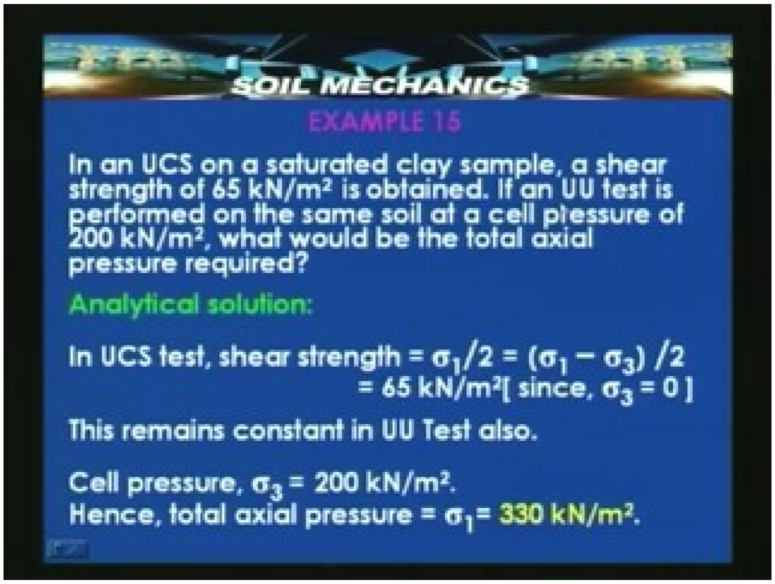

The next example 15 gives you the shear strength of a saturated clay sample from an unconfined compressive strength. Since the unconfined compressive strength is very similar to the UU test except that the sigma$_3$ value is zero, we can use this result to calculate the axial pressure required to cause failure in an unconfined compressive strength if we know the sigma$_3$. So here it is from the unconfined compressive strength test we get sigma$_1$- sigma$_3$ by 2 where sigma$_3$ is zero. Since this remains constant in the UU test, from here we can calculate sigma$_1$- sigma$_3$ + sigma$_3$ and get sigma$_1$ the axial stress which comes to 330 kilo newton per meter square in this particular case. This is a total pressure because it's an unconsolidated undrained test.

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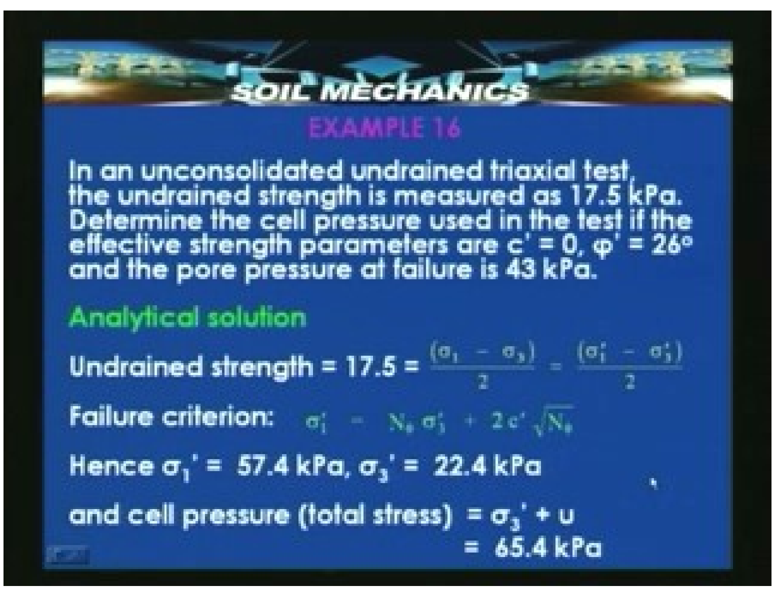

The next test is again an unconsolidated undrained test. Here we have the total strength measured as 17.5 kilo pascals and the cell pressure used, if the effective strength parameters are c dash equal to zero, phi dash equal to 26 degrees and the pore pressure equal to 43 kilo Pascal's, what would be the cell pressure? We know the undrained strength that must be equal to sigma_1- sigma_3 by 2 or sigma_1 dash minus sigma_3 dash by 2, both are equal. We use this failure criterion which gives us a relationship between sigma_1 dash and sigma_3 dash. Since we know the pore pressure value and also the undrained strength sigma_1 - sigma_3 by 2. We can use this relationship and calculate sigma_1 dash and sigma_3 dash and once we know sigma_3 dash, using the value of pore pressure 43 kilo Pascal's, we can calculate the total cell pressure which is the unknown.

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Now lets take a quick look at cohesion less and cohesive soils and their behavior. We have seen this slide before in lecture 3. Here is a typical direct shear specimen when we conduct a test on sand, if the sand is dense we get shear stresses versus shear displacement diagram like this and if the soil is loose sand then the shear stress verses shear displacement diagram becomes like this.

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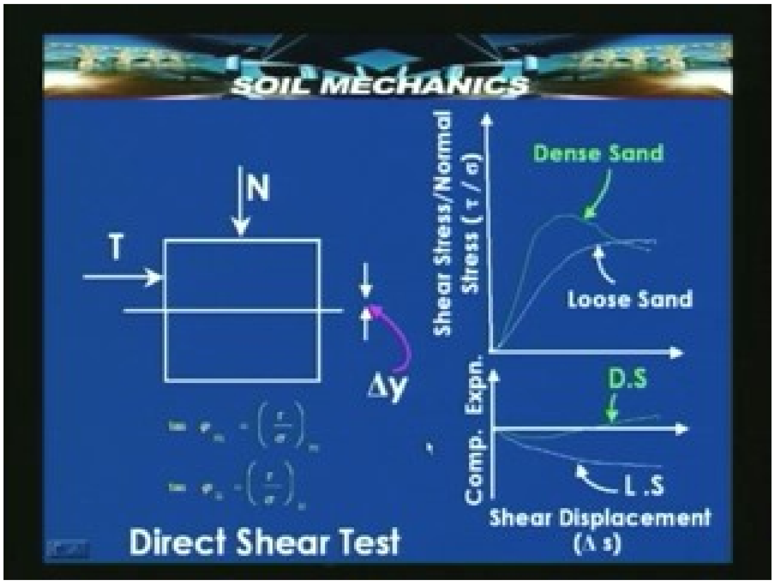

The volume slightly decreases, there is a slight compression initially and then the soil starts expanding if the soil is dense sand. On the other hand it continues to show a compression if it is loose sand and which means that we will have peak value of tan phi equal to tow upon sigma peak or a tow upon sigma ultimate corresponding to the portion beyond the peak. And in the case of loose sand only the second aspect will apply.

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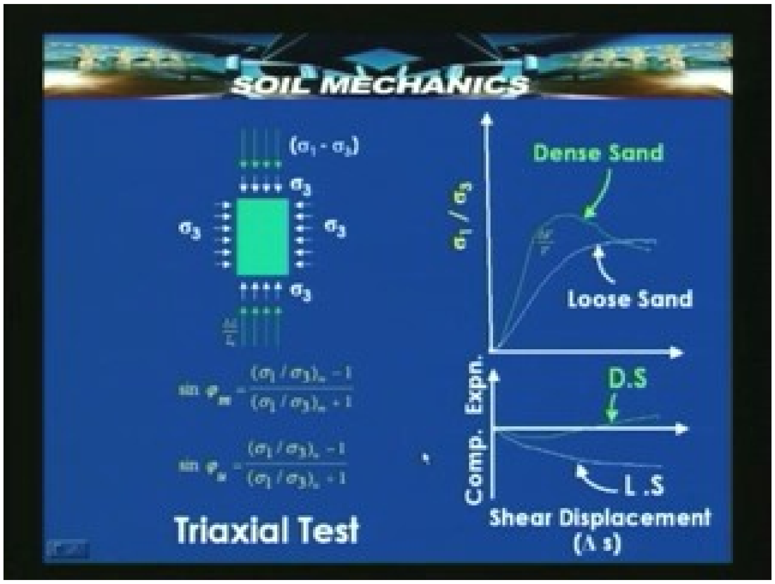

Now if you take the triaxial test again dense sand and loose sand will behave in a similar manner. If we plot sigma$_1$ upon sigma$_3$ and the shear displacement or the axial displacement in this case, we will get a diagram very similar to what we got for the direct shear test and this again shows that there is a peak value of phi and an ultimate value of phi both of which can be computed from this simple relationships. Here again there is a compression in the early stages, in the case of dense sand this gets converted into an expansion as the shear displacement increases, whereas in the case of loose sand continuous compression goes on taking place.

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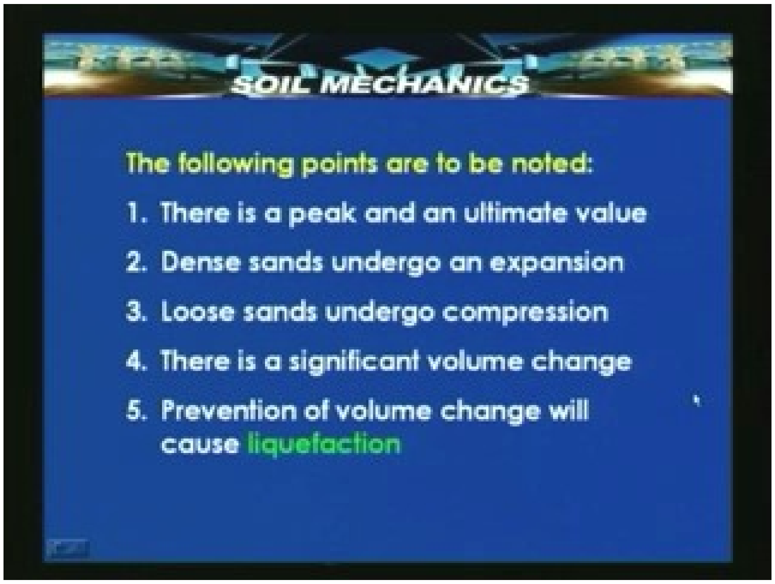

We can summarize our observations like this. There is a peak and an ultimate value when sand is subjected to either direct shear or triaxial shear. Dense sands undergo an expansion, loose sand undergo compression. There is a significant volume change in both cases, prevention of this volume change is not going to permit development of the effective stresses and the friction angle and therefore it will lead to a kind of failure which arises because this behaves like a liquid, shear resistance is not allowed to develop due to volume change prevention and this phenomenon is known as liquefaction.

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Now let us take a look at the concept of critical void ratio and the meaning of liquefaction, significance of liquefaction and how to determine it. When volume change is prevented there is a gain in strength in dense sand because as we have seen in the compression behavior of the dense sand there is a compression initially and if volume change is prevented this compression does not takes place and therefore there is a gain in strength.

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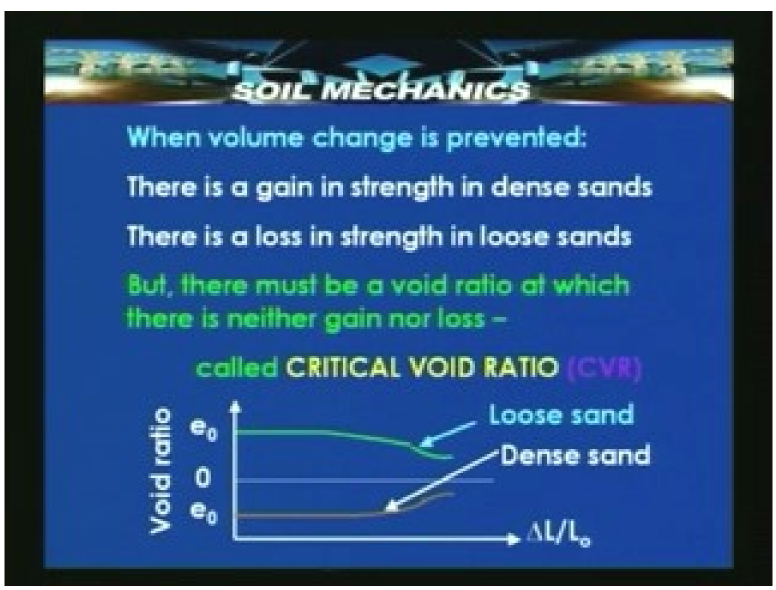

Whereas on the front of loose sands, if we do not permit volume change we are preventing the sand from becoming denser and gaining more strength. Therefore it actually loses strength but this shows that there must be a void ratio at which sand neither is dense nor is loose and therefore neither gain strength nor loses strength. Take a look at this diagram on the above slide. This shows void ratio verses the axial strain in a typical triaxial test. As the axial strain goes on taking place and compression goes on taking place, if we do not permit volume change, a loose sand will go on showing decreasing void ratio whereas dense sand will go on showing slight increase in void ratio. Ultimately if we compare these void ratios with a void ratio corresponding to the zero line, we find that there is a void ratio here at which the axial strain is such that no volume change takes place.

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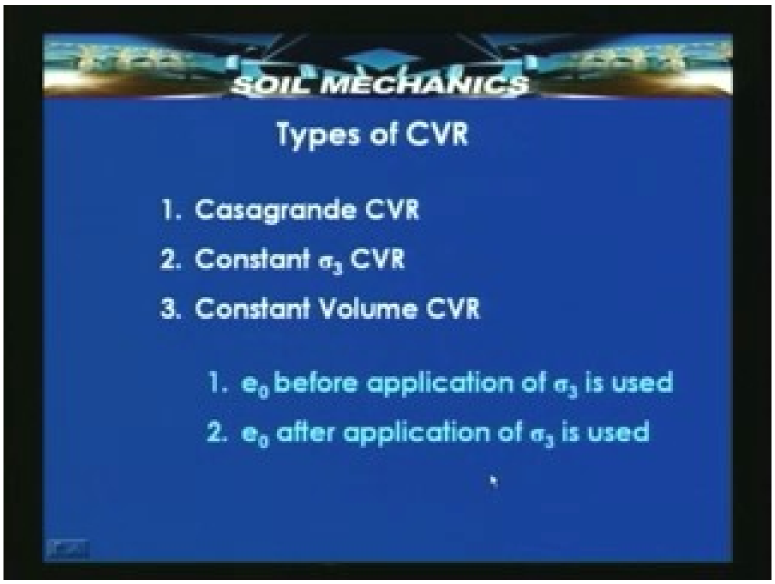

That means void ratio is a critical value at which neither gain in strength is there nor loss in strength is there and the soil is neither dense nor loose and it is this void ratio at which volume change does not take place and therefore soil undergoes liquefaction during shear. This void ratio actually is defined differently by different persons; Casagrande has proposed a critical void ratio of his own which is nothing but the void ratio which is determined before the application of the confining pressure $sigma_3$.

On the other hand if we do what is known as the constant $sigma_3$ tests that is the conventional triaxial test where $sigma_3$ is kept constant in the first stage of loading then the e_0 after the application of $sigma_3$ if that is used the corresponding CVR is known as constant $sigma_3$ CVR. If we use what is known as a constant volume test that is through out the test we adjust $sigma_1$ in such a way that there is no volume in the triaxial sample then that's known as the constant volume CVR.

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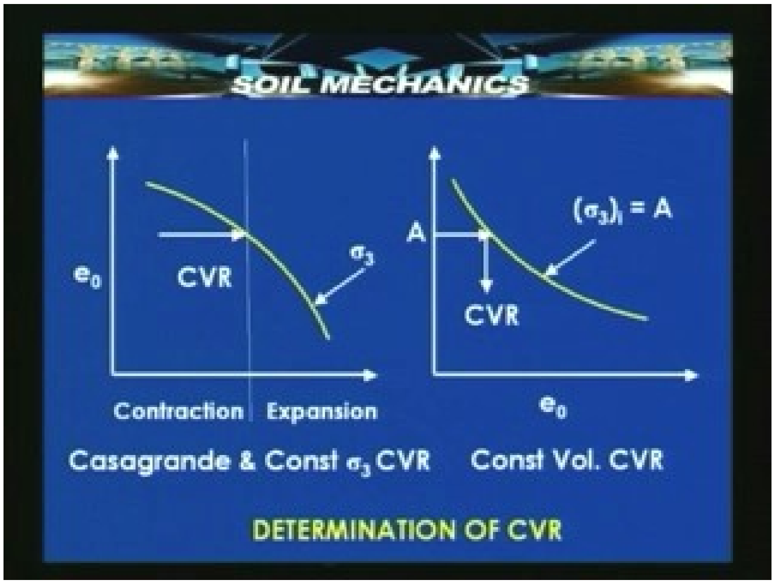

Here is a method to determine the critical void ratio, Casagrande and constant $sigma_3$ CVR can both be determined by conducting a typical triaxial test plotting the volume change either contraction or expansion as against the void ratio along the y axis. Here is the vertical line which signifies that on the left of this there is contraction, on the right of this there is expansion depending upon void ratio. Then this line also shows that there is no volume change either contraction or expansion, if the volume change if the void ratio is such that it falls on this line and on the curve shown here. This curve is the e_0 versus volume change curve and this vertical line is a no volume change line and therefore this point of intersection is nothing but the critical void ratio. This critical void ratio is the void ratio as per Casagrande or constant $sigma_3$test because Casagrande test is also a constant $sigma_3$ test.

If we do a constant volume test on the other hand then we do not end up with any contraction or expansion here, volume remains the same and therefore what we need to do is to plot a parameter A that is nothing but $sigma_3$ at the end of the test against the initial void ratio and get this curve for different values of initial void ratio. Then the critical void ratio is that which corresponds to a value of A which is equal to $(sigma_3)_I$ where $(sigma_3)_I$ is the initial confining pressure that is applied to the soil. This is a constant volume test remember, therefore whatever $(sigma_3)_I$ we apply gets adjusted by us and goes on changing during the test. So that value of $(sigma_3)_I$ which does not change which remains the same at a particular void ratio gives us the critical void ratio. So this is how we determine the critical void ratio.

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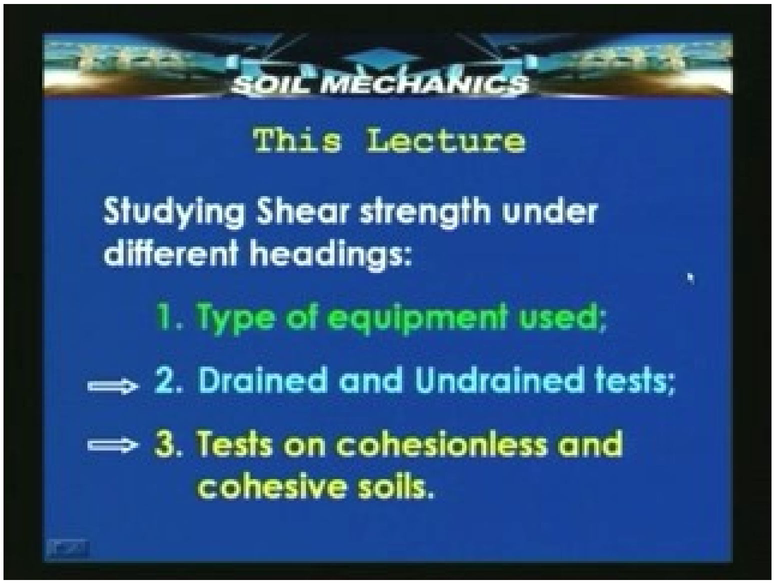

With this we come to the end of today's lecture, we have studied shear strength during the course of today's lecture with specific reference to the drained and undrained test and with respect to tests on cohesionless soils and cohesive soils. But actually so far we have covered in explicit detail only test on cohesion less soils, the reason is whatever we have studied with respect to drained and undrained test themselves are nothing but the explanations for test on cohesive soils. In the next lecture, we will continue a little bit with tests on cohesionless and cohesive soils and also take up a few special topics. Thank you.