Geology and Soil Mechanics
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Lecture - 59
Tutorial on Shear Strength - a

Hello everyone. So, welcome to our last lecture on the tutorial section for Geology and Soil Mechanics. Now today we are going to continue the shear strength behaviour of soils and we are going to end at the shear strength behaviour of soils. The rest of the lectures that whatever Professor Priyanka Ghosh will cover will be on the earth pressure theories and a few on bearing capacity which I have already covered in the previous which I have already covered a bit in the previous lectures in the previous tutorial sections. So, we are discussing about a problem on elastic soil and how to find out the Poisson's ratio for the problem on the elastic soil.

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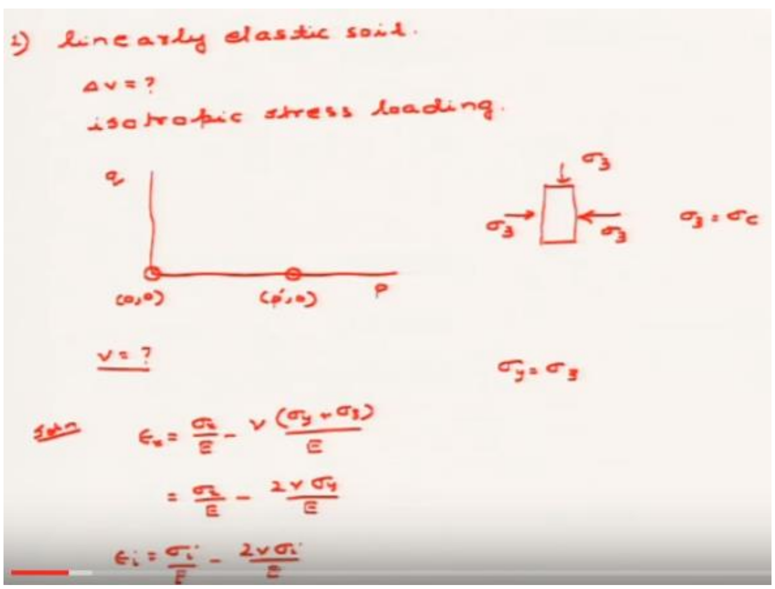

So, basically, the problem stated that a linearly elastic soil was present and you are supposed to find out the change in the volume that is delta v when subjected to isotropic stress loading. Now we also said that the isotropic stress loading is basically, in a pq space. An isotropic stress loading means this the sample is not subjected to any shear stress. That means basically, if the sample starts from a point at 0,0 then it will be at some point P dash 0 after an isotropic stress.

Isotropic stress also means in realistic sense actually an equal confinement from all the 3 directions in case of a triaxial sample which we generally termed as sigma 3 or you can also call that as sigma c. So, an equal confining stress from all the 3 directions means it is an isotropic behaviour. Now it is asked that what can you say about the Poisson's ratio of the soil. So, we said that from the basic definition of mechanics we said that epsilon can be written in the form of sigma by epsilon - nu into if this is epsilon x this is sigma x by y sigma y + sigma z by E. Now in case of a traction we also said that the 2 directions sigma y is equivalent to sigma z. So, this equation turns back to sigma x by E - 2 nu into sigma y by E. In a generalized term we can write this as epsilon i is equal to sigma i by E - 2 nu sigma i by E.

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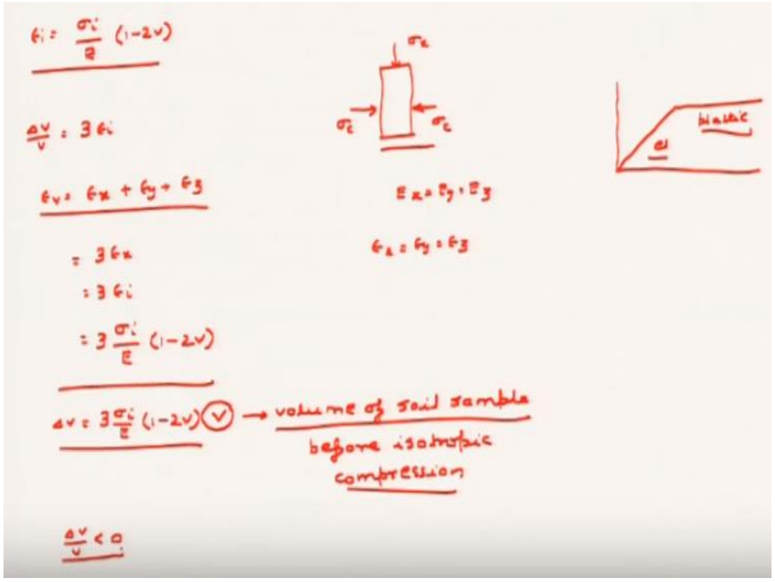

Now this can also be written as so epsilon i is equal to sigma i by E into 1 - 2 nu. The volume change as we all know is equivalent to 3 epsilon i. Now how did I get that? This is because the volumetric strain epsilon v is equivalent to the strain in x direction the strain in y direction and the strain in z direction. Now from the problem itself it is given that actually it is an isotropic stress loading.

That means basically, it is subjected to equivalent stresses in all the 3 directions which is sigma c, sigma c, and sigma c. So, obviously if the elastic material is constant means if E is constant in all the 3 directions that is E x equal to E y equal to E z then epsilon will also be constant in all the 3 directions. This epsilon x will be equivalent to epsilon y will be equivalent to epsilon z. So, you can write this as 3 times of epsilon x or you can write just 3 times of epsilon i so 3 times of sigma i by E into 1 - 2 nu.

So, this is the total volume change. Now you have to consider now this is this is the first question the question for the first answer that is delta v. What is delta v equal to? So, delta v you can write as 3 into sigma i - E into 1 - 2 nu into V where V is the total volume of the soil sample. Now this is obviously volume of the soil sample before consolidation or before isotropic loading before isotropic compression. So, obviously this is only valid remember this is only valid for a linearly elastic soil but however it may be but however you also know that a soil in a stress-strain curve obviously has a elastic zone and a plastic zone.

So, obviously in this plastic zone these conditions are not valid. So, more higher terms will come in the plastic zone which we will ignore because this is a basic soil mechanics course so only in the elastic zone whatever terms are only in the elastic zone whatever the volume change is this can be found out from this formula. Now you have to consider in order to find out the Poisson's ratio you have to first consider that the volume change is compressive in nature. So, if volume change is compressive in nature that means delta V by V should be less than 0.

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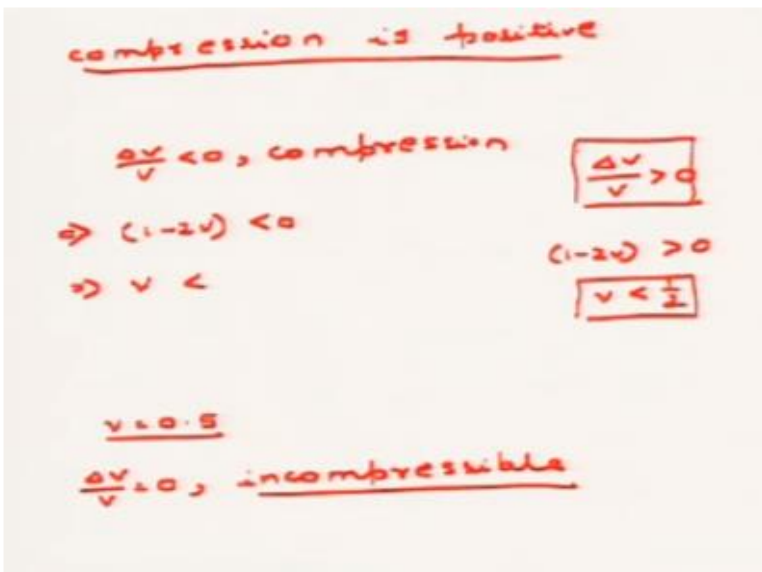

That is one of the most important part because in soil compression is positive but when you consider a general from the general mechanics if you consider the definition of compression or from general soil mechanics from general mechanics if you consider the tension and consider the tension is always positive while compression negative. Since we derived this formula from the basis of general laws of mechanics or general laws of soil mechanics so in this case actually we consider that delta V by V should be less than 0 for compression.

If this is true then we can also write that 1 - 2 nu should be less than 0. So, this implies that nu should be nu should be 1 - 2 nu should be greater than 0 sorry delta v by v should be greater than

0 for compression delta v by v should be greater than 0 for compression because as I have said that in case of soils if you consider the compression then compression has to be positive in nature.

So, delta v by v should be greater than 0. So, in that case 1 - 2 nu should be greater than 0 so this implies nu should be less than half or nu should be equivalent to less than 0.5. Now what happens if nu is equivalent to 0.5? Now you all know that when nu is equivalent to 0.5 then this corresponds to delta v by v equivalent to 0 that means the soil is termed as incompressible. So, this is the general this is the general very easy problem on considering the linear elastic soil.

Now as I have said that you are given different you have to perform different tests and you are given different parameters to find out. Let us say from an oedometric modulus you have to find out the oedometric from an oedometric test or basically, from a consolidation test you have to find out the oedometric modulus and from triaxial test you have to find out the value of E. So, the next question that I said is basically, how you can find out the oedometric modulus from this case.

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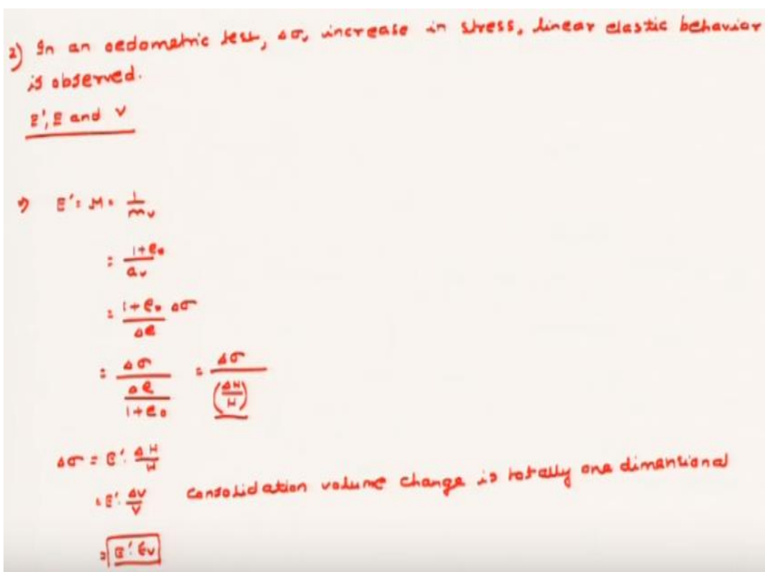

Now it is said that in an oedometric test for delta sigma v increase in stress a linear elastic behaviour is observed. So, it is clearly given that the linear elastic behaviour that means all the definitions that we said in the previous part are all valid. Now in this case you are asked to find out the relation between oedometric modulus E dash the Young's modulus E and nu. So, as I have said in the previous tutorial section that E dash can be also written as M which is equivalent to 1 by m v.

So, this means it is equivalent to 1 by a v into 1 + e 0 which means it is equivalent to delta e delta sigma. So, delta sigma divide by delta e by 1 + e 0 now if you remember the definition from consolidation correctly then delta e by 1 + e 0 is nothing but equivalent to delta H by H. So, this is what is the definition of E dash. So, from here you can find out that delta sigma is equivalent to E dash into delta H by H.

Now this is also equivalent to delta v by v since consolidation volume change is totally one dimensional. That is why in earlier lectures also I said that it is termed as Terzaghi's one dimensional theory of consolidation. So, delta v by v as we said in the previous definitions the delta v by v is nothing but actually equivalent to epsilon v. So, this is E dash into epsilon v. So, this is from the basic definition that we get the definition of oedometric modulus.

Now delta sigma is obviously nothing but just the increase in vertical stress because you can apply oedometric stress or consolidation test only in the vertical direction. So, if you consider that the oedometric test in the vertical direction then how can you find out the relation between oedometric modulus, the Young's modulus, and the Poisson's ratio.

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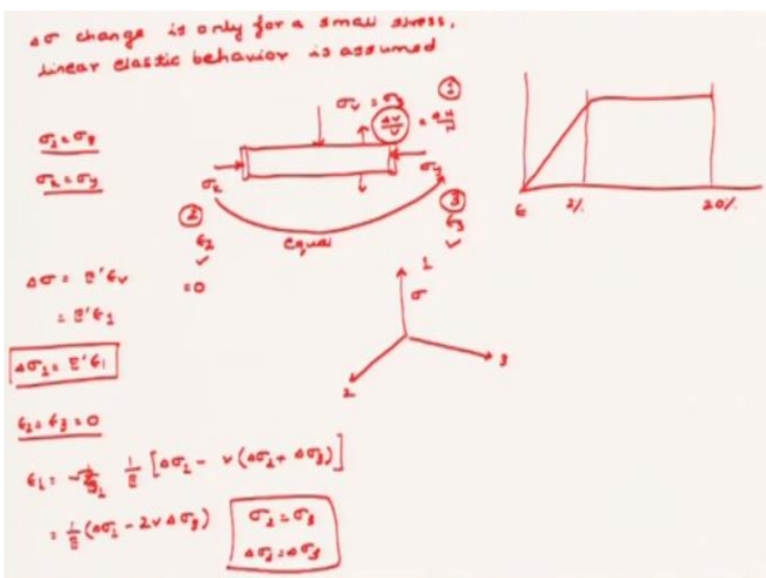

So, one thing you should remember that this delta sigma change is only for a small stress linear elastic behaviour is achieved. That is obvious because if you perform a if you perform a stress-strain test as well triaxial direct shear or oedometric you will see that this elastic range is very small. It may come within the 2% of the total strain or it may come with whereas the total strain may go up to as high as 20%.

So, basically, the 2 percent of the strain or basically, it is a it is just a negligible portion compared to the total strain and within that range only the elastic range is actually assumed. So, in this case delta sigma is also assumed to apply for that small range only because it is said the linear elastic behaviour is there. Now one thing you should also remember in case of an oedometric test that though it is never mentioned that sigma 2 is always sigma x is always equivalent to sigma y that means sigma 2 equivalent to sigma 3 because if you remember the oedometric sample or basically, the consolidometer sample this is how you will apply the load sigma v.

Now it is confined in both the directions. Now since it is confined in both the direction it is it can extend or it can contract or it can expand and contract only in the vertical directions only. So, this is where basically, the delta v by v comes which is equivalent to delta H by H because if it expands in the horizontal if it expands in the vertical direction then there is a change in the reading delta H that is corresponds to or that corresponds to the change in the volume of the sample because it is confined in other directions.

So, obviously this stresses in these 2 directions are actually constant that means sigma x and sigma y are actually equivalent to one another actually equivalent to one another. So, since they are equivalent to one another so basically, we can write that delta sigma is equal to E dash into epsilon v. Now epsilon v is only in one direction so basically, we can write that as epsilon 1 if we consider the vertical direction to be 1 these 2 directions as 2 and 3.

That means our axises are like this 1, 2, 3. So, obviously sigma in this direction is the only thing that is changing the sample. So, epsilon dash into epsilon 1 or you can also write as delta sigma 1 is equal to E dash into epsilon 1. Now you can also think that basically, you can also come from this relation that epsilon 2 and epsilon 3 are actually confined by the metal rings because you see that epsilon 2 and epsilon 3 cannot expand.

Only the stresses in these directions can expand because if it is a linearly elastic soil then all the 3 all the 3 stresses could be interrelated through the strains. So, epsilon 2 and epsilon 3 has no chance or they are equivalent to 0 because the only fact that it is confined with metal ring on both the sides as you have seen in the oedometric sample. So, obviously epsilon 2 will be equivalent to epsilon 3 will be equivalent to 0.

So, I can write from the definition of epsilon 1, 1 by E into delta sigma 1 - nu into delta sigma 2 + delta sigma 3 because the stresses in these directions are not 0. Only the strains in these directions are 0. So, it comes out to be 1 by E into del sigma 1 - 2 nu del sigma 3 because again I have said that sigma 2 is equivalent to sigma 3 so that means del sigma 2 is equivalent to del sigma 3.

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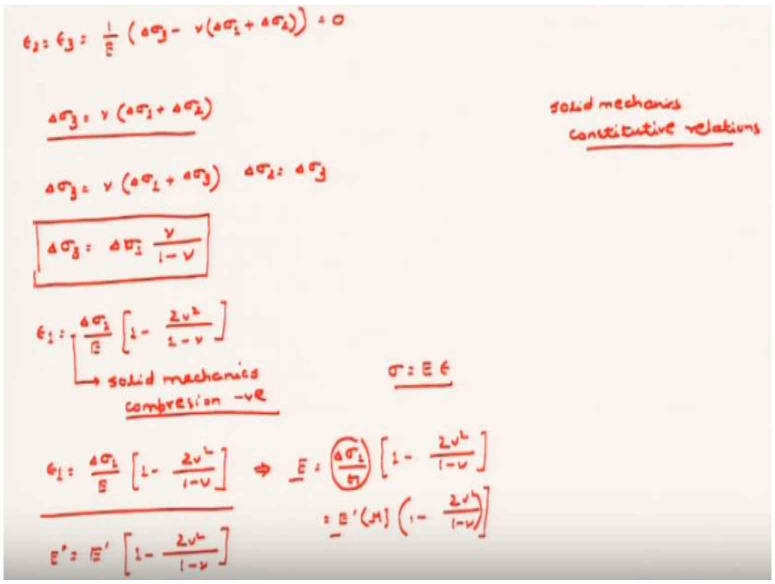

So, if this is the case then epsilon 2 is equal to epsilon can be also written as 1 by e into delta sigma 3 - nu into delta sigma 1 + delta sigma 2 which is equivalent to 0 because as I have said previously that the strains in the sigma 2 and sigma epsilon in 2 and 3 directions equivalent to 0 because their the sample is confined by metal rings on both the sides. So, delta sigma 3 must be equivalent to nu into delta sigma 1 + delta sigma 2.

So, from here we get that delta sigma 3 is equivalent to nu into delta sigma 1 + delta sigma 3 because delta sigma 2 is equivalent to delta sigma 3. So, from here you can write delta sigma 3 is equal to delta sigma 1 nu by 1 - nu. So, if you replace this in the equation of epsilon 1 you will get epsilon 1 is equal to delta sigma 1 by E into 1 - 2 nu square by 1 - nu. Now remember we are deriving all this from the definition basic definitions of solid mechanics considering compression is positive because we will always take in case of soil compression is positive.

So, that is why I have ignored a minus sign here because if you really are to observe or if you really want to see that from the basic definition of soil mechanics how you can derive this you will always see that there is a negative sign in front of this. There is a negative sign if basically, you consider from the basic definitions of solid mechanics where you take compression negative but this all are from definitions of from the definition of soil mechanics but we take the same convention of soil mechanics so that is why epsilon 1 is always said to be positive.

So, 1 - 2 nu square by 1 - nu. So, if this is the case you all know that delta sigma 1 delta sigma 1 by E or if you compare these 2 equations then you know that epsilon dash now will be equivalent to sorry epsilon will be equal to epsilon dash into 1 - 2 nu square by 1 - nu because epsilon is nothing but actually equivalent to sigma is equal to E into epsilon okay so this E goes here is equal to delta sigma 1 by epsilon 1 into 1 - 2 nu square by 1 - nu.

From this it comes out and delta sigma 1 by epsilon 1 you can easily see it is oedometric modulus because from your basic from the basic diagram of an oedometer or basic from the consolidometer you can see that sigma 1 is the stress applied in the vertical direction epsilon 1 is the volume change in that direction so obviously it is a one-dimensional consolidation and hence it is nothing but equivalent to the oedometric modulus.

So, that is why we can write this as E dash or M whatever you want and 1 - 2 nu square by 1 - nu. So, within an elastic range or if you consider an elastic range this is what should be the basic relation between a Young's modulus a oedometric modulus and the Poisson's ratio. Now for a particular Poisson's ratio you can also find out the Young's modulus and oedometric modulus from this relations.

Now one thing is that this is a bit high end problem because this all requires some basic things which are termed as the basic definitions of solid mechanics and one of the things that it requires is the constitutive relations. This is just a demonstrative problem that how to generally model or to generally model a soil mechanics from an experimental from an experimental to a computer program. So, this is how to use the definitions of stress, strain, Young's modulus, and all those things to basically, predict the behaviour of soils at different stress levels and confinements.

This is only considering linearly elastic range similarly you can consider plastic range and you can go for the plastic behaviours of soil but this is an entirely this is an entirely different domain. This is only just to demonstrate that how basically, the shear strength relations of soils can also be used for modeling behaviours. Now coming back to our original basic shear strength relationships, we will now cover a very important problem regarding stress paths.

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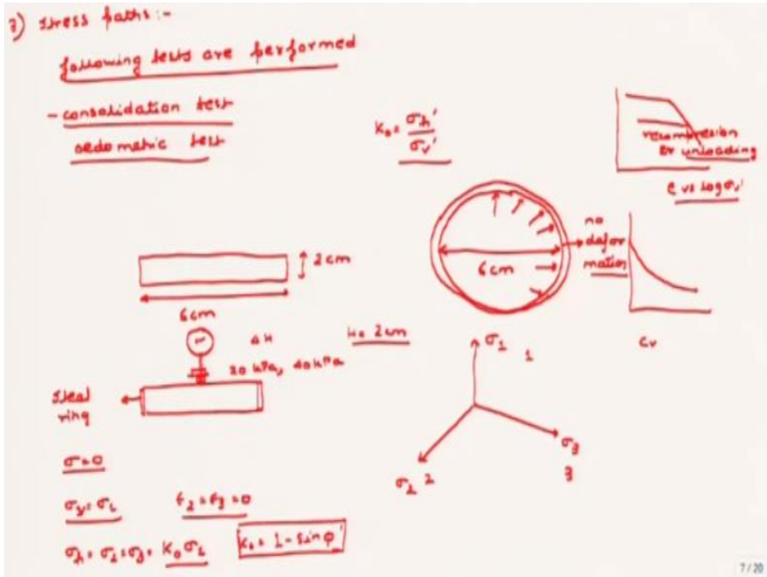

Now this stress paths problem is also a discussion problem and not a actually a solving type of problem. So, here basically, we will discuss that how in different types of tests the stress paths actually differ from for a particular material. So, it has been asked that the following tests are performed one of which is termed as the consolidation test. This can also be termed as an oedometric test because now you know that an oedometer is equivalent to consolidometer so in this case they are asked to find out that how the stress paths is how the stress paths actually look like in a pq space.

Now let us start from the basic definitions or from the basic diagram about how you can define a consolidation test or an oedometric test. Now as I have said previously that this is a oedometric sample that is a height of let us say 2 cm and this width is of 6 cm. Now from a 3D view it looks like a circle of dia 6 cm and a height of 2 cm.

Now this is placed in a steel ring like this and a dial gauge is applied dial gauge is observed to observe the change in delta H where H is the original height of 2 cm generally and what happens is that as the dial gauge reads different loads are applied on the sample so there is a load here that will be applied to the sample and as different loads are applied to the sample let us say it vary from 20 KPa, then 40 KPa and like that you get you observe the change delta H by H and then you plot all this consolidation curves.

The square root of time the C v then basically, the e versus log p. This (()) (20:14). Now similarly just the same case when basically, you are going for a recompression or basically, when you are going for a swelling part what you do is that you remove the load. You remove the load and you observe that what is the change in the dial gauge**.** So, you get this recompression part recompression or you can be termed as unloading part. So, this is how in general an oedometer behaves.

Now see in the very beginning when basically, you are starting the test then in that case let us see that whatever stresses in the soil. So, when it is first made then basically, the stresses in the soil are actually 0. There is no stress in the soil. Now let us consider 3 directions. So, let us say this is sigma 1 just like the previous sin dimensions we will consider sigma 2, sigma 3 means this is 1, this is 2, this is 3.

So, now as I have said in the previous problem also only sigma 1 is the governing factor in case of an oedometric modulus because you can see these are steel ring this is a steel ring. So, it bounds the surface like this. So, obviously no deformation is allowed along the horizontal direction no deformation, only vertical deformation is allowed. No deformation in this directions. If this is the case then when you apply when you apply the first load let us say that first load is equivalent to sigma 1 in the horizontal direction in the vertical direction sorry then some changes in the horizontal direction is observed though the deformation is 0 means the strain in epsilon 2 and epsilon 3 are supposed to be 0 but actually the stress exist in that direction because if the strain is because when you apply a vertical load then due to the Poisson's ratio of the soil as you all know from the definition of soil mechanics it will try to expand in this directions but since it is (()) (22:16) so basically, stresses will develop which is an internal force.

So, obviously sigma h is equal to sigma 2 is equal to sigma 3 will be equivalent to K 0 into sigma 1. Now do not confuse this K 0 with the definition of earth pressure at rest. This K 0 may be some different K 0. Though it is often said that this K 0 is actually is equal to 1 - sin phi dash but let us for the time being assume that this K 0 is just a coefficient that this K 0 is just a coefficient that actually is the ratio of the horizontal stress to the vertical effective stress okay. So, if this is if this is true then this is equal to K 0 into sigma 1.

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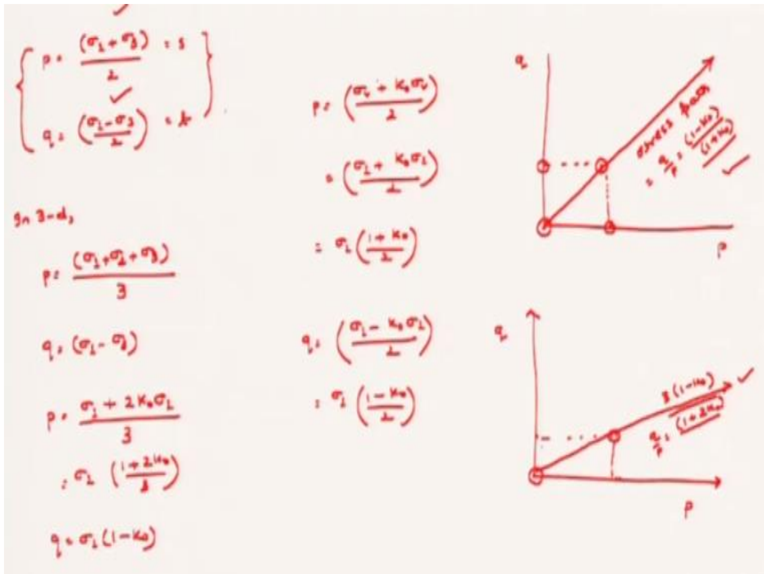

So, now from our definition p as you know is sigma 1 + sigma 3 by 2 while q is equal to sigma 1 - sigma 3 by 2. Now this is valid for a 2-dimension case. In 3D p can be defined as sigma 1 + sigma 2 + sigma 3 by 3 while q is equivalent to sigma 1 - sigma 3. Now generally these are termed as the s t ten sets. This is also defined by the name of s t while this is defined as the name of p q in that case.

So, since this part is already covered in our syllabus so we will consider from the basic definition this part is already covered from this definition of stress path so we will consider this pq rather than this pq but I will show for a demonstration problem that how this pq also looks like. So, p now you know is sigma v + K 0 into sigma v by 2 or it can be written as sigma 1 + K 0 into sigma 1 by 2.

So, it is equal to sigma 1 into 1 + K 0 by 2. So, in the stress test if this is p if this is q your initial position was 0, 0. Now at a particular stress sigma 1 let us say this is my p point. Now what about q? q is equal to sigma 1 - K 0 into sigma 1 by 2. So, it is equivalent to sigma 1 into 1 - K 0 by 2. So, it will be somewhere here. So, this is the point join this line. So, this is the stress path and the slope of this stress path is obviously q by p which is equal to 1 - K 0 by 1 + K 0.

So, the this is how basically, you can define the stress path definition for an oedometer. So, for an oedometer the stress path ratio is always in the ratio of 1 - K 0 by 1 + K 0. Now for this 3D for this for the 3D case of pq we can consider this as equivalent to sigma 1 + 2 into K 0 into sigma 1 by 3. So, sigma 1 into 1 + 2 K 0 by 3 and q will be equivalent to sigma 1 into 1 - K 0.

So, in this case your p should be somewhere here while your q should be somewhere here so it will be at this point. It follow a slope of q by p equivalent to 1 + 2 K 0 by 3 1 - K 0.

So, 3 into 1 - K 0 by 1 + 2 K 0. So, that is where is the stress path ratio should be or basically, that is where is the slope of the line is but for the time being you should consider only this path because this is the because this is the path that is taught in the syllabus. So, basically, this type of questions are very common that how what should be the stress path in case of a consolidation test and oedometer test and how do you find out from the basic from a basic knowledge of the test that the stress how should the stress path be.

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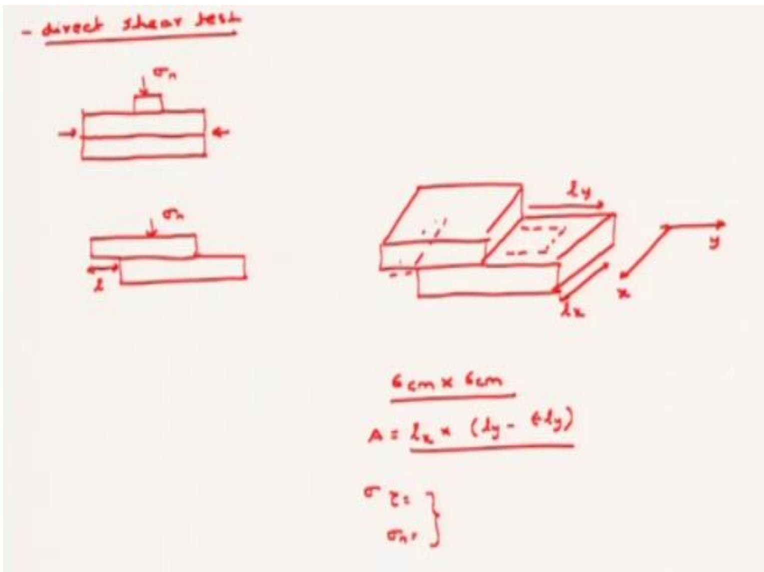

Now for the next part it is asked the stress path of a direct shear test. Now before you start before you start moving over the direct shear test let us discuss a bit about the direct shear test and how the stress path should be just like the consolidation test. So, as you know in case of a direct shear test there is a shear box that is again confined in this 2 directions and a load is applied from here. This is the normal load. This is the sigma n or the normal load.

Now this is where the box is supposed to slide so obviously the box will slide along this direction and this is the sigma n. Now a common question that is often asked in case of a direct shear test is basically, when the box lies then there should be a area correction. Yes, obviously there should be an area correction because you see that this is the length that is to be covered or this is the length that is to be neglected because this length has already been sheared off by the sample.

So, generally what is said is that since from a 3-dimensional view if you see the box it looks somewhat like this so this is where the soil is kept and this is the soil mass that is sheared so this part l along let us say if this length is let us say along x and if this is along y then this ly as you can see is changing while lx is constant. Now from the from the basic from you know that basically, the shear box is of a high is has a dimension of 6 cm into 6 cm generally except under only special circumstance where basically, you have to consider bigger shear boxes.

So, in that case what you have to do is that a general way to correct the area is that you have to take lx as it is and ly should be minus epsilon where epsilon is the strain or basically, the strain that you observe into ly. That is how your general corrections would be. Now remember that this should be corrected not only for the tau that means the shear stress but also at the same time for the normal stress because you see that the area under the normal stress is also decreasing.

So, often in certain books it is ignored that if you do not have to calculate for normal stress but actually you have to calculate both for the shear stress as well as for the normal stress for the both tau and sigma n that how your direct shear test is going that how your area correction is going to come into the factor. So, considering all this now let us move back to the move back to the point where basically, we were discussing about the stress paths. So, let us again consider from the basic beginning that how the direct shear test actually gives the definition of the stress paths.

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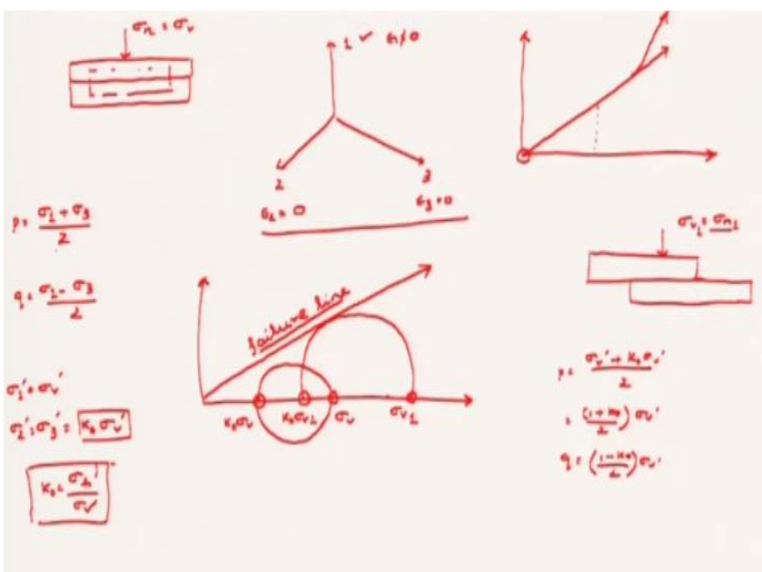

So, in the beginning what you do is that this is how you keep the soil sample as. So, this is the soil sample in between and the soil and the sample is not subjected to any stresses. Now what you do is that so it starts from a point 0, 0. Now what you do is that you apply a normal stress sigma n. So, in this case in order to actually compare with the previous definition of consolidation so we will again consider this as 1 this as 2 and this as 3.

Now see this test is almost equivalent to consolidation test because here also the confinement is applied on both the sides to a steel the steel body because you see that this portion is actually steel body. So, the soil mass is allowed to move only in the vertical direction but no displacement along both the horizontal direction that means again only one or epsilon 1 is allowed epsilon 1 is not equal to 0 while epsilon 2 and epsilon 3 are both equivalent to 0.

So, the soil mass is not allowed to move along both the directions. If this is the case then again from your basic definition of consolidation stresses will develop from basic definition of solid mechanics I am sorry that stresses will develop both in the second and third direction due to the Poisson's ratio. So, again here p will be equivalent to sigma 1 + sigma 3 by 2 while q will be equivalent to sigma 1 - sigma 3 by 2.

Now let us consider a Mohr circle and see that how it comes out to. So, this is the Mohr circle. So, the initial stress or basically, when you apply the initial stress then immediately along this direction so sigma 1 is equal to sigma v if you apply then immediately along sigma 2 and sigma 3 you will see K 0 into sigma v again where K 0 is the ratio of the horizontal stress to the vertical effective stress.

Remember that all the stresses that we are considering actually here are all effective stresses. So, these are all effective stresses. Now just ignore the fact but these are all actually effective stresses because in consolidation effective stress is the only thing that matters. So, obviously these are all effective stresses. So, here also K 0 into sigma v dash is the effective stress. Right now, we are only considering about effective stresses.

So, sigma 3 dash 1 point is here sigma 1 dash sigma v dash 1 point is here. So, your Mohr circle looks somewhat like this. So, this is K 0 into sigma v while this is sigma v. Now as you apply shear what happens is that this circle at the end of the shearing let us consider that the end of the shearing what happens. So, at the end of the shearing this is what is the condition. Now you see that why I discussed about this area correction because you see this sigma v or sigma n that was present has now changed its position because the area has been changed.

So, whatever the load was at the beginning it does not matter. The area now has changed. Similarly, the stress has also now changed. So, obviously this point has shifted somewhere here. So, now let us consider this n as sigma n1 and sigma v 1. So, this is equivalent to K 0 into sigma v 1 and this point as sigma v 1 this stress. So, this is the final one and if you join these 2-line considering that it is a sand this is the failure line.

So, this is different from the so this is so basically, what now happens is that the stress now has changed in the third sigma in the vertical direction so since the stress has now changed in the vertical direction so similarly the stress in the horizontal direction has also changed. So, K 0 into sigma v1 and sigma v 1 and now basically, when it touches the Mohr circle when the failure line touches the Mohr circle then you consider it as the failure point.

Now if you do the same thing in case of a direct shear test then you will see that it will start from somewhere here then it will go because here also again p is equal to sigma v dash + K 0 + into sigma v dash into 2 is equal to 1 + K 0 by 2 into sigma v dash and q similarly is 1 - K 0 by 2 into sigma v dash. So, it should again follow a line somewhat here but remember that this line is going to change accordingly because this sigma v dash is also going to change.

So, somewhere this point this line may go like this okay. So, in case of in case of oedometer and in case of direct shear the only problem the only problem is that the sigma v dash is constantly going to change. So, obviously this points where basically, in that case the sigma v dash whatever you are plotting the sigma v dash there also is going to constantly change but it is going in linear increment but here it may or may not go in linear increment and this K 0 may vary depending on the fact that the area is also changing okay.

So, that is why basically, this direct shear test is often direct shear test in case it may not be 1 - K 0 by 1 + K 0 but generally for the general considerations we will consider it at the time being for 1 - K 0 by 1 + K 0 to be the slope. Another important factor that you have to consider is that in this fact we had totally ignored the effect of consolidation but in direct shear a consolidated drained test is also possible.

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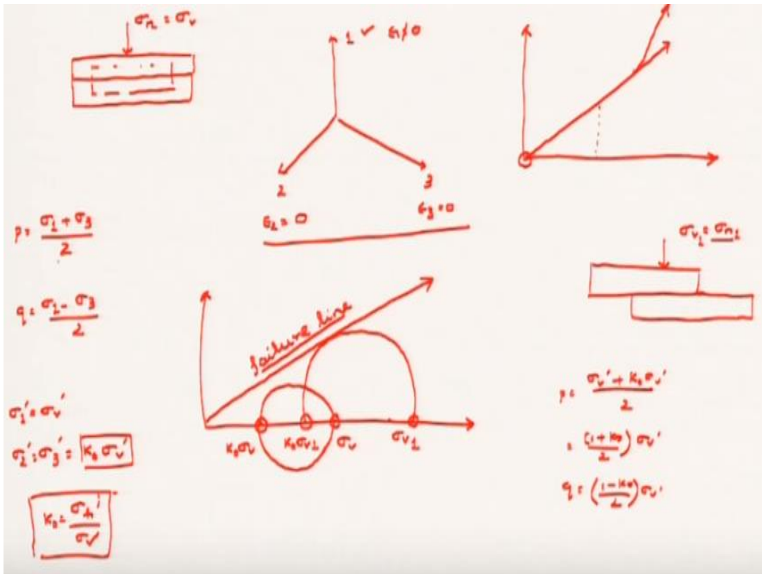

So, if you take into consideration the effect of consolidation then in that case it is different from an isotropic compression because in this case again if you consider the if you consider the effect of consolidation then you just apply the load here sigma n which is equal to sigma v and you apply the consolidation to occur. Now obviously you do not know what is the horizontal or what is the what are the horizontal stress.

You do not know that you do not apply any horizontal stress. Obviously, the horizontal stress will be equal to K 0 into sigma v dash. So, q in the earlier case q was said to be sigma 1 - sigma 3 by 2 for a triaxial. In general case q is equal to sigma 1 now for a triaxial case what happens is that you know that in case of an isotropic compression sigma 1 is equal to sigma 2 is equal to sigma 3 that is what I have said in the very beginning.

So, obviously in this case what happens is that sigma 1 - sigma 1 by 2 is equal to 0. So, the line in case of a triaxial is actually moves along the p but in this case what happens is that q is equal to sigma 1 - K 0 into sigma 1 by 2. So, it is equivalent to sigma 1 by 2 into 1 - K 0. So, obviously the line would move somewhere in this direction because now it is a ratio this sigma 1 dash and then along the shear it moves along this direction.

So, this is for a consolidation line this is for a shear. So, these are the 2 definitions that you have to that you have to remember that what happens basically, in case of a that what that how basically, in case of a direct shear the stress path is going to behave. If it is a if it is directly a shearing line then obviously it should go directly from a shear point from the point 0, 0 but if basically, you allow a consolidated drained test for example a CD test in case of a direct shear then obviously there is a consolidation line and from that point of the stress let us consider from this point of the stress it will move along the shear line.

Now in case of a direct shear as you have seen that you get a plot of shear stress versus the strain so it somewhat looks like this for a particular normal stress. So, this is already known to you. So, I am not discussing all these facts and one important point regarding the direct shear test is and one important point regarding the direct shear test is that remember that obviously in direct shear test since it is confined from all the direction this I am again telling that since it is confined from all the directions so K 0 or the effect of K 0 should always come into consideration in case of a direct shear test. Now the third type of test where basically, the stress path is asked to draw is termed as the standard CD triaxial test.

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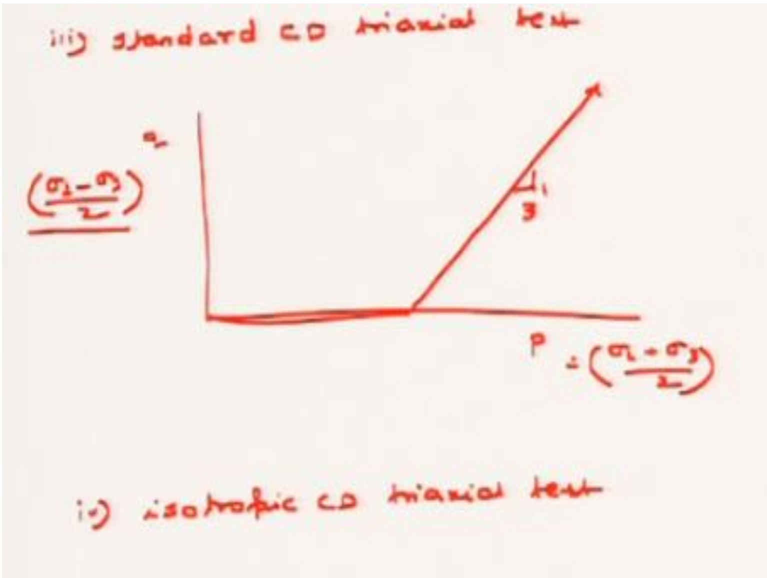

Now this has already been discussed so I am not going in details because you already know that this goes in the ratio of 1:3 and if it is a consolidation then also I told you many times that it will go in the in this flat portion and then it will go in the ratio of 1:3 so in a pq space where p is equal to sigma 1 + sigma 3 by 2. This is sigma 1 - sigma 3 by 2. So, this I am ignoring totally and the next one is the triaxial isotropic CD triaxial test and the fourth one is termed as an isotropic CD triaxial test so this has already been discussed because this is the flat portion and why this is the flat portion this is all this also I have explained here.

So, in case of an isotropic CD triaxial test what happens is that the q is always 0 that means there is no shear stress generated theoretically but practically many problems may arise because of the anisotropy in the sample. So, the isotropic CD triaxial test basically, gives q equal to 0 that is the only value of p and basically, from a standard CD triaxial test the q:p ratio always goes in the ratio 1:3 that has already been discussed in the lectures.

Now the last part that basically, we are going to cover is basically, we have almost discussed almost every part that we have discussed is basically, covers the effect of foundations the effect of consolidation the shear strength of soils and many important aspects like permeability compaction and regarding compaction I also showed one practical problem that how basically, you should assess the degree of compaction when basically, a soil sample is given to you.

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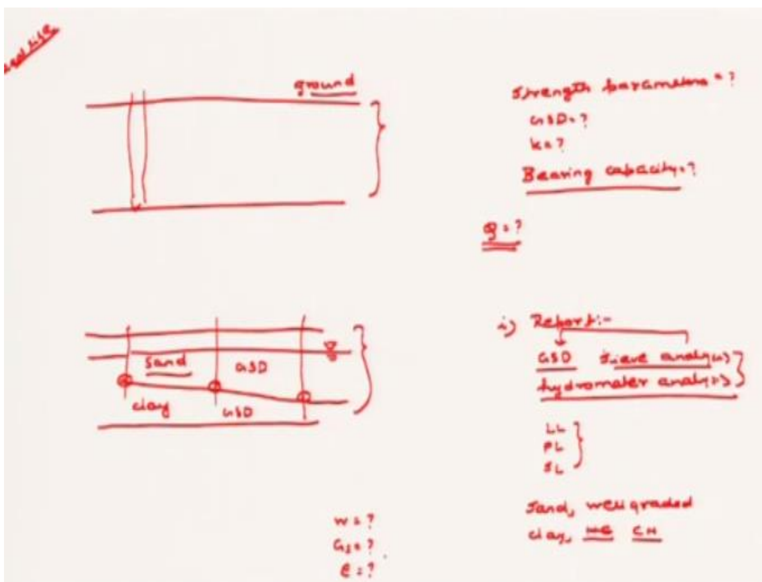

Now in real life actually what happens is that now this is also a demonstrative problem how basically, you can combine all these facts together in one part. So, in real life what happens is that suppose a ground is given to you and depending on that you are asked to perform or basically, you are often asked to find out the strength parameters GSD, permeability, and one of the most important factors which is termed as the bearing capacity.

Often ignoring this term of bearing capacity people will ask that whether the soil let us say will basically, adjust or basically, it can take a load of q or not. Now how do you say that whether the soil will take a load of q or not? The first thing that you have the first thing that people does in this case is basically, they perform a bore hole test. Now what do you mean by a bore hole test often talked about is the bore hole test means nothing but actually digging the soil profile to a particular depth.

Now when once you dig the soil profile to a particular depth you actually know what type of soil life you are going to encounter. Now this I have already told in consolidation problem that it is you have to dig almost bore holes depending on the engineer's judgment or basically, from the point of view of the engineer you have to take different bore holes means you have to construct different bore holes depending on the test site.

Now let us say I have constructed 3 bore holes along this length and I have observed that the layer of clay actually is here at this point here at this point and this point and said that you join this clay layer and basically, so this is the layer of clay while this is the layer of sand that you have observed. The water table also you can observe from the bore well because you know from what point the soil is going to be moist so depending on that let us say the water table is somewhere at this point.

Now apart from all this once you have done all this you also know now from the laboratory test you perform a grain size distribution on this soil you perform a grain size distribution on this soil. So, obviously you know what are the grain size distributions now. Now depending on the grain size distribution, the first thing that you have to do is that the first thing in the report that you have to prepare is that you have to perform a grain size distribution.

Then you have to go for a hydrometer analysis. You have to go for a sieve analysis then you have to go for a hydrometer analysis. Combining these 2 you will get the grain size distribution. So, depending on the grain size distribution and according to the IS code classification now you have to define what is the soil is. So, for that effects of fine grained soil then you have to go for LL PL then SL are the shrinkage limit or the Atterberg limits.

Once you define all these things then let us say that according to the definition sand is obviously as we know is a coarse grain and let us say that this is a well graded sand while the layer of clay says to be high plasticity clay so obviously HC or CH that is the high plasticity clay in the Atterberg. Now once this report has been once you have defined that what your soil is at the 2 point the next thing that you define is basically, the water content at the soil the specific gravity of the soil solids and from this relation what is the water what is the void ratio and the degree of saturation.

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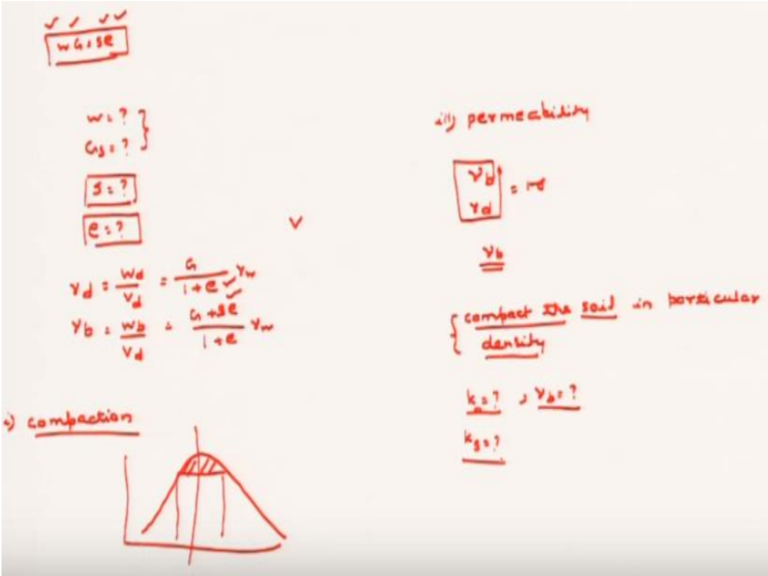

So, from all this you will get these 4 parameters are very important in real life. So, the void ratio can never be found out actually. Now this is the this is the trick. The water content you can find out where specifically because water content you can do it as most of the people do is that you can dry it in oven and take out the water content. The specific gravity of soil solids there is a test for specific gravity of soil solids that I am not going to discuss here but you can the degree of saturation similarly you cannot find out neither the void ratio.

Now these 2 can be found out from if you can remember from the basic soil relations gamma d and gamma b. Once you dry the soil you know what is the volume of the soil then basically, this gamma d can be found out because gamma d is nothing but the weight by the weight of the dry soil by volume of the dry soil. Gamma b is weight of the bulk soil by volume of the bulk soil. Now if you remember the relations correctly then this is G by 1 + e into gamma w and this is G + s c by 1 + e into gamma w.

So, from both these relations you can derive the e and s. So, you find out the relations of w, G, S e. Now you know exactly that what are the different saturation degree of saturations for this sand at different levels for this clay at the different levels. Once all this has been prepared then the next thing that you have to do is basically, you have to go for the compaction.

So, depending on the compaction that on which side of the soil now this is not a necessary case but suppose you have to ground improve the soil as I have said that in certain case you have to make improvement in the soil so in that case as I have said that depending on the dry side or the wet side depending on what your structure is you have to define then the dry side or the wet side of the compaction.

So, depending on whether you want a low permeability soil or a high permeability soil you will go for a dry side you will go for a wet side or the dry side of the compaction. Now the third thing that you have to do is basically, about the permeability. So, once you know or once you fix what is your density of the soil is once you fix what is your gamma b and gamma d is this 2 are to be fixed now you know that what is your unit weight of the soil.

What is the unit weight that means what is the gamma b is and once you know what is the gamma b is from this from the point of gamma b you can compact the soil in particular density and then from there you can find out what is the permeability is. So, that permeability is varied valid for that particular gamma b. So, that permeability is particularly valid for that gamma b. Now you can perform a concentrated test like let us say you have a layer of sand at the top so you are going to perform a concentrated test there while basically, for the layer of clay there as you know you have to perform a falling head test. So, the then basically, the case K CL K clay and K sand is found out.

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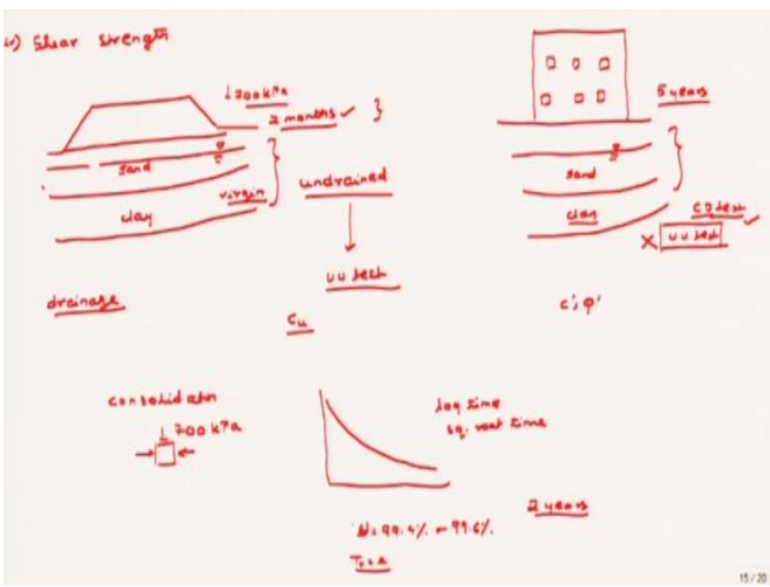

Now the next part that to move is basically, the shear strength and consolidation. Now these 2 parameters are going to tell you that whether your soil is going to support the given load or not. So, from the point of shear strength view from the point of shear strength what you have to do is that you have to assess that what type of situation is there is in the field. So, let us say that you are going to construct an embankment on that soil.

This is the layer of sand. Now the given plate for the embankment now the clay is virgin or let us say that it has not been subjected to a a consolidation before and this is the water table and the embankment is to be constructed or let us say an extra load of 700 KPa is to be imparted let us say within 2 months. In the period of construction during the period of construction what the constructor said is that that basically, I am not allowing any drainage.

Now you should know by this time that when you are saying that you are not allowing any drainage means you are not allowing any consolidation. So, if this point of fact this entire system now behaves as an undrained system. So, the most suitable test immediately your judgment will be the most suitable test that you have to perform is a UU test. That means you are not allowing any drainage.

So, you are just applying load after load and you have to see that whether that is able to take that load or not. So, in that case basically, depending on the strain rate basically, you have to perform a UU test. But suppose let us say I have to construct this I have construct building on this soil and this takes let us say 5 years to construct now in this process of course allowing a drainage to occur. Now this is a general judgment that you are of course allowing the drainage to occur.

So, if basically, you are allowing the drainage to occur then in that case you have to perform a CD test whether or not the time is sufficient or not but a UU test will never be able to judge a situation in this case. You have to perform a consolidation drained test in this case in order to get that what are the correct parameters like C dash and phi dash. As you know that in this case the parameter is CU.

So, depending on whether the soil is drained or undrained you have to judge accordingly that whether the whether you have to take a drained test or an undrained test or not. Depending on whether you are allowing consolidation or not you have to check that whether the sample is actually allowing drainage or not. So, let us say suppose for example clay. Clay takes a very high time to clay takes very long time to actually drain.

So, obviously when basically, the building the period of construction is rapid let us say like 2 months or 3 months in that case it is obvious that the clay will not drain and you have to consider the UU case. Now a general behaviour or a general expression that whether the clay will be draining within 2 months or not is I have showed in the example of consolidation. So, that is where the effect of consolidation actually comes out.

So, what you do is that you actually extract a sample of clay subject it to the same stress conditions let us say the embankment load is 700 KPa. So, you subject it to a consolidation of 700 KPa and observe that what is the coefficient of consolidation let us say from log time method or square root of time method. Now you will know exactly that I have said like t has to be 99.4% or 99.6% U then only the total consolidation is supposed to end.

So, if this is reached where basically, the coefficient of T v is 2 if this value is reached then only you say that the consolidation has ended. So, if the consolidation has ended then only you can say that basically, your means you can perform you can go for a consolidated drained test. Now if you see that let us say that if the construction is happening in the period of 2 months but however your time takes 2 years to complete the consolidation take 2 years to complete then in that case it is obvious that situation to be assessed is an undrained and obviously it is an UU test rather than a CD test. So, this is from where the effect of consolidation will occur.

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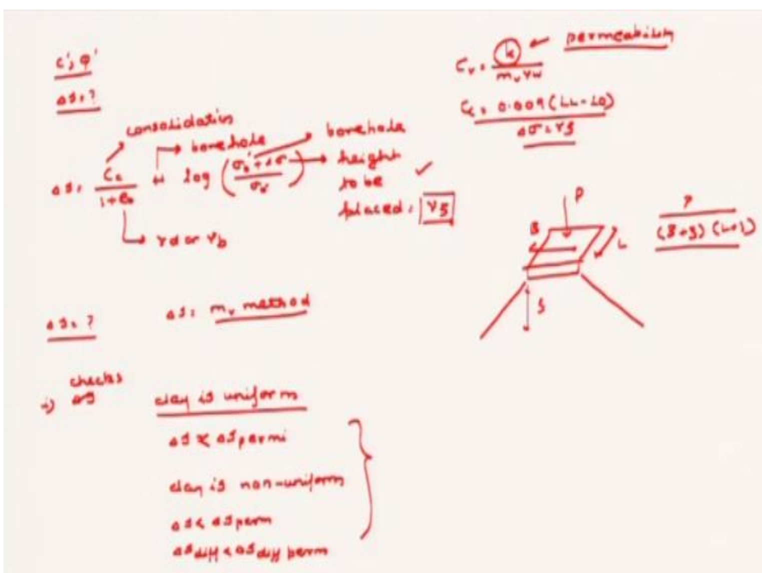

Now the now let us see that you obtain the parameters C dash and phi dash and from the consolidation you have obtained that what is the total settlement because you already know the stress levels and delta S as you know is C c by 1 + e 0. Now you see e 0 you have already founded out form here into H you know what is the height of clay layer from the bore hole test log sigma 0 dash + delta sigma by sigma 0 dash.

See this you will get from consolidation test this you have already obtained from gamma d or gamma b this you have already obtained is the from bore hole test and this also from bore hole and this is the height to be placed. This is gamma into z. So, you have obtained every settlement calculations. Now once you have obtained all these every settlement calculation you know now that what should be the settlement under the permissible load.

If the settlement since you have considered a virgin clay layer so basically, I have considered that delta S is actually only available for the virgin consolidation and not the preconsolidation part. Now one important aspect you have to remember here is that the clay layer maybe of varying thickness. That I have also said that in case of varying thickness how you can use the effect of stress distribution you have to find out the effect of varying thickness.

Now this also this delta sigma also I have said it to be gamma into z. Now you know that depending if it is a footing that also I have explained if it is a footing basically, how the stress distribution is supposed to be so it is going to be P into B + z into L + z that z is this height this is B this is L so and this is the load that is coming is P. So, depending on that you have to find out delta sigma. So, all these have to be adjusted accordingly.

But the general formats would remain the same. So, once you know what is the value of delta sigma and delta sigma you have I have found out from log method. You can also find it delta sigma from as I have said m v method okay. So, any one of them is sufficient. So, once you find out delta S now as I have said you know what is your permissible settlement is. So, the first thing there are 2 checks that you have to do.

The first check that you have to do is that whether under the clay layer if the clay layer is uniform let us say the clay is uniform. Then in that case what you have to do is that what you have to suggest is that delta S whether it is within S permissible or not. If the clay is non-uniform then not you have to check that whether delta S is permissible or not but also whether delta S differential is less than delta S differential permissible or not. This is a very important part that you have check because you cannot allow detrimental effects to damage the structure.

Now you may ask one thing that you have to perform a lot of test in this case. Now often people what they do is that they will not go for all these tests like this consolidation and all those things what they will do is that instead they will perform a rapid consolidation problem and they can find out C v from the relation k by m v into gamma w. So, this is where the permeability actually comes in q.

Rather than performing a test like this you can directly go from C v by k by m v into gamma w and you can just assess if basically, soil samples are not adequate or basically, you can feel that the time is not adequate then you can go also for the general formulas that are available C v by k into m v into gamma w and from there you can assess out what is the value of but what are the values of coefficient of consolidation to be. Similarly, you can assess C c from 0.009 into LL - 10. You all know this formula. So, these formulas are helpful but a general soil investigation is always required in order to get the parameters correctly. So, once the settlement is checked now you have to check that whether the load is sufficient or not.

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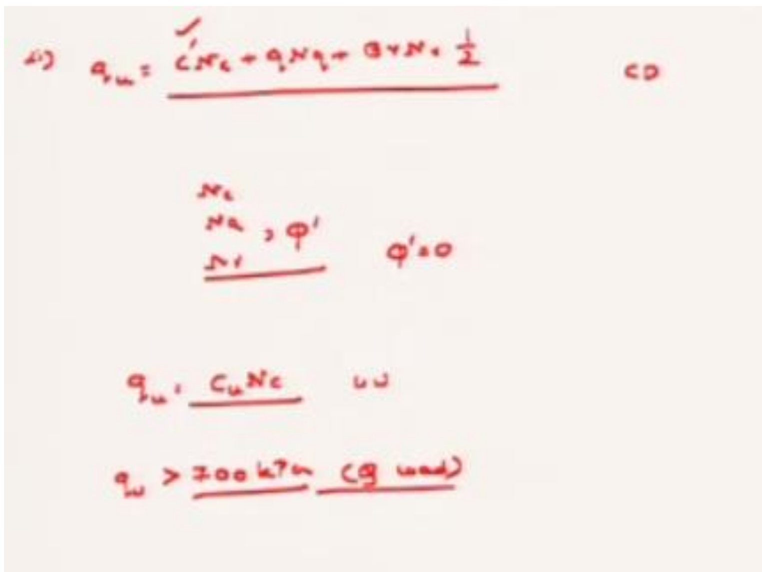

Now this has not been covered and this is a introduction to the foundation engineering so I will just cover a basic formula like the bearing capacity C into N c + q into N q + B gamma N gamma into half. Now here N c, N q, and gamma are basically, parameters that basically, are depending on N c, N q, and gamma are the parameters that depends on phi dash. So, this is where the phi dash comes into problem.

This C is the C dash if you consider a consolidated drained test while basically, if you consider an UU test then what happens is that as you all know phi dash is equivalent to 0 so this equation comes around to C u into N c. So, either using CD or UU you have to check that whether q u is greater than 700 KPa which is said is the allowable load or q load or not or the load that is to be subjected to the soil or not.

So, once all this facts are considered then only you can say then only from all these facts or from gathering all the basic soil mechanics definitions all the soil mechanics problem that you have assessed you can actually say that whether this q u whether the soil is actually sufficient taking shear strength and settlement or not. So, this is how in real life problems we actually assess a general way of using the permeability compaction then consolidation then shear strength in the field. So, this is where the tutorial session ends. Thank you.