Advanced Soil Mechanics

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Lecture – 23
Pore Water Pressure and Skemptons Equation

Welcome all of you, so we are now currently discussing shear strength of soils, in the last lecture we have seen shear strength of cohesive soils and we have noticed that there are 2 3 important aspects which we should be knowing very well to understand the shear strength behaviour of cohesive soils properly. One is the drainage condition which results in drained and undrained conditions and the other aspect is with respect to stress history.

Now with respect to drainage condition, let us say when we have a drain undrained condition existing the net result is the pore water pressure and as we have already seen in the past and when we have understood soil mechanics for the first time we know that voids the pore water present in it and the pore water pressure makes soil mechanics different from the other solid mechanics.

So, in today is lecture we will discuss a bit on pore water pressure now in advanced soil mechanics why we should bring the subject again? So, in undergraduate we have learned about pore pressure and the method of calculating effective stresses but in this we will try to understand a bit more in detail like if you need to predict pore water pressure normally it is measured.

Now in certain circumstances for solving some problems numerically you also need to have what is the undrained condition, what sort of total stress that gets developed and what is the kind of effective stress which is responsible for the soil mechanical behaviour. If that is the case, we should also need to have the knowledge of pore water pressure. So, we will get into some details of pore water pressure the method of prediction of pore water pressure and certain other aspects which are very relevant for module 3 and module 4.

Again, we are building up the platforms in all these subsequent lectures what we are doing is we are building platforms for a better understanding of module 3 and module 4.

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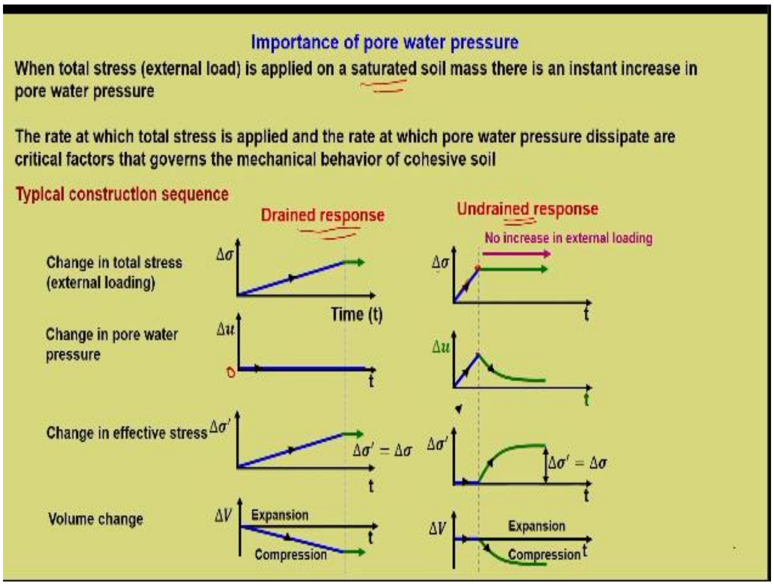

So, let us start with pore water pressure, so importance of pore water pressure, so when total stress. Now total stress we say it is the external load which is acting this also we have seen when we say total stress that is the stress acting on the medium. When total stress is applied on a saturated soil mass there is an instant increase in pore water pressure nothing new, the rate at which total stress is applied and the rate at which pore water pressure dissipate are critical factors that governs the mechanical behaviour of cohesive soil.

So, whenever we say mechanical behaviour we know that it is effective stress condition which is important but undrained condition do exists. Now the rate of loading governs what type of undrained condition it has and how much of the pore water pressure gets developed. Now the hydraulic conductivity of the soil determines how fast the pore water pressure dissipates. Now these 2 jointly determines what will be the effective stress which is gained in this soil at a given time.

And what will be the equilibrium effective stress and when it is going to reach because all these factors, it influences the mechanical behaviour or the stress strain behaviour or the strength of the soil. Let us see a typical construction sequence now for comparison have placed the drained response as well where the soil drains fast and there is no possibility of development of water pressure.

So, the first figure represents how total stress changes that means let us say there is a construction that is happening there is a foundation and there is a building load which gets added up or maybe an embankment load which gets added up. Now, what happens? The total stress increases with time. So, this is the pattern in which the total stresses increases now what is the instant response when soil masses saturated soil is loaded there will be pore water pressure.

Now since this is a completely drained response you can see that there is no development of pore water pressure, so it is 0. So, this can be treated as 0 now it remains same. Now what will be the change in effective stress since this is 0 whatever is the total stress that will be the gain in effective stress as you can see here. So, $\Delta\sigma' = \Delta\sigma$ this is equilibrium but any point along the curve also this is satisfied because it said drained response.

Similarly there is a volume change has happened and this is a typical compression case where it compresses and it follows the same sequence as that of the loading. Now let us see the undrained response let us see that this is the increment of total stress this is the loading and after this particular point it remains constant. So, let us see what happens for pore water pressure in the case of pore water pressure this is the instant response now this is an undrained response where water is not allowed to drain off fast.

So, there will be a mounting of pore water pressure since it is saturated there is an instant capture of $\Delta\sigma$ in terms of Δ u. So, whatever is the change in total stress that remains same in the case of pore water pressure up to this particular point t. Now for us as a layman, there is no further change in total stress that is there is no further change in loading. So, as a layman one would always understand we have already completed the loading on the soil mass till now nothing has happened to the system. So, there is nothing to worry.

Now this aspect gets defeated when you know or when you learn soil mechanics the reason is, whatever has to happen will happen after the completion of construction or after the external load gets completed. So that is what you can see here. Now after the construction got completed beyond that point the pore water pressure will slowly get disappeared and the excess pore water pressure it tried to achieve or it becomes close to 0. I am cautiously using the term close to 0 because there will be some differences.

So, whatever be beyond the construction period the pore water pressure starts reducing what is the net effect? You can see that the effective stress remains 0 till the construction got over because that is a saturated system and whatever loading it is all captured by pore water pressure and beyond the construction stage, you can see that the effective stress which starts increasing now effective stress starts increasing you can see that beyond certain time period.

Whatever is the total stress applied what is the load applied effective stress becomes equal to that. So, change in effective stress is equal to change in total stress after some time now what is the net effect of this as the intergranular stress goes on increasing as the effective stress goes on increasing this will induce some sort of volume change. So that volume change you can see till the construction got over there is no volume change which is happening.

Now all the volume change happens after that now this particular information is very vital to understand what is the importance of pore water pressure? So, at the end of the construction, it is important for us to assess what will be the kind of pore water pressure either it is measured or it is estimated. Now it is not possible to measure always because instrumentation for us is pretty costly. Only very very important projects instrumentations are done and it is monitored.

But in normal cases to understand the effect of pore water pressure it is not always possible to measure it rather it is estimated and that is the whole crux of today is lecture that we will try to see how we can estimate pore water pressure. This is what I am summing up there is no increase in external loading you excess that is excess pore pressure start dissipating and there is an increase in effective stress which results in volume change.

Now volume change is nothing but settlement as a function of time imagine in the case of an embankment or a building the settlement happens once the construction phase gets over. So, if we do not anticipate the kind of pore water pressure, if we do not estimate the kind of pore pressure that we anticipate in the field which is dependent on the soil characteristics then we may end up in trouble that is why there are lots of failures that has happened in terms of settlement.

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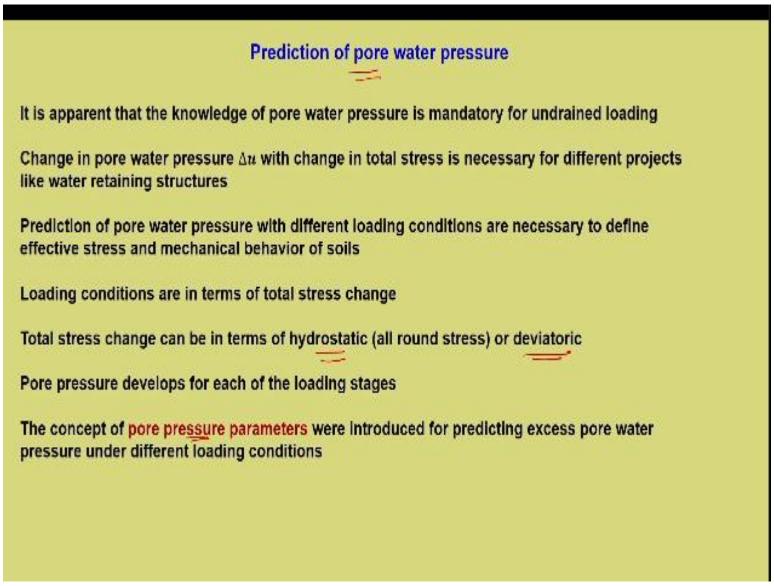

So, with that topic very clear that pore water pressure need to be estimated let us see how we can go for prediction of pore water pressure. So, it is apparent that the knowledge of pore water pressure is mandatory for undrained loading change in pore water pressure Δ u with change in total stress is necessary for different projects like water retaining structures where it is bound to be undrained and pore water pressure keeps changing.

Predictions of pore water pressure with different loading conditions are necessary for what to define the effective stress and then effective stress dictates what is the mechanical behaviour of soils Loading conditions are in terms of total stress change that we have to keep in mind, whenever we say loading on a soil that is always expressed in terms of total stress. Now total stress change can be in terms of hydrostatic which causes volume change and deviatoric which causes shear failures both are important.

Pore pressure develops for each of the loading stages the concept of pore pressure parameters were introduced for predicting excess pore water pressure under different loading conditions. So, the bottom line is we will be introducing something known as pore pressure parameters for predicting the excess pore water pressure corresponding to whatever is the loading condition. Now those who have understood this concept of pore pressure parameters still I would suggest that you go through this in detail because this chapter and this discussion is important in stress pat as well.

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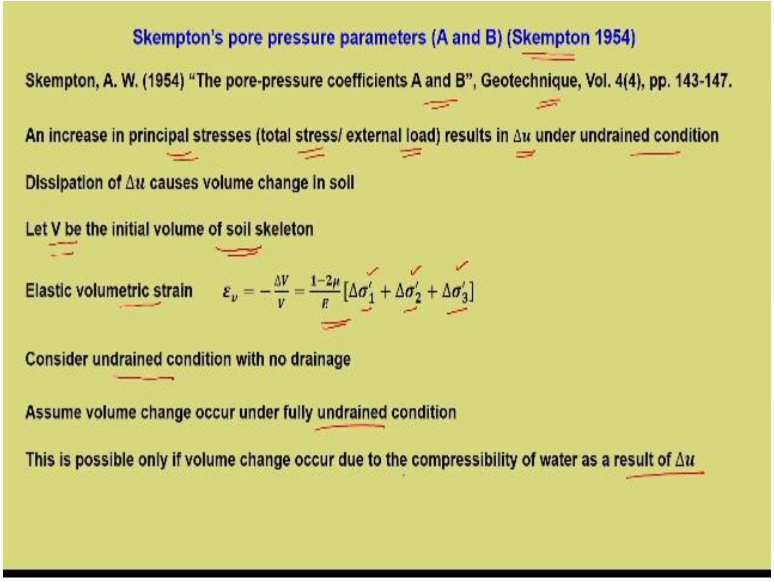

So, having known that pore water pressure parameters are important, let us see Skempton pore pressure parameters defined A and B and this is age old paper in 1954 it is published by Skempton the pore pressure coefficients A and B in geotechnique in 1954. But over these all these periods, I think this still remains a fundamental aspect in many of the soil mechanics problem. So, it is important for us to know what was the genesis or where it all started with.

So, Skempton pore pressure parameter A and B becomes very important there are different variants of this but the concept remains the same. An increase in principle stresses which is defined in terms of that is total stress and external load whatever B that is defined in terms of principle stresses. So, any increase in principle stresses results in Δu under undrained condition. Dissipation of Δu causes volume change we have known this.

Now for deriving this A and B we need to do some indirect approach, let us see what it is, let V be the initial volume of soil Skempton when we say it undergoes volume change or there is sport a pressure it is all associated with some sort of strain. We know the elastic volumetric strain

$$\epsilon_V = \frac{\Delta V}{V} = \frac{1 - 2\,\mu}{E}\ [\Delta\sigma'_1 + \Delta\sigma'_2 + \Delta\sigma'_3].$$

This is a drained condition characteristic where effective stress characteristics are used, so volumetric strain is always associated with drained condition. So that is why $\epsilon_V = \frac{\Delta V}{V}$. So, this is the expression and this we have seen already in the previous lecture. Now, this we know is drained conditioned response consider undrained condition with no drainage now this is what we actually need.

Assume some sort of volume change occur under fully undrained condition now, this is not a possible condition because under undrained conditions only pore water pressure develops but we are adopting an indirect method for obtaining this pore water pressure. So, what we are doing is we are trying to assume make an assumption that there is some volume change that has happened in an undrained condition.

Now what could be the possibility of such a volume change now this is possible only if volume change occur due to the compressibility of water which is associated with Δu now there is Δu water, let us assume that water is compressible and because of this compressibility property the volume change can happen during undrained condition now water is incompressible it does not undergo volume change.

So, it that concept remains same and it is very well valid but here we want to understand we want to determine what is Δu, so that is why we are making such an assumption. So, we associate that is assumed volume change in undrained condition to compressibility of water.

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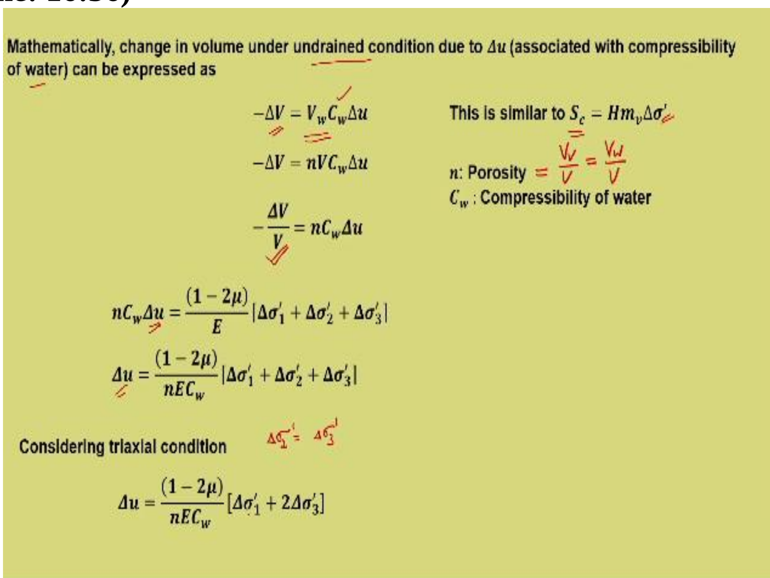

So, mathematically change in volume under undrained condition due to Δu which is associated with compressibility of water can be expressed $as - \Delta V = V_w\, C_w\, \Delta\, u.$

-ΔV is the water volume change that has occurred under undrained condition which is associated with compressibility of water now, compressibility of water is C_w and V_w is the volume of water in the voids.

Since it is saturated volume of voids remains equal to volume of water Δ u is the pressure which creates this volume change. Now you may be wondering from where this expression has come, this is similar to consolidation settlement equation you might be remembering this equation very well. This is consolidation settlement $S_c = H\, m_v\, \Delta\sigma'$. $\Delta\sigma'$ is the one which causes this settlement.

What is it settlement? It is nothing but the volume change here it is one dimensional so, it is ΔH, instead of ΔH here it is ΔV. So that is C corresponds to ΔV here H is the original height here it is what is the original volume and here we are associating the volume change to only the compressibility of water, so where is water water is in voids. So, volume of water which is equal to volume of voids becomes the original volume.

So, H corresponds to volume of water here, m_v is coefficient of volume compressibility in consolidation, the same property which represents compressibility of the soil skeleton here it is C_w which is exclusively related to compressibility of water and what causes this settlement that is $\Delta\sigma'$ and consolidation here the volume change is caused by Δu please remember this is assumption.

Now we can also write V_w we can replace V_w by total volume how because n is porosity, n is the porosity, C_w is the compressibility of water and $n = \frac{V_v}{V} = \frac{V_w}{V}$. So, you can always write $nV = V_w$. So that is what has been replaced here $-\Delta V = nVC_w\Delta u$. So, we can write $-\frac{\Delta V}{V} = nVC_w\Delta u$ Now this is already obtained in the previous slide.

So, if we equate these two we get

$$nC_w\Delta u = \frac{1-2\mu}{E}\left[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3'\right]$$

$nC_w\Delta u$ is the volume change that has occurred we are now trying to equate these two. In essence, like it does not make much sense because this is not going to happen but then we are assuming that it is going to happen and that volume change is associated with the volume change that would have happened under drained condition. So, Δ u is that term what we need we need to estimate what is the excess pore pressure that gets generated.

So,

$$\varDelta\, u = \frac{1-2\,\mu}{nC_w\mathrm{E}}\,[\Delta\sigma_1' + \Delta\sigma_2' + \Delta\sigma_3']$$

Now triaxial condition is a very prominent condition in the lab. So, let us now specifically discuss about triaxial condition, what is the triaxial condition? We have $\varDelta\sigma_2{'} = \varDelta\sigma_3{'}$. So, if you put that we have

$$\varDelta\, u = \frac{1-2\,\mu}{nC_w\mathrm{E}}\,[\Delta\sigma_1' + 2\Delta\sigma_3']$$

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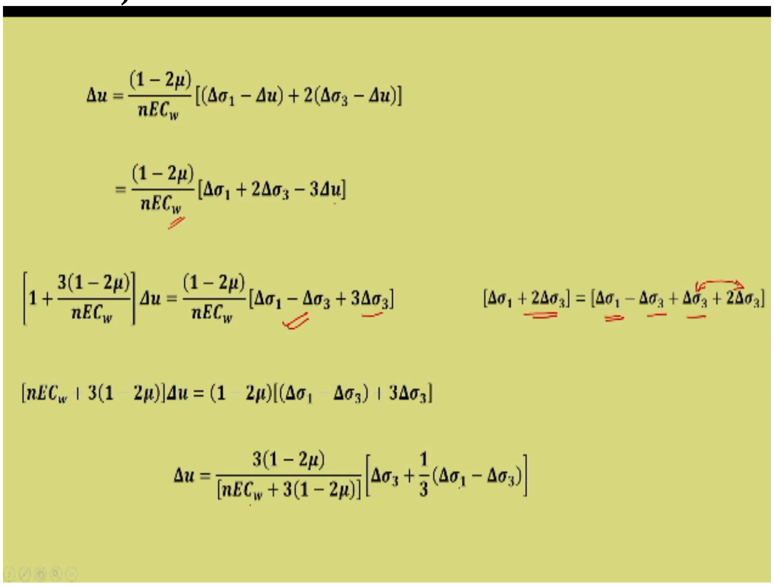

We can replace the effective stresses by total stresses

So,

$$\varDelta\, u = \frac{1-2\,\mu}{nC_w\mathrm{E}}\,[(\Delta\sigma_1 - \varDelta\, u) + 2(\Delta\sigma_3 - \varDelta\, u)]$$

$$\varDelta\, u = \frac{1-2\,\mu}{nC_w\mathrm{E}}\,[\Delta\sigma_1 + 2\Delta\sigma_3 - 3\Delta u]$$

Since,

$$[\Delta\sigma_1 + 2\Delta\sigma_3] = [\Delta\sigma_1 - \Delta\sigma_3 + \Delta\sigma_3 + 2\Delta\sigma_3]$$

We have,

$$\left[1 + \frac{3(1-2\,\mu)}{nC_w\mathrm{E}}\right]\varDelta\, u = \frac{1-2\,\mu}{nC_w\mathrm{E}}\,[\Delta\sigma_1 - \Delta\sigma_3 + 3\Delta\sigma_3]$$

$$[nEC_w + 3(1 - 2\mu)]\Delta u = (1 - 2\mu)[\Delta\sigma_1 - \Delta\sigma_3 + 3\Delta\sigma_3]$$

Simplifying, finally we get the expression,

$$\Delta u = \frac{3(1 - 2\mu)}{[nEC_w + 3(1 - 2\mu)]}\left[\Delta\sigma_3 + \frac{1}{3}(\Delta\sigma_1 - \Delta\sigma_3)\right]$$

$$\Delta u = \frac{1}{\left[1 + \frac{nEC_w}{3(1 - 2\mu)}\right]}\left[\Delta\sigma_3 + \frac{1}{3}(\Delta\sigma_1 - \Delta\sigma_3)\right]$$

$$\Delta u = \frac{1}{\left[1 + \frac{nC_w}{C_s}\right]}\left[\Delta\sigma_3 + \frac{1}{3}(\Delta\sigma_1 - \Delta\sigma_3)\right]$$

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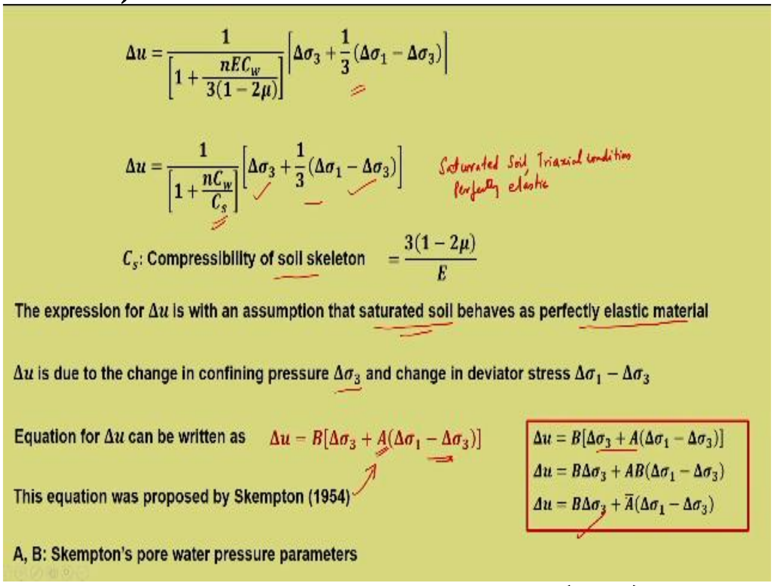

Now, C_s is known as compressibility of soil skeleton $C_s = \frac{3(1-2\mu)}{E}$. So, the expression for Δu is with an assumption that the saturated soil first of all it is for saturated soil it behaves as perfectly elastic material.

And this expression corresponds to that this expression whatever we have obtained it corresponds to first of all saturated soil this corresponds to saturated soil it corresponds to triaxial condition and it also goes by the assumption of perfectly elastic. So, these are some of the conditions which are pertaining to this particular expression, so Δu is due to change in confining pressure because $\Delta\sigma_3$ is related to confining pressure interaction testing and change in deviator stress $\Delta\sigma_1 - \Delta\sigma_3$.

So, there are 2 components one due to confinement the other due to the deviatoric component both results in Δu. So, the equation for Δu can be written as the

$$\Delta u = B\,[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

Now $\frac{1}{3}$ is replaced by A, why because $\frac{1}{3}$ corresponds to strictly a perfectly elastic condition. Now we will not expect soil to behave as elastic like any other materials.

So, for generality, so to deviate from elastic behaviour to give that option we have introduced Skempton introduced A parameter. So, this equation was proposed by Skempton in 1954. So, A and B are very popularly known as Skemptons pore water pressure parameters the use of A and B becomes prominent in the subsequent chapters, so, it is important to understand this.

So, now the same equation this equation is written here

$$\Delta u = B\,[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$\Delta u = B\,\Delta\sigma_3 + AB(\Delta\sigma_1 - \Delta\sigma_3)]$$

This is an alternate way of representing pore water pressure with Ā

$$u = B\,[\Delta\sigma_3 + \bar{A}(\Delta\sigma_1 - \Delta\sigma_3)]$$

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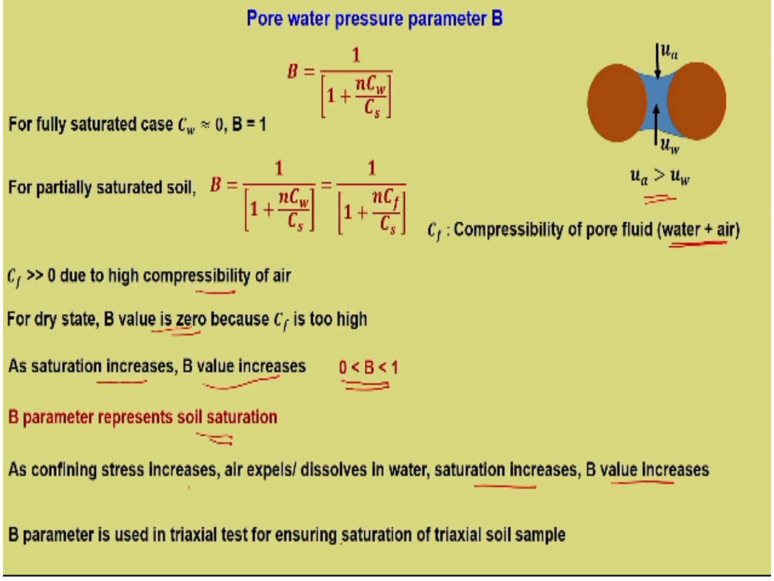

Now we will discuss a bit about both these parameters what are its physical significance, the first one is pore water pressure parameter $B = \frac{1}{\left[1+\frac{nC_w}{C_s}\right]}$. Now let us say the situation is for completely saturated case, if it is completely saturated now we have assumed water to be compressible but in fact water is not compressible. So, C_w for a completely saturated state with state when there is only water C_w tends to 0.

Even though we made an assumption that it is not, so that whole assumption was to correlate for obtaining Δu, so if there is no volume change which is going to happen. Then what is going to happen, so that was the kind of logic which was used. So, when for fully saturated case C_w approaches 0 and hence the value will be equal to 1. So, this clearly indicates for a fully saturated system B = 1.

Now what is happening to partially saturated soil, so the same expression instead of C_w it becomes C_f where C_f is called compressibility of pore fluid which includes both water and air now, it becomes more compressible it is not equal to 0 because the contribution of air is there which is highly compressible. So, $B = \frac{1}{\left[1+\frac{nC_w}{C_S}\right]} = \frac{1}{\left[1+\frac{nC_f}{C_S}\right]}$. Now what is happening this component is no more 0 it is an appreciable amount.

So, in partially saturated this is what it is u_a pressure is more u_w which is held in the pore pressure is negative, it is u is 0 and u_w is negative that we call it as negative pore pressure or soil suction and u_a is greater than u_w the compressibility of the whole system is now more than one and it is appreciable. So, C_f is very much greater than 0 due to high compressibility of air, what is the net result?

Because of this, for example it is a dry state completely dry state C_f is only due to the contribution of air this will be exceptionally high quantity. Now if this is very high, the whole of the B comes down drastically and it will be very much close to 0. So that is what is written here for dry states B value 0 because see if this denominator becomes very high. Now as saturation increases from the dry state as the saturation increases the B value also increases by the time it reaches fully saturated B = 1.

So, the range of value of B is in between 0 and 1, where 0 indicates dry state, 1 indicates saturated state B parameter hence can be used to represent soil saturation, so this parameter B is indication of saturation. Now as the confining stress increases air expels or it gets dissolved in water saturation increases and B value increases. So, this is a very effective way of using B parameter for ensuring the saturation of triaxial soil sample.

We all know and we have seen triaxial testing the first procedure is after mounting the soil is to get it saturated because conventional triaxial testing we discussed only about saturated soil. So, you need to saturate the soil first and we need to know whether it has actually got saturate now be parameter which comes from Skempton pore pressure parameter is used to check whether the soil is saturated or not, this will become clear in the subsequent slide.

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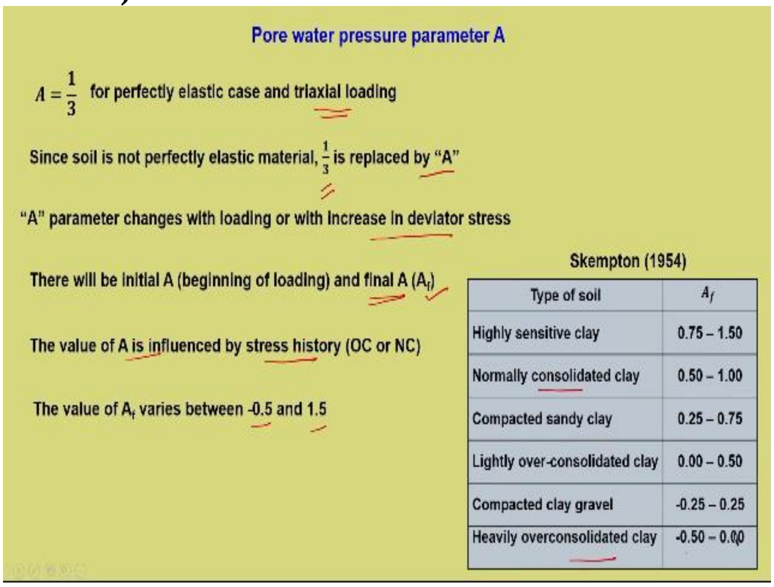

Now let us talk about pore pressure parameter A we have already seen $A = \frac{1}{3}$ for perfectly elastic case and triaxial loading. Note this is also important whatever we have discussed as Skempton equation is with respect to triaxial loading but we know soil is not perfectly elastic, so $\frac{1}{3}$ is not valid always, so that is why it is replaced by A. A parameter changes with loading or with increase in deviator stress that means.

There is a variation of A with loading which gets which takes place. So, there will be initial A so, before the starting and during the loading A changes and finally there will be at the failure condition there will be a value of A which is denoted as A_f. So, A_f is the A parameter at failure. Now, the value of A is also influenced by stress history depending upon whether it is OC or NC. Normally, A_f varies between - 0.5 and 1.5 that is what has been observed in the lab.

So, this is typical A f values which is given by Skempton in his paper in 1954 where we can see that for example, normally consolidated clay it varies from 0.5 to 1 and for a heavily over consolidated clay it is from - 0.5 to close to 0. This clearly indicates that the stress history of the soil influences the parameter A and with loading also the A will change because it is A is associated with deviator stress and deviator stress keep changing. So, the A value also changes.

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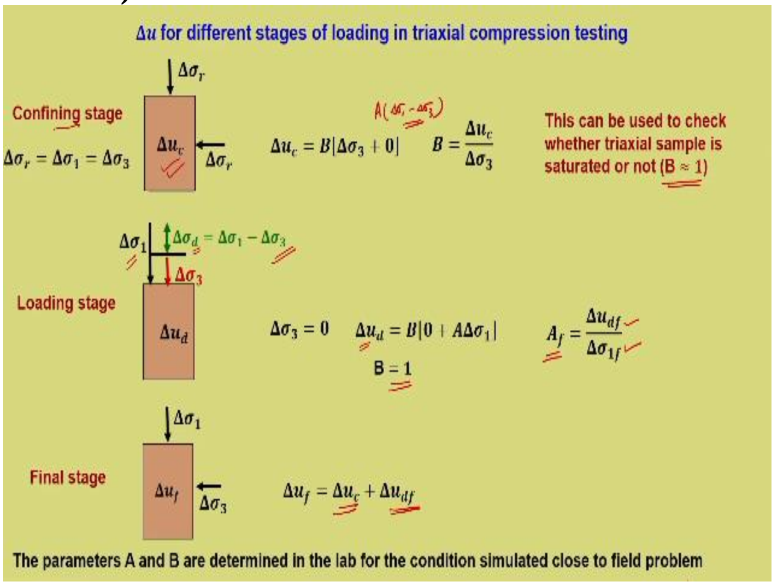

Now Δ u for different stages of loading in attraction compression testing. So, we have discussed about the actual condition here. So, let us see how Δ u develops for different stages of loading. So, the first stage is confining stage soon after saturation of the soil sample in triaxial testing we go for confinement application of confining stress. Now if it is an undrained condition, how pore water pressure develops that is what we will see you know.

So, this is a typical traction sample and $\Delta\sigma_r$ is the confining stress where $\Delta\sigma_r = \Delta\sigma_1 = \Delta\sigma_3$, it is an all-round stress condition only volume change happens. Now, if it is an undrained condition volume change will not happen but pore water pressure increases the excess pore pressure for confining stage is represented by $\Delta\, u_c$.

So, substituting this in the equation of Skempton equation we can write $\Delta\, u_c = B[\Delta\, \sigma_3 + 0]$ is very much there that is the order on pressure plus $A\,[\,\Delta\sigma_1\, -\, \Delta\sigma_3]$. Now this is equal, so it becomes 0. So, $B = \frac{\Delta u_c}{\Delta\sigma_3}$. Now this is where I told it can be used to check whether triaxial sample is saturated or not when will it be saturated when b value is equal to 1.

So, if you have done saturation, if you have done the triaxial testing in the lab you would have definitely looked for B value. Now, how B value the presence one now if for a fully saturated condition, what happens is whatever is the increase in $\Delta\sigma_3$ that will be fully reflected by Δu_c if there is some amount of air present in the sample whatever is the load that is applied that is $\Delta\sigma_3$ what is it total stress which is applied $\Delta\sigma_3$.

It will be partitioned between pore water pressure Δu_c and some volume change that happens because of the expulsion of air. So, it is not 1 is to 1 hence, Δu_c will be always less $\Delta\sigma_3$. So, B value will be less than 1 and that is how it is, so if you improve the saturation and the whole of the pore are filled with water, whatever $\Delta\sigma_3$ is applied that will be reflected as Δ u but it is very hard to get the value 1.

But it will be close to 1 anything about point 9.95 can still be considered a saturated sample, so after completing the confining stage then we go for the deviator stress that is the loading stage. Now, here please be careful this is the soil mass to which $\Delta\sigma_3$ was already applied. So, the order on stress is there now, you start increasing the axial load which is $\Delta\sigma_1$. Now here in the loading stage $\Delta\sigma_3$ remains constant or σ_3 whatever has been applied in the beginning is not changing in the loading stage.

So, $\Delta\sigma_3$ there is no change in the order on stress, so whatever actual load you are applying that itself will be $\Delta\sigma_d$ or $\Delta\sigma_1$ but the concept of $\Delta\sigma_d$ is what causes failure in triaxial soil sample. Now what stress causes failure whatever actual load that has been applied $\Delta\sigma_3$ was already existing before. So, whatever is causing failure is the total load minus whatever has been applied on to the sample minus this whatever was there initially that is $\Delta\sigma_3$.

So that is how it becomes $\Delta\sigma_1$ - $\Delta\sigma_3$ this σ_d is very much valid for the failure condition but here in this case we should know that whatever is the actual stress that we are applying that is $\Delta\sigma_1$ and that remains $\Delta\sigma_1 1$ because there is no increment in $\Delta\sigma_3$. $\Delta\sigma_3 = 0$ and hence,
$\Delta\, u_d = B[0\ +\ A\Delta\sigma_1]$.
So that here in this case it is saturated, so B = 1. So, you can write A_f that is the A parameter at failure $A_f = \frac{\Delta u_{df}}{\Delta\sigma_{1f}}$. So, this both corresponds to at failure condition what is the $\Delta\sigma_1$ at failure and what is the pore pressure at failure. So, A_f is defined. So, the final stage is the summation of the two which is $\Delta\, u_f = \Delta\, u_c + \Delta\, u_{df}$.

And total pore pressure at the end of the stage is equal to the one which comes from the confining stage and the one which comes from the loading stage. So, the parameters A and B are determined in the lab for the conditions simulated close to field problems. So, if you want to apply this for estimating the pore pressure corresponding to any given geotechnical problem, we need to understand what is the A and B parameter. B is not important A is important, what is the kind of a parameter corresponding to that soil for that particular field condition.

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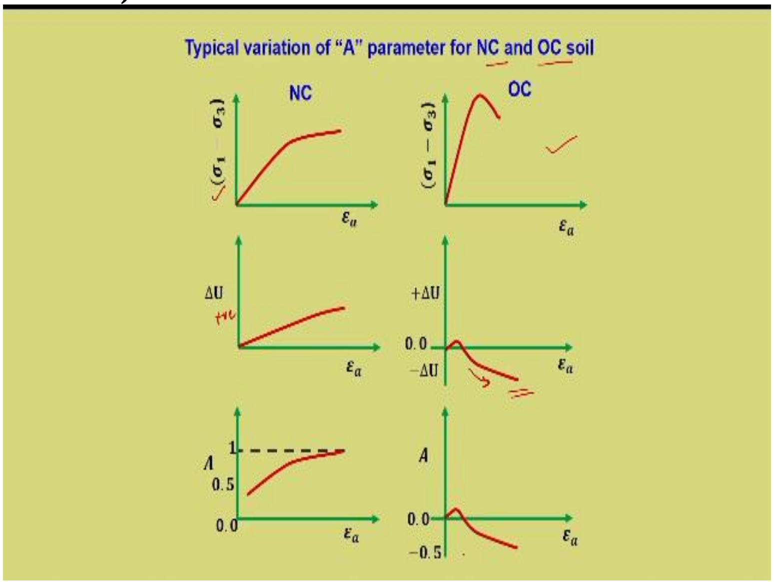

So, this is a typical variation of A parameter for with respect to stress history this we have already discussed what will be the kind of stress strain response this is a strain hardening and this is strain softening for NC and OC for corresponding to NC for an undrained condition, the pore water pressure will be positive. So, this is positive whereas for OC there will be initial compression for OC in the volume change.

That corresponds to a positive pore water pressure in the beginning and then it comes to negative for OC because OC tends to dilate. So, those which tends to dilate will try to suck the pore water inside, so that is negative pore water pressure, so that is given here. So, pore water pressure will be negative for OC or it keeps on reducing. So, correspondingly A parameter for NC you can see that it will reach close to one at the end of the test for NC and for OC it will become progressively negative. So, you can see this close to -0.5 for OC.

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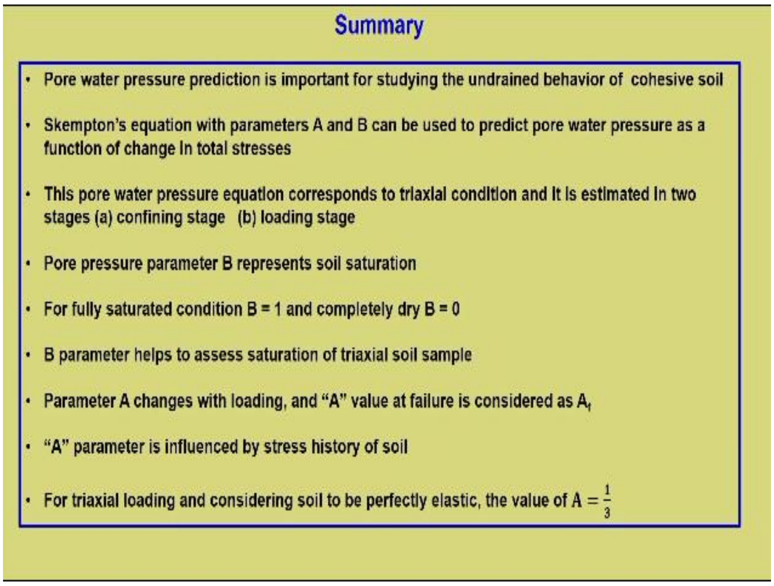

So, to summarize this chapter, it is pore water pressure prediction is important for studying the undrained behaviour of cohesive soil. Skempton equation with parameters A and B can be used to predict the pore water pressure as a function of change in total stresses. This pore water pressure equation corresponds to triaxial condition that we have already seen and it is estimated in 2 stages one corresponding to confining stage the other one is loading stage.

Pore pressure parameter B represents soil saturation for fully saturated B = 1 completely dry B = 0. So, B parameter helps to assess saturation of actual soil sample parameter A changes with loading and A value at failure is considered as A_f. A parameter is influenced by stress history of soil for triaxial loading and considering soil to be perfectly elastic the value of $A = \frac{1}{3}$. So that is all about pore water pressure estimation using Skempton equation.

In the next lecture we will see that how to use this same equation rearranged in a different format and how that becomes important for certain field problems that is all for now. Thank you.