Geotechnical Engineering - II

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Lecture No. 21

Pore water pressure Parameters

I have been talking about triaxial testing and in the previous lecture I introduced the concept of pore water pressure parameters, and a bit of the stress paths. So, these two things I will be covering in detail in another two lectures. Particular in today's lecture, I will be talking about the pore water pressure parameters, how to determine them by doing triaxial testing, and how to interpret the results, that is more important.

Now, this subject is of practical importance, and most of you must be dealing with it either in academics or in the consulting field. So, what I will try to do is I will try to balance both the sides and give you an optimum way, which you can follow for understanding these concepts in a better manner.

So, yesterday, we introduced this concept of A parameter and B parameter and AB parameter which we have defined as A' parameter. Now, normally we talk about two types of loading, which are the genesis of this type of pore water pressure parameters. The first one is what we call it as an isotropic loading and the second one is axial loading.

I hope you can understand isotropic loading is the case where the σ_3 is normally changed, typically the consolidation state of the material because σ_3 corresponds to the confinement. And if you remember the Skempton's equation this is how we defined it that,

$$\Delta u = \Delta u_1 + \Delta u_2$$

where Δu_1 is during the consolidation. And this is because of the change in the deviator stress. Many times, change in deviator stress can also be written as $\Delta(\sigma_1-\sigma_3)$, but in most of the circumstances, you will come across later on you will realize that mathematically $(\Delta\sigma_1-\Delta\sigma_3)$ might not be corresponding to $\Delta(\sigma_1-\sigma_3)$.

$$\sigma_1 - \sigma_3 = \sigma_d$$

Deviator stress term and sometimes people mistake this as like this depends upon the initial condition from which we are doing the testing. So, it is always better to follow the incremental change in deviator stress as $\Delta\sigma_1$-$\Delta\sigma_3$. So, this σ_3 is responsible for giving Δu_1, isotropic condition.

Isotropic condition means, if I consider an element of soil and if I load it from all around directions σ_1, σ_2, σ_3, suppose if it is general case, so σ_1, σ_2, σ_3 are going to be same equal to σ_3, which is going to cause increase in pore water pressure. So, to be precise, what we should be writing is change in $\Delta\sigma_3$ is causing Δu_1. The ratio of the two we have defined as B parameter. This is equal to Δu_1 upon $\Delta\sigma_3$.

$$B = \frac{\Delta u_1}{\Delta \sigma_3}$$

So, a bit change in confining stress gets resulted into change in the pore water pressure under isotropic loading condition, isotropic means all around, in all the three directions. Now, if you go into the analysis of this function what happens is B is normally defined as, I am skipping out the derivation of all these things because this is at UG level it is not required you have to understand the parameters which are involved in constitution of B parameter as such.

So, B is normally defined as 1 plus porosity multiplied by C_v upon C_s and these parameters are utilized mainly for characterizing the soil mass.

$$B = \frac{1}{1 + \frac{C_V}{C_S}}$$

Porosity is this term, C_v corresponds to the compressibility of the fluid under isotropic compression and C_s corresponds to the compressibility of the soil skeleton. So, the mineral portion of the soil which is giving a solid phase is the compressibility of the soil skeleton under isotropic compression.

I hope now you can realize that I can create a situation where all the pores are filled with water and in that case the compressibility of the pore fluid would be tending to 0 otherwise also because water is incompressible. Is this correct? So, if this is a situation what happens is B normally comes out to be unity or close to unity. So, yesterday we were debating upon this fact normally the range of saturation is from 0.95 to 1.

If this is the condition which is fulfilled, we say the soil sample is saturated, if not apply the back pressure and saturate the soil sample, so that you can use the concepts of Terzaghi's principles and so on. Is this part, okay?

Now, the A parameter is something different. Now, A parameter comes in the picture when, let us say I apply an incremental change in σ_1 itself uniaxial. So, it is understood that this is uniaxial and this condition occurs when you are shearing the sample because σ_3 is kept constant, remember changing σ_3 is not an easy task, once you have consolidated the sample at σ_3 you try to shear it by changing σ_d value or σ_1 value.

So, this is what is going to cause the shearing of the sample $\Delta\sigma_1$ which is equal to σ_d. Now, because of this whatever pore water pressure develops gets translated into Δu_2. So, that means, Δu_2 is due to the shearing this will be equal to $(\Delta\sigma_1-\Delta\sigma_3)$. This is the cause; this is the effect and the parameter which we get is A' which is equal to A into B.

$$A' = A.B$$

And this is defined as,

$$A' = \frac{\Delta u_2}{\Delta\sigma_1 - \Delta\sigma_3}$$

I repeat, $(\Delta\sigma_1-\Delta\sigma_3)$ should not be mistaken as $\Delta(\sigma_1-\sigma_3)$ in most other circumstances because now you can realize once you do not do that mistake, I am free to change σ_1 independently and I am free to change σ_3 also independently unless the initial conditions are fixed and that is it.

Now, it so happens that this AB parameter or A' parameter is truly speaking turns out to be B into 1 by 3 because what we are doing is this Δu_2 which is getting generated is to be distributed in all the three directions this assumption. So, one third we apply everywhere and hence most of the time A parameter can be taken as theoretically one third. But if you are doing an experiment, you should be measuring A parameter separately.

Now, if this part is clear, let us try to solve problem number 6. Δu_1 at the consolidation stage Δu_2 at the shearing stage that is it nothing greater than this.

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

$$u = B(\Delta\sigma_3 + A.(\Delta\sigma_1 - \Delta\sigma_3))$$

And put together we have written this equation as B $\Delta\sigma_3$ plus AB $\Delta\sigma_1$ minus $\Delta\sigma_3$ and this can be written as B $\Delta\sigma_3$ plus A $\Delta\sigma_1$ minus $\Delta\sigma_3$.

So, those of you who are clever can utilize this equation the way you would like to. In simplest possible form I can get rid of B if it is saturated soil sample, $S = 1$. And then I can get the value of A parameter as total pore water pressure minus $\Delta\sigma_3$ upon this is $\Delta\sigma_1$ minus $\Delta\sigma_3$.

$$A = \frac{u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$$

Now, choice is yours the way you want to play with A parameter. So, in a conventional triaxial test, what we do is we fix σ_3 during consolidation. So, by all means, $\Delta\sigma_3$ is going to be 0.

So, truly speaking A parameter is nothing but Δu upon $\Delta\sigma_1$ now how I achieve $\Delta\sigma_1$ we had a lot of discussion on this and I gave you a logic that the sample is confined in a rigid frame and you are lifting it up and the reaction is being recorded by the load cell or by the proving ring whatever it is. So, that gives you $\Delta\sigma$ value directly.

So, let us do the problem number 6. A compacted soil was tested, what comes to your mind? Compacted soil was tested, what compaction does to the materials? Now see, we are talking about the pore water pressure parameters, positive and negative. So, the moment we somebody gives us a hint that is a compacted soil mass, what compaction does to the pore water pressures?

Go to the compaction curve as your γ_d increases, what happens? Suction starts building up Geotechnical Engineering I concepts two concepts now have been clubbed together, capillary action in soils and compaction process. So, the connotation is the more and more complicated material you create the more and more suction you are creating into it.

So, this is the first connotation that compacted soil mean I am dealing with suction probably. So, was tested in a triaxial setup in an undrained condition at σ_3 equal to 400 kN/m^2. Consolidation stage. Now, there is a rider on this, the rider on this is before the application of σ_3 the pore water pressure was 0, initial condition of the pore water pressure is given and which is 0, the following results were obtained.

We have strain, we have deviator stress, we have pore water pressure, what type of pore water pressure this would be u_1 or u_2? That you have to differentiate you are straining the sample, so u_1 gets filtered out that part has been taken care of the shearing process is on. So, this is nothing but u_2 what we want is Δu_2 At 0 strain, 0 deviator stress, initial condition of pore water pressure is 0, the moment you are straining it, the pore water pressure shoots up to 250. It could be because of the disturbance caused to the sample when you are mounting it.

So, you have just taken a sample, you have balanced this to be the pore water pressure is 0 and then the moment you started shearing the sample the pore water pressure is 250. Now, let us write this complete series of tests. Hopefully, failure comes at this point, I have missed out 7.5 here. That is the problem. Pore water pressures are 250, 285, 150, 105, 75, 60, 50 kN/m^2. Normally the convention is everything is taken with respect to 0 strain.

So, this becomes my initial value and this is the final value. So, when I have to compute Δu_2 this will be final minus initial always. No confusion about this. From this point onwards you are shearing the material. So, this becomes your origin at this stage total consolidation has occurred. This becomes my background pore water pressure, which I am filtering out from further values to know what is Δu_2 value.

So, for that matter to 285 minus 250, 150 minus 250, 105 minus 250 and so on change in DS is simple. Starting from this point, this minus this, this minus this and so on. So, the way to understand this would be I think some of you are debating in the previous lectures, my logic remains same when you are shearing this material, what is happening?

Starting from 00, I am crossing these intermediate states of the material and then somewhere here the failure would take place. So, each of these states corresponds to the intermediate state in terms of shear stress versus ε_a relationship. So now, this is fairly simple, you get 0 this is 35 minus 100 minus 145 minus 175 and so on having done this I will like to compute $\Delta\sigma_1$-$\Delta\sigma_3$, why we are doing this?

Because I want to get the A parameter. From the moment Δu_2 is known this is your denominator term. So, this will be 570, 950 and so on. I can get from here A' parameter. A' parameter is equal to A into B so I should be obtaining B parameter first, can I obtain B parameter by any chance? This is a tricky thing, but you have to understand if your concepts are clear, you can obtain it easily.

So, read the problem statement carefully and see whether we can obtain B parameter or not. A compacted soil was tested in a triaxial setup in an undrained condition at σ_3 equal to 400 before the application of σ_3 the pore water pressure was 0 the incremental change in a pore water pressure happens to be 250. So, that means I can obtain B as, how do you characterize this soil?

This is the first characterization of the soil unsaturated sample, and that is correct because you are working on compacted soils so that is what our hunch was. So, the way you started analyzing the problem is that you are dealing with the unsaturated soil sample which has been having suction in it by virtue of being a compacted material.

So, for a quick review, I know all of you have forgotten but please remember these things. So, this is the OMC. Now, if I ask you to plot pore water pressure as a function of w, what is going to happen? So, the logic is simple, the more and more you compact, the negative pore water pressures develop. If I plot is as suction at these OMC where all the pores almost getting filled up, what will happen? This part we should remember.

So, rest is all simple you have the B parameter, you have the A' parameter, compute the A parameter from A' and then if OCR is given what we did in the problem number 5, I can relate this A to OCR value. That was problem number 5. So, this is one step ahead of the problem number 5, there we were knowing OCR, but we were not knowing how to compute these parameters.

Interesting thing would be if I plot the variation of A with respect to strain, what I will be getting is you remember, we studied this? If this is the response, NC, if this is the response OC, try to see in what category the material falls. So, this reiterates that most of the time the triaxial testing is done to characterize the soil mass. See, you should always remember, 0 strain is this situation, the moment you started shearing what somebody had asked the question sometime back, how pore water pressures develop?

I do not know who has asked this question, but that was a very good question and I think I gave you an answer that because of the change in the microstructure of the soil mass. So, 0 to 2.5 of shearing induces so much of pore water pressure 0 to 5.0 of shearing induces this much of pore water pressure, this is how the material is this is a history of formation, whatever, you cannot control it.

So, we make this as a benchmark, so everything is happening with respect to this change in DS is with respect to this change in pore water pressures also with respect to this and gain in axial strain is also with respect to this state. You will realize that this was inbuilt over here. So just a minute I got your point. So rather than giving this value over here, what I would have done is have good given this value over here itself. So, the moment you get the pore water pressure because of this process, and then I would have sheared it does not matter. But then the concept of back pressures I am trying to explain to you.

You will find this relationship slightly inverted. So, if you plot this relationship what they do is they plot pore water pressure like positive and negative and this happens to be my OMC. So, I hope now you can realize this is how the pore variation would be. So, when you come close to the saturation line, or the maximum compaction state your pore water pressure tends to become positive or 0.

Those of you who might get a chance to work in unsaturated soil mechanics will link this u verses w relationship with the k unsaturated with respect to moisture content. So, those of you who might get a chance to work on THMC and other things thermo-hydro-mechanical coupling of soils because of high temperatures, soil sample is becoming unsaturated suctions are becoming negative suctions are getting developed in the system, the pore water pressures are becoming negative because of loss of moisture.

And the material might be transforming from a NC to OC state, desiccation. How would you characterize the soil based on the parameters?

So, the thumb rules are like this,

B = 1	Saturated soils
A = 0.5- 1.0	For N.C. soils
A > 1	Highly sensitive clays
A = 0- 0.5	O.C. soils (lower compressible soils)
A < 0	Heavily O.C. soils

So, do not try to mug it up this just for your understanding, nothing more than that. So, I repeat most of the time triaxial testing is done to understand the condition of the soil. You want to diagnose this.