Geotechnical Engineering - II

Professor D. N. Singh
Department of Civil Engineering
Indian Institute of Technology, Bombay

Lecture No. 22

Pore water pressure Parameters II

I hope you understand when we are talking about Pore Pressure Parameters, we are talking about $\overline{CU}$ test and $\overline{UU}$ test. Prime is the word which prime is how we differentiate them. Consolidated undrained test with pore water pressure measurement and consolidated undrained test with pore pressure measurement.

So, by means of a $\overline{CU}$ test the shear strength parameters of the soil were found to be c' equal to 10 kN/m^2, ϕ' equal to 24°, so this problem is an abridged version of the previous problem. In the previous problem we were shearing the sample and we reached up to failure and there we were computing A value here all that has disappeared, the final state of the failure is known.

So, c', ϕ' is known your 50% job is done you need not to understand the material under effective stress parameters. Now, added to this is A_f and what you are observing is a negative sign. So, what it corresponds to? What type of state of the soil is this? Very good, it is heavily over consolidated material, heavily over consulting material will always show you suction.

So, the pore pressure which we are going to compute would be negative, that is it. So, this is a type of characterization of the soil mass. Now, in a UCS test, what UCS gives? Remember, go back to your lecture notes and check what UCS gives you. Unconfined compressive strength, what type of parameter it gives you.

So, in a UCS test a sample of this soil failed at a compressive stress of 162 kN/m^2. Determine the initial value of suction in the soil. So, this problem is a very straightforward direct problem, no complications at all everything is well known. Most of the time these types of tests are done, sorry this type of analysis is done to check whether your pore water pressure measurement devices are working alright or not.

You cannot believe you have to calibrate them. I hope now you realize that from this what I am going to get is pore pressure and pore pressure I am measuring also $\overline{CU}$. So, consolidated undrained tests with pore pressure measurement. I want to check both the things what could be another reason, why I am so much eager to know about the pore pressures? So, this answers your question what you have been asking?

Very soft sensitive soils the way you have made the sample the way you have touched them and the way you have installed them on the pedestal might have changed the state of the materials also. In short, there are ways to check our experimentation and the results which we obtained from there.

So, rest of the problem is very simple, we have done this problem number 2 I suppose, where what we did is we obtained first c', ϕ'. State of stress was given there and then we obtained pore pressure if you remember. Now, what we are doing is we are going to use the concept of A_f over here. So, A_f is equal to Δu_2 upon $\Delta\sigma_1$ minus $\Delta\sigma_3$.

$$A_f = \left(\frac{\Delta u_2}{\Delta\sigma_1 - \Delta\sigma_3}\right)$$

So, what is given to you is a compressive stress of σ_1 minus σ_3 will be equal to 162 and because this is a UCS, so σ_3 becomes 0. So, σ_1 becomes 162. How are you going to use the value of A_f? So, A_f multiplied by 162 should be giving you Δu value. So, what is the value of Δu? Δu will be equal to initial pore pressure minus the final pore pressure and the pore water pressure which is coming from here, let us say u_2 rest is simple.

So, you have the Mohr-Coulomb envelope given to you, A_f value was known compute the suction values and this comes out to be 66.64, be careful with the sign of the pore pressure it is a negative value in kN/m^2 because if you compute the positive pore pressure this will be redundant solution, what is the genesis of this pore pressure development? Try to go by that.

So, you are achieving the failure by shearing the sample and then determining the initial value. This is a reverse problem of what we did just now, there we fixed that at 00 value something was given I do not know that value. So, I want to compute that so what it says is determine the initial value of the suction in the soil sample before you started testing it.

Problem 8: In a triaxial test on a partially saturated soil, if you remember when we were talking about the capillarity in the soil and when you were computing γ_d values, I had created a situation like this that you might be having a multi-layered system of soils and somewhere here is the water table depending upon the type of the filling which you are going to use here.

And the compaction efforts you are going to create a zone where saturation would be a function of z, everything below the water table is saturation equal to 1. We call this as a variably saturated soil deposit. Then I introduced the concept of capillary fringe here, if you remember. So, because of this water table over here and again depending upon the soil type and its compaction, you might be having a zone in which the capillarity dominates.

And we could compute this by using the h_c value if you remember. So, this thing we classified as h_c equal to some point 3 upon eD_{10}.

$$h_c = \frac{0.3}{eD_{10}}$$

If you remember because this is an mm and this was in metres. So, we have computed the capillary fringe also, so what happens beyond capillary fringe is dry material unsaturated or variably saturated.

So, this becomes a very interesting problem. So, triaxial test on a partially saturated soil it appears that I have taken a sample which is slightly above the capillary fringe. So, it is not saturated, it is not submerged, it is a different state of the material, which is defined like this. I was very eager to know now how the shear strength is changing, shear strength parameters are changing as a function of z because, the question was that where should I lay the foundation?

So, I have given you a profile which I can obtain by doing investigations, I will do sampling, I will do different types of non-invasive test, geophysical investigations, I will establish the subsurface profile. Now, the question is where should I lay the foundations? So, suppose if I decide that I will go for a shallow foundation or a strip footing.

The question is what type of shear strength parameters are going to prevail there. This is the type of problem which we are doing, this is the background. So, there is a context for which you are doing triaxial testing. Trying to obtain the parameters, so that you can complete the design. So, with all this story in mind at the back of your mind now, move ahead.

So, in a triaxial test on a partially saturated soil which is clay from particle size analysis from your Atterberg limits and all the characterization whatever you have studied until now, soil characterization. The sample was consolidated at a σ_3 of 150 kN/m^2 after which the cell pressure was raised to 300 kN/m^2 and the failure was achieved.

I hope it is understood that you have sheared the sample, so DS is changing deviator stress and then you are failing the sample, determine $\sigma_3{}'$ and $\sigma_1{}'$ at failure. First question is trying to understand whether this problem can be solved or not without giving additional information? So, quick answer is it cannot be because you need further information because failure condition is not defined yet properly.

When we started this course, we were talking about failure if you remember we said σ-τ at failure and this was related to σ_x, σ_z, σ_y and τ series and angle of inclination if you remember at a point where the material did not come into the picture anywhere, that was a limitation of the Mohr circle.

Now, once we go into the material, material can be depicted in several ways, two easy ways would be either you talk about A parameter, because A includes everything and we will see in the rest of the lecture today because A is a parameter which talks about the pore water pressures because of shearing related to OCR, related to hydraulic conductivity, related to particle size distribution and even plasticity index.

So, in short, if I want to solve this problem, I have to give the material properties. So, if I give you c' is ϕ' is same as the previous problem only thing is saturated sample unsaturated sample less than 0.95 and $A_f{}'$ is given as 0.25 rest of the things are same. Another thing is train your mind in such a manner that answers to all these problems should be not more than three, four steps that part will come to you with the more realization.

So, this highly has much too much to be done, you will be getting $\sigma_3{}'$ as 300 kN/m^2 and $\sigma_1{}'$ will be equal to 120. Now, where is my friend who was talking about $\sigma_3 > \sigma_1$ and $\sigma_1 > \sigma_3$, are you realizing something? So, in effective form what has happened?

So, your Mohr circle has shifted on the second quadrant A_f value is positive read the problem, in a triaxial testing on a partially saturated soil, the sample was consolidated at this from this state σ_3 was increased to 300. So, your $\Delta\sigma_3$ becomes 150. Now, listen $\Delta\sigma_3$ becomes 150 you know the B parameter you can go ahead with all those things, $A_f{}'$ is known so you know $\Delta\sigma$ value $\Delta\sigma_3$ try to fit in that Mohr circle within this envelope, that is it. Do both the ways numerically as well as graphically.

Most of the infrastructure projects which are going on everywhere, one common mistake which people do is that they do not talk about the gain in shear strength of the soil concepts. Now, suppose if this is a formation of the soil and this soil is poor, soil is poor means either void ratios are very high, moisture content is very high, this is compressible m_v values are very high, C_C values are extremely high very close to unity let us say or more than unity, C_Cs are very high, void ratios are extremely high and plasticity index is extremely high.

Now, this is the delight of a geotechnical engineer because this becomes a difficult situation to train, we call them as problematic soils general public call it as a problematic soil but those who are technologists for them, this problematic word gets changed by very good excellent that is the spirit and not only the soils, but this is also the conditions. So, what are the domains in which we are active these days? Offshore engineering, challenging conditions improving the soils in the marshy lands, waterlogged areas, organic clays and so on.

This is something new which has to be learned by all of you, our generation could do geomechanics with good formations, unfortunately now, good formations are not available, so you have to go and encroach upon the challenging grounds. Now, what happens is suppose if I consider this point O, what is the state of stress at this point?

Say $\sigma_v{}'$ and $k\sigma_v{}'$, you have studied effective stress principle so, now I am using effective stress terms. Now, starting from this state, if I load this system under undrained conditions, what is going to happen? This system will get converted to let us say $\sigma_v{}'$ plus let us say $\Delta\sigma$, and what will happen over here?

This will also get changed to $k\sigma$ plus $\Delta\sigma$ or a function of $\Delta\sigma$, I would try to tear as a multiplier of let us say, $A.\Delta\sigma$, the more and more I compress, what is happening? The system is getting improved, number 1, that was the whole concept of one-dimensional consolidation and by putting the you know, pre loading on the top of this deposit, so, that the simple sample gets compressed e values change, C_c decreases, m_v decreases and hence shear strength changes.

So, all these things are getting reflected into C_u. So, truly speaking, what is happening is that C_u and $\Delta\sigma_v'$ they play a balancing act. So, the more and more $\Delta\sigma_v'$ you apply, what is going to happen? C_u is going to enhance, why? Because this is the function of the material and this hypothesis was given by Skempton in 1957 that it is equal to 0.11 plus 0.0037 into PI, this is what is known as Skempton's equation (1957).

$$\frac{C_u}{\Delta\sigma_v'} = 0.11 + 0.0037.PI$$

Ultimately, this whole thing has to be analyzed. So, if you follow the first principles, what you will be getting is the undrained cohesion or undrained shear strength of the material would be, do not remember this equation, my whole purpose of writing it over here is to demonstrate to you. So, this is a form of the equation which we will be getting, importance of the parameters which we have talked about until now.

Now, K_0 is the earth pressure coefficient at rest. A_f is what we have discussed until now, A parameter at failure so what you are realizing is that there is a relationship between the undrained shear strength with the effective stresses which are getting caused onto the sample. Normally, design charts are available between $C_u/\Delta\sigma'$ and they are plotted with respect to OCR and this is how you have these relationships only thing you have to remember is for c=0 is the condition for NC material.

So, truly speaking this function gets transformed to $C_u/\Delta\sigma'$ equal to this whole term. So, if this is 0, $C_u/\Delta\sigma'$ would be this parameter. One quick hint is if you do not include the change in the C_u value, there could be a situation where you might come across the consolidation which are more than 100 percent and that is not valid.

There were beautiful bypasses which were designed in the country and the designer forgot this concept. And to your surprise you will realize that the entire report the degree of consolidation achieved for more than 200, 300, 400%. This is known as is undrained shear strength and effective overburden relationship.