Geotechnical Engineering - II

Professor D. N. Singh

Department of Civil Engineering

Indian Institute of Technology, Bombay

Lecture No. 20

Interpretation of Triaxial Test Results III

In a triaxial test, a sample was consolidated at σ_3 equal to 700 kN/m^2. Now, one of you should have asked this question that why we are so particular about selecting a σ_3 at 700, 200, 300. So, let me repeat suppose, you are taking out a sample from this point which is at a depth of z; and what we are trying to simulate is by doing a triaxial test, the state of stress which the sample is exhibiting at this location.

So, if I know the γ_d value of the soil sediments, I know the depth at which I am working. So, γ_d multiply by z is going to give me some idea about this σ_3 value; and this σ_3 I am using for confining the whole sample. So, the best possible simulation of the field conditions can be done like this.

There are a lot of limitations; everybody knows by the time you bring the samples and all. Now, let me introduce here the concept of back pressure. So, in a triaxial test, a sample was consolidated at σ_3 equal to this, consolidation is over. Followed by when I was discussing the schematic diagram of the triaxial testing, I had shown you a port which if I open, I can drain out the water during drained conditions.

If I close it, it becomes undrained condition. I need another port to pump in saline into the body of the sample, the way doctors do. The question is why we are applying back pressure to the sample? So, that means, if this is a sample. Now, rather than taking out something, I am applying a back pressure. Now, this could be done either by passing a gas or a fluid, depending upon what is that you are working on.

Mostly we apply water, and we pressurize the sample from inside. You are doing testing at σ_3 to begin with. If the sample is very fragile and if I shear it from this point onwards, what is going to happen? This σ_d might result in instability of the sample itself or sometimes sample may get crushed.

So, what we do is we apply a manipulated pressure from the inside of the sample; and what it is doing? It is getting added up to the pore water pressures which are developing in the sample. So, this goes and becomes a part of the pore water pressure in the sample. So, I am basically taking care of the health of the sample, nothing more than that. Another possibility is it might so happen that my samples are not saturated.

And being a third-year student whatever you are studying is the class of problem, which is known as saturated soil mechanics. Unfortunately, all these triaxial testing is not valid for the unsaturated state of the material. So, if you wanted to do the direct shear strength parameter analysis for the soils which are unsaturated, then you have to come to a laboratory like ours, where we have facilities of doing unsaturated soil testing.

Otherwise, I am conventional test setups, you can't do unsaturated soil samples. So, this is where you can do a manipulation and the manipulation is I will saturate the sample by applying back pressure; so that it falls into the category of saturated soils. I am camouflaging the situation in such a manner that unsaturated sample has become saturated, and I will keep on doing the test, that is it. So, this is the purpose of applying back pressure to avoid collapse of the sample.

So, truly speaking, now what I have? I have σ_3 minus back pressure which is positive number one, even in your gas hydrates also this valid. So, we just make some finite difference; let us say 10 kPa or so to make the samples stand over there and rest of the things are same. So, let me now continue with this system. So, I have introduced this concept of back pressure into the sample and what it does.

Primarily, apart from the physical integrity of the sample, it goes into the pore water pressures; let us start this story again. Now, having done all these things thereafter with drainage not allowed. That means it is an undrained test, typical undrained test. The cell pressure was raised to 800 kN/m^2. Sometime back I had introduced this concept of stress paths if you remember; I had drawn some arrows which were touching the Mohr-Coulomb envelope.

So, triaxial testing is done to torture the sample the way you want and to reveal its response the way you want. Now, where all these things are going to be number one valid. You are seeing in a Bombay city lot of basements have been done everywhere. So, this is the ground surface, this is the soil mass. And my client says that I want to construct the underground facility over here. What has happened to the state of stress at this point from initial to final?

We call it as a remover; so, that means at this point, your σ_z. And if I write incremental form, it becomes $\Delta\sigma_z$; and this is becoming less than zero excavations. $\Delta\sigma_z$ becomes, let it be σ_v; this z is also fine as long as I consider this as z direction, no problems. Or make it v because we have been dealing with v. So, $\Delta\sigma_v$ is less than 0, so it is decreasing.

I hope you can realize having done all these things, rather than increasing the σ_3, I would have brought it down to value less than 700 also, if I really wanted to see the response of sample. I would have tested the sample at 500 kN/m^2 σ_3, you got it? So truly speaking, what we are doing is we are simulating exactly what is happening in the nature in our laboratories on the specimens.

Another possibility would be you are doing a big embankment over here apart from this excavation; do not mix it together, otherwise it becomes quite complicated. So, what has happened to the state of stress at this point? This point, I hope you understand, $\Delta\sigma_v$ has become greater than 0. So, I want to understand now what is going to happen to the stability of this foundation system? Whether this facility can be built over there or not?

And of course, the third situation would be you have very good neighbours who always like you; so, you are constructing something over here. And the next plot, there is a deep excavation going on; beautiful case of litigation, because you know what is going to happen. Now, this is a beautiful geotechnical engineering problem 1, 2, 3 all put together.

How would I simulate it here and see how the material is responding. Now, how I do all these tests, and all is a different issue altogether, concepts are simple. So, starting from this undrained situation, we have raised σ_3 to 800 kPa or kN/m^2. And there is a pore water pressure equal to 445 kN/m^2 as a result of this torture. Now, we define a term which is known as B parameter.

We have been talking about the pore water pressures all the time. And in my last lecture, I think I gave you an idea that pore water pressure constitutes of two parts. The first part is during consolidation and second part is during shearing; normally we call them as u_1 and u_2. B parameter is normally used to understand what the state of saturation of the sample is; so this is equal to $\left(\frac{\Delta u}{\Delta \sigma_3}\right)$.

If you read the problem carefully what has happened is the final pore water pressure is 445 minus the back pressure you have applied is 350 divided by 800 minus 700; this is 95 correct. Now, this is the catch. First part of the triaxial testing is done to understand what the state of the patient is and what I should be doing with it, though we have already applied the back pressures. By applying back pressure, I have got a B of 0.95.

For your information if B drops less than 0.8, it becomes unsaturated state of the soil. So, who knows? The guy must be aware of this. And in order to bring the sample to the saturated state all this has been done. Now, the second part of the problem is relatively simple. Having done all these things, and this corresponds to from 0.95 to 1.0 range is correspond to saturated sample; so state of saturation the sample was achieved by back pressure.

The axial load was then increased. And remember the axial load can be increased just by shearing the sample, by lifting the whole thing from the bottom at a constant rate of straining, to give a deviator stress of 575 kN/m^2; while the σ_3 remains constant, even σ_1, σ_3 will also vanish.

And we will be using only vectors which would be termed as state of stress or stress paths to deal with the problem; so, all this goes in the background. And what I will do is keeping σ_3 constant means $\Delta\sigma_3$ is 0 from this state onwards. And hence, I am going to shear the sample keeping σ_3 constant. At this stage the pore water pressure reading comes out to be 640 kN/m^2.

Remember we defined the term parameter A; and A was the response of the material to $\left(\frac{u_d}{\sigma_3}\right)$. And the way we write this is, this is the efficiency factor, how much σ_3 you are changing and what type of pore water pressure develops in the system doing undrained here; so that is what is being done. The axial load was increased to give a σ_d of this much, and the pore water pressure is this much computing the parameter A.

So, if you do quick computations, you will find the parameter A comes out to be 0.339. Now, this parameter A, I will be defining as A'; and A' will be equal to A into B.

$$A' = A.B$$

So, from here I will be getting A as $\left(\frac{A'}{B}\right)$ and this comes out to be 0.357. These are the nomenclatures you have to just remember. A is defined as Δu divided by $\Delta\sigma_1$ minus $\Delta\sigma_3$.

$$A = \left(\frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}\right)$$

And later on, we will derive an equation which is known as Skempton's pore water pressure equation, where this total pore water pressure can be written as,

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

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

This can be simplified as,

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

This is what is known as Skempton's equation. For saturated samples B is 1. So, if B is equal to 1 for saturated samples, this is how you characterize the soils. A parameter will become,

$$A = \left(\frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}\right)$$

Understand the philosophy. $\Delta\sigma_3$ is appearing both in the denominator and the numerator correct. So, truly speaking $\Delta\sigma_3$ nothing is sort of a cushion of the σ_3 which you have applied. And A parameter is nothing but because of change in $\Delta\sigma_1$.

And σ_1 is coming from the σ_d, because $\sigma_d = \sigma_1$-σ_3; incremental form would be $\Delta\sigma_1$-$\Delta\sigma_3$. The moment I say σ_3 remains constant; $\Delta\sigma_3$ gets knocked out. So, this becomes $\Delta\sigma_1$, and A parameter is nothing but the pore water pressure which is getting developed because of shearing process. So, we use two parameters A and B and A' to characterize the soil.

On the same material a series of, so, by doing all this analysis what we have established is the sample is saturated only by applying back pressure. We have made sure on this soil the results obtained are. I hope you understand the meaning of these notations. Check this, be careful and see what I am writing over here, and what is the significance; yesterday also we discussed this thing. What type of soil response is this?

These types of dubious materials are OC materials. And that is the reason we do not depend too much on OC materials and the OC response. There is a switch over, you are right. So, if you remember we plotted A versus percentage strain. NC and OC. So, the OC character is dilly-dallied; it switches over and shows instability. Now, the rest of the problem is simple or whatever we have discussed until now.

So, you determine the shear strength parameters and parameters in terms of effective stresses; and plot the variation of A as a function of OCR, if the pre-consolidation pressure happens to be 700 kN/m^2. So, for your quick reference OCR is (σ_c/σ_3), the pre-consolidation pressure. A very close look on the system is going to tell you that if I would have decreased the value of σ_3, the response would have been something like this.

So, this is σ_c', this is OC response, this is NC response; NC response gives you only ϕ and this gives you only c'. So, everything is dependent upon σ_3. And if I extrapolate this thing to the one-dimensional consolidation; this goes further down, this becomes your NC behaviour.

If I consider two points, one A and another one B at a given σ' value; so, this corresponds to point number A and this corresponds to point number B. What it indicates is that the shear strength is a function of inverse of void ratios. So, from all sides we are safe, we have cross-checked. Like this ϕ' is 25°, c' is 30 kN/m^2.

The first time we are plotting A_f versus OCR; until now we have been plotting A as a function of percentage strain. So, what you will be getting is; this graph is useful for the designers. So, we have discussed five types of triaxial testing problems. I hope I have given you enough ideas about how to interpret the results. Keeping in mind the material which you are using; and the stress range in which you are working, and the objective of doing a project.