Earthquake Geotechnical Engineering

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Lecture 33

Liquefaction Susceptibility (Conti.)

I welcome you again for this NPTEL online course on earthquake geotechnical engineering and today we are entering in lecture number 33 which is part of module 4 which is on liquefaction of soils. So, the module 4 we have already covered two lectures, one lecture was related to introduction, the second lecture was on liquefaction susceptibility. Today 33 lectures we continue with the liquefaction susceptibility. So, we are going to discuss in detail the liquefaction susceptibility which is like in continuation of lecture number 32 where we have discussed historical, geological and composition criteria for the liquefaction susceptibility. Today we are going to talk about the last criteria that is state criteria which is based on the conditions, or the state related to density and stress conditions these state criteria are defined.

What we are going to discuss is mostly like from the Kramer's book, for the today's lectures many of the figures and other things will be directly coming from this reference. So, as far as state criteria is concerned you know that even if a soil meets all the criteria's like historic, geologic and composition criteria which is for liquefaction susceptibility still it is not necessary that it will be susceptible to for liquefaction. It may be susceptible or may not be. So, they may give you an indication preliminary.

So, those criteria are okay for preliminary analysis historic, geologic and composition criteria but the state criteria will confirm to you whether yes soil is susceptible and not susceptible. Because the susceptibility will not only will depends on those criteria historic, geologic and composition, but it will also depends on the initial state of the soil and when we talk about initial stress that means stress and density characteristics at the time of the earthquake. So, whether your soil is loose or dense so it will make a difference. Similarly, stress conditions will also govern whether this the liquefaction will take the susceptibility is there or not. So, coming to this when we say because as we discussed and basically in the liquefaction of soils there is a generation of excess pore water pressure of due to the shaking and this liquefaction is strong and this generation of excess pore pressure is very much influenced by the density and initial stress conditions.

Therefore, liquefaction susceptibility depends strongly on the initial state of the soil. So, it is directly coming because generation of pore pressure depends on these factors that is why the liquefaction susceptibility also depends on the state of the soil which is related to two factors stress condition and density. So, and unlikely other criteria which were discussed and these criteria are divided into two categories. So, like that is different these criteria are different for flow liquefaction and cyclic mobility, and we will discuss in detail that. Long back Casagrande in 1936 performed many triaxial tests and these are the strain control triaxial test which was drained that means drainage is allowed.

So, pore water pressure did not develop on initially loose and initially dense sand specimens. So, two types of specimen was taken one specimen was initially loose another was dense and here when you say strain control that means the maximum value of a strain is already fixed that value is not crossed. So, the results are shown here in this slide what you have a you have deviatoric stress versus axial strain and in triaxial I think you already know what is the deviatoric stress. We already discussed when we talk about dynamic soil properties you may be aware that normally the triaxial sample is a cyclic the cylindrical sample, sigma c which is the cell pressure is applied all around and in addition to this deviatoric stress is applied like deviatoric stress is applied to on axial direction. So, basically the deviatoric stress is the difference between sigma 1 minus sigma 3 where sigma 1 is the major principal stress and sigma 3 is the minor principal stress.

So, on y axis you have deviatoric stress sigma d and on axis when you strain the sample in strain control test you are straining the sample at different strains. So, when you increase the strain for the dense and loose sample how they behave dense sand the deviatoric stress goes to maximum peak value in dense sand but may not go so peak in the loose sand. But ultimately at the higher value of a strain whether you have dense sample or loose sample they will reach to the almost same deviatoric stress at a higher value. So, that is the point here. Similarly, when you have this when we talk about because ultimately what happens when you put understanding the voids ratio whether it will increase or decrease.

In case of dense sand or the void ratio will increase, in case of loose sand it will decrease because it is closer packing. So, which can be seen in figure B. So, for the figure B which is with respect to void ratio for the loose sample the initial void ratio was EL and when it undergoes straining it will start decreasing and reach to a point what is called EC critical void ratio. Similarly, in case of dense sand initially the void ratio little bit decreased, but after that it is start increasing and ultimately the void ratio in both the samples even the starting point was different that is one was loose another was dense, but still the void ratio reach to the one point what we call the EC which is called critical void ratio. So, these are given on this slides also.

When we have one term have come that critical void ratio is a void ratio where at higher value strain axial strain or let us say shear strain corresponding shear strain all the soil samples will reach to one point like that is the void ratio which you can obtained at high value of straining. So, the EC irrespective of that whether you have the initial condition as a loose or dense. So, this is a critical void ratio which is sometimes in the short also called this term is also called CVR and many times we will discuss later what is CVR line. CVR stand for critical void ratio. At large strains all specimen approach the same density and continue to shear with constant shearing resistance as we discussed in the last slide. The void ratio corresponding to this constant density was termed as the critical void ratio that is EC and by performing test at different effective confining pressures Casagrande found that the critical void ratio was uniquely related to the effective confining pressure and called the focus the critical void ratio line that is CVR line. What is CVR line? C, there is a one critical void ratio EC, but this critical void ratio is not constant rather it depends on the confining pressure. Here what I could say that this test has been conducted at particular confining pressure in the triaxial sigma c is constant.

Suppose the same test for the same soils I conduct the test on different confining pressure then the value of EC which I will obtain will be different. So, the EC for the dense and loose sand may be the same however, it will depends on the confining pressure and the locus of the points if I plot a curve EC versus confining pressure then that line is called CVR line and the CVR line is given here. So, in this case nothing but you have confining pressure on x axis and corresponding deviatoric stress. This deviatoric stress is corresponding to the critical void ratio that means, this is this point. So, here the deviatoric stress plotted is this value at the end which is the same for whether you have the loose sample or dense sample.

With this you plot and this is called the CVR line. CVR line is the locus of the points with the variation in deviatoric stress with respect to confining pressure where deviatoric stress corresponding to the critical void ratio. And this CVR line is used in many cases a boundary between loose contractive state and dense dilative states. So, if you are below lying below the CVR line then your sample will be dense sample and then sample will expand when you apply the straining then it will dilate while on the top you have the loose sample where it will start like when you give the shaking to it then it will contract. So, ultimately both dense and the loose sample will try to reach near to the CVR line.

The dense sample will come upward, and loose sample will come downward so this way. So, continue with the CVR we use this one. So, we define the state of the soil in terms of void ratio and effective confining pressure. The CVR line could be used to mark the boundary between we already discussed this. The equipment needed to measure the pore water pressure was not available at that time.

So, as a result Casagrande hypothesized that stress controlled undrained testing would produce positive excess pore water pressure that is tendency for contraction. So, here is the point here. The test which we have discussed was drain test because if you recall that this was drain test initially. So, and in the drain, test was there conducted because at that time there was no equipment in earlier to measure the pore water pressure. However, there was a hypothesis that is in during a strain control test if it is conducted in undrained condition then there will be development of positive excess pore water pressure and this positive excess pore water pressure will develop for the loose sample which have tendency for contraction in loose specimen while negative pore water pressure will develop which like for the dense specimen and dense specimen have the tendency to dilate.

So, one sample loose sample will contract dense sample will dilate. So, the positive pore pressure develops in case of loose sample if you do not allow the drainage, if drainage is closed undrained. So, in undrained condition there will be development of pore pressure or pore water pressure, but this development of EWP stand here excess pore water pressure. So, this development will depends on the initial density of the soil.

So, now these are the results here. In the first case both the graphs A and B are same only what is the difference in A case X axis on the normal scale sigma 3c, Y in the B it is on the logarithmic scale. So, if you change X axis in the logarithmic scale then instead of curve you get a straight line for this and this is like you know again this is nothing but CVR line. So, the CVR line both A and B are representing nothing but CVR line. So, now we are above the CVR line and below the CVR line.

Loose sample, coordinate of loose sample will come above the CVR line and then sample will be below the CVR line. Now, so it shows the behavior of initially loose and dense specimen undrained undrained condition. So, whether you allow drainage or you do not allow for so you have. So, let us see let us let me explain it. So, once you have let us say for the loose sample.

Now, for the loose sample you have two path one path you do not allow you do not allow the drainage that is undrained condition. When you do not allow the drainage in undrained condition there will be no change in volume that means the value of E world ratio is going to be constant. So, in undrained condition will reflect the position where E is constant. So, you are moving E constant and because there will be it is a loose sample there is a development of positive pore water pressure your effective confining pressure sigma 3 c will decrease. So, the path for the loose sample in undrained condition will be shown by these arrows.

While if I allow the drainage then what will happen there will be change in the volume the E will decrease because it is loose sample. So, E it will start compacting. So, E will decrease, but there will be no change in sigma 3 c because there is no development of pore water pressure. So, in drain condition the path taken will be there here. Similarly, for dense sample if I do not allow drainage that means in undrained condition there will be development of positive pore water pressure as a result sigma 3 c will increase, but there will be no change in the E value.

When you allow the drainage in the drain condition the E value will be changed and now because your sample is dilating. So, the E will be moving so and then when you change the value of E that means E will increase. But ultimately whatever path you take whether you have loose sample or dense sample whether you have the drain condition or undrained condition ultimately your path will lead to your point of on this CVR line. And when you reach these two points this for the loose sample and this is for dense sample then you do not have any like you know that they are lying on the same CVR line. So, that means critical void ratio though critical void ratio is different for these two samples.

However, they have reached to the same the CVR line that for the confining pressure. So, B part is the same as E only the difference is in straight line rather than the curve. So, I think this do not require any other explanation. Now coming to the state criteria, this hypothesis was verified experimentally that whatever we have discussed. The CVR line described the state forward which was not any state to the state of the soil which any soil specimen would migrate at large strength whether by volume change there may be two possibility either by volume change under drain condition or change in effective confining pressure under undrained condition.

In undrained condition why effective confining pressure changes because there is a development of pore water pressure or there could be combination under partially drained condition. So, suppose here in one case it is undrained condition, drained condition if you allow partial drainage then your it will not be horizontal or vertical line rather you will move along some path maybe like this or like this or maybe like this source. So, depending on that if you are allowing partial drainage or partial like development pore pressure. So, continue with the CVR the since CVR line mark the boundary between contractive and dilative here it was considered to mark the boundary between state similarity particles soil was or was not susceptible to flow liquefaction. So, because this is the condition actually as we said if your sample is loose then chances are that that it may be susceptible to liquefaction.

So, the CVR line define, and that definition will come with respect to confining pressure whether it is loose or dense that definition is not unique but at what confining pressure. So, confining pressure needs to be defined. Saturated soils with initial void ratio which is high enough to plot above the CVR line were considered susceptible to flow liquefaction and soils with initial states plotting below the CVR line were considered non-substitutable. So, what we are talking is here about flow liquefaction. See I think you mind it that this condition what we are putting is for flow liquefaction not for cyclic mobility.

So, as in the initial of this lecture today we said that these conditions will be set differently for cyclic for loading as well as flow liquefaction. So, if your sample is plotting above the CVR line then it will be susceptible but if it is plotting below then it will not be susceptible and this susceptibility is with respect to flow liquefaction. So, this is here or whatever we have discussed susceptible, non-subsitable, susceptible and this is your CVR line can be used. So, this was about CVR line. Now in this lecture today we are going to talk about the second concept.

First concept was related to CVR line which determined the susceptibility of the soil sample. Second is steady state of deformation which is more advanced than the CVR line and for that like CVR line concept have come from Casagrande. Castro in 1969 performed static and cyclic triaxial test on isotropically consolidated specimens and several static tests on anisotropically consolidated specimen. So, triaxial static and cyclic was conducted on isotopically specimen while static test was on anisotropic consolidation. Three different types of stress-stratiation behavior which is illustrated for anisotropy consolidated specimens are shown next.

So, the text results shown in this figure is for anisotropically consolidated specimen. So, three conditions are there. One is A which we can say liquefaction, B where dilation is taking place, dilation will happen in the loose sample and C where you have the limited liquefaction. So, liquefaction limit and dilution in monotonic loading test and these results is not of cyclic test rather it is from monotonic loading test that means you apply the load in one direction that is static loading basically. What you have in the first case, what is Q? Q is the parameter which is Q and P are parameters related to stress path and if you recall we have already discussed Q is nothing but Q effective same thing sigma 1 minus sigma 3 by 2 while P dash is what you have sigma 1 plus sigma 3 by 2 and which will be nothing but sigma P minus u.

So, that is the sigma 1 and sigma 3 are major and minor principal stresses respectively. So, this is basically sigma 1 minus sigma 3 by 2 this is because Q d this is you have this sigma d basically if I say this will be sigma d. So, this can be treated as a difference sigma d divided by 2, deviatoric stress half of the deviatoric stress. So, Q is basically representing your deviatoric stress interaction and on x axis you have strain here this is this curve is called the second figure is called a stress path. Third figure you have pore water pressure versus strain.

So, if I work on the first for pore water pressure the last one C figure. In this case when the liquefaction for the A sample which was a loose condition what will happen when you put a straining then it will be subjected to the positive pore water pressure and it will is 0 and then excess pore delta u increases at a particular value and at a very high level of strain where you reach to the point of EC the pore water pressure become constant. This is the kind of similar results which we have discussed. But in case of dilation the sample B, A this was A sample in B initially the you develop the positive pore water pressure and positive pore water pressure is related to when the contraction behavior and after that certain point the pore water pressure becomes 0, maximum and then it becomes 0 and then it start negative.

So, this is dilation B. So, this was A you can treat like a loose sample B is like a dense sample, but there could be a sample of combination of loose and dense then in that case what happens for a quite some time you get the positive pore water pressure peak value and then it becomes 0 and the 0 value comes after at a very high strain level and then you get a negative pore water pressure. So, as a result you have in case of liquefaction what will happen the deviatoric stress will reach to the peak value and then it is start decreasing and then it becomes almost constant where the pore water pressure become constant then this will become constant. So, this decreases and then it becomes constant, but at a low value. Similarly, for dilation because there is a development of positive pore water pressure so sigma d will increase here. Same thing we can explain limited liquefaction and the stress path this point the origin Q which was like this point is here in this case.

So, in this case for the case of liquefaction what will happen Q will decrease like it is happening in this figure and p dash what is p dash p dash should be treated p minus u because there is increase in pore water pressure. So, as a result this point will move towards the like you know it will decrease the value p dash will decrease. Similarly, for dilation case there will be increase in Q, but at the same time p dash will be increased which can be reflected. But in the C case you will have for some time limited liquefaction occurs up to this point which is phase transformation point and then after dilation starts and then it is reached to the point C.

So, this is a different test. So, I think I have already explained about this that what is happening. Very loose specimen such as specimen A exhibited a peak undrained strength at a small shear strength and then collapsed to grow rapidly to large strength. This type of behavior is called liquefaction. While in the dense specimen B initially contracted little bit only, but then dilated until a relatively high constant effective confined pressure and large strength. If you have intermediate densities that is specimen C in that case it will be combination of A and B the result will be combined from the A and B and limited period of strength softening behavior takes place which with undrained after once a dilation takes place.

So, this reversal from contact to dilate at the phase transformation point. So, what is the phase transformation point? Phase transformation point in this case is here where your sample first contract and then it start dilating here. As a result, so for this case for some time the positive pore pressure develops, positive pressure is developing up to this point and then it start decreasing and after this phase transformation point negative pore pressure will develop because dilation takes place. So, this happens in because most of the sample could be between loose and dense.

So, let us work on this C1. So, the loading produce continued dilation to higher effective confined pressure and higher large strength. This type of behavior was termed as a limited liquefaction. So, continue with this, but the testing program have shown a unique relationship between void ratio and effective confined pressure at large strength and this kind of relationship we have already seen that this kind of relationship also existed for the case of CVR line when we discuss the CVR line. Graphically this relations plotted below and roughly parallel to the CVR line which is the same case obtained from drained strength control test, the difference being attributed to the development of flow structure under stress control conditions.

So, here like we are doing it under stress control. The state in which the soil flowed continuously under constant shear stress and constant effective confined pressure at constant volume and constant velocity are defined as the steady state of deformation. So, this is I think very important that how we define a steady state of deformation. So, the definition of a steady state of deformation is given here. This is the state of the soil in which soil flowed continuously and this flow takes place under constant shear stress that means you fix the value of shear stress and at constant effective confined pressure sigma 3c. So, shear stress means basically sigma d, sigma d is constant and if I say constant effective confined pressure that means sigma 3c is constant and constant volume, volume constant means basically you fix the value of e which is at critical and constant velocity.

So, velocity will be also like and then in that case we say that soil have reached to the steady state of deformation. Since the steady state of deformation is reached only at large strain and this condition will come only large strains after the effect of initial conditions such as soil fabric, stress and strain history and loading conditions have been like you know that you have reached to that point. The effective confining pressure in an element of soil in the steady state of deformation was considered to depend only on the density of soil. So, when it will reach whether you start as a loose sample or dense sample. So, this is similar to what we have in this case of CVR line, but it is in advance compared to CVR line.

Now, in this case also the locus of points describe in the relation between the void ratio that is e and effective confined pressure in the steady state of deformation is called the steady state line. So, in general it looks like CVR line what you are doing you are plotting the critical void ratio with respect to confining pressure. Here we are plotting the relationship between the void ratio which corresponding to the steady state condition with the effective confined pressure and this first thing is that these coordinates of effective void ratio and confining pressure should belong to the steady state condition and we already defined the steady state condition. So, the locus of these points will called steady state line. In most general form the SSL can be used as three dimensional curve and in 3D case what you will have you have three axis one is void ratio e another is sigma prime that is normal effective stress and then toe which is shear stress.

So, this is called e, p and q space. Basically toe is replaced by q because toe and q, q is nothing but sigma d by 2. So, like you have shear stress is replaced by q. So, that is the same thing and if I have I do not have e then p dash and q will be called this curve is called in 2D stress path. So, there could be 2D version and 3D version.

Let us start from 3D and then will come. The SSL therefore represent the projection of three dimensional SSL into a plane of constant toe. So, you have like one of them if you keep constant then you will have in two dimension. So, in 3D how the SSL looks? This is the SSL which is denoted by thick line this is a curve and what you have three axis one is e another is sigma prime that means effective normal stress and on vertical direction you have toe which is similar to toe or it could be toe or this axis could be q also. So, it can be also replaced by q that which is the same thing representing the same thing. Now what do you have? If I draw the projection of this SSL line on different planes like for example on the horizontal plane which consists of your e versus sigma then I get this projection on e sigma plane this will be the here.

If you draw the projection on e toe plane then this will be the curve and so if you draw the projection on what you call toe sigma plane then you will get a straight line horizontal line. So, different projections can be drawn and then it can be worked. But let us because it will be difficult to deal with the three dimensional case so we for simplicity we will work 2D SSL. The SSL can also be projected so this we already discussed. The SSL can also be expressed in terms of the what we call the steady stress strength which is SSU. SSU stand for steady stress strength. Since the shearing resistance of the soil in the steady state of deformation is proportional to the effective confining pressure which we have also discussed the strength based SSL is parallel to the effective confining pressure based SSL when both are plotted on the logarithmic scale. So, it comes like this. So, you have two SSL here on both A and B figure on y axis what you have word ratio while on x axis in the first case you have logarithmic of confining pressure sigma 3 c and on B you have logarithmic of steady state strength log SSU. So, both are more or less similar. Here one thing is important if you recall in CVR line you have the same axis as A, E was there on and on confining pressure was there.

Then what is the difference? The difference comes here that this line in this case locus of the points belong to the steady state condition. So, we need to make sure that this these points are related to steady state condition. And proportionality of SSU to sigma 3 to produce strength based on the effective confining, if you see both are more or less same. So, that means here what we say the sigma 3 c and SSU are proportional to each other steady with initial identical slopes. So, now what is the use of SSL? The SSL steady state line is like CVR line similarly you have the SSL is useful for identifying the conditions under which a particular soil may or may not be susceptible to flow liquefaction.

Soils whose state float below the SSL are not susceptible to flow liquefaction. So, the flow for the flow liquefaction condition it is must like that they should float above the SSL. A soil whose state lies above the SSL will be susceptible to flow liquefaction only if the static shear stress exceed its steady state or residual strength. Now, here is the difference. Simply because your soil sample is floating above SSL will not ensure that it is subjected to flow liquefaction.

Then another condition need to be satisfied that static shear stresses should exceed its steady state. So, this is the case here like you have, so in the same slide, yeah this is the same slide as we discussed last. So, if it is floating above or below this SSL accordingly we can decide. Since the SSL can be used to evaluate the shearing resistance of liquefied soils it is also useful for evaluating the effects of liquefaction. So, this will be also useful when we talk about how that and what is the effect of liquefaction.

Although determination of the position of the SSL can be difficult in practice the SSL though finding the SSL line is difficult but once SSL is ready it is quite useful for understanding the basis of liquefaction. Then the second phenomena which was we did not discuss when we talk about that CVR line that is cyclic mobility. Cyclic mobility can occur in soils whose state float above or below the SSL, in other words cyclic mobility can occur in both the loose and dense end. So, the for FLS for the flow liquefaction the condition must that it should float above the SSL. But for the cyclic mobility case it can even float above, or it can be floating below the SSL.

The nature of the steady state line illustrate the limited applicability of absolute measure of densities such as void ratio and relative density for characterization of a potentially liquefiable soil. So, as shown in the next figure an element of soil at a particular void ratio hence a particular density and relative can be substitute of flow liquefaction under a high effective confining pressure but non substitute at low effective confining pressure that is there. So, this is done using what you call the state parameter. So, what we are discussing is state parameter here and this state parameter is one actually what happens in the state parameter. Whatever we discuss we try to compact in one single parameter and what is that single parameter that can be explained using this.

Here SSL we already defined in the last slides. What is in SSL you have E versus logarithmic of sigma 3c which is looks like similar to a severe line, but it is in a steady state condition. Now in this case initial state of the void ratio of the sample is here and steady state if I draw a vertical line down here then it will projection of this point at this SSL will be this one. So, difference between these two value of E is defined by parameter psi this parameter which is a what is this parameter this is nothing but difference of E minus ESS. So, if this parameter psi is positive then the chances of liquefaction will be there.

If this psi is negative then it will not liquefy rather it will be so there. So, depending on the whether this parameter positive or negative it can be liquefaction, susceptibility related can be defined by this and this psi is called state parameter is nothing but this state parameter psi. So, this state parameter is used for determining the susceptibilities of liquefaction. Using the concept of critical state soil mechanics the behavior of a cohesion less soil should be more closely related to the proximity of initial state than to the absolute measure of density. In other words soils in states located at the same distance from the steady state line should exhibit similar behavior and state parameter was defined this state parameter was defined by Been and Jeffries and psi and this is nothing but E minus ESS. Now if E is greater than ESS psi will be positive E is less than ESS then it will be negative.

Where ESS is the void ratio of the steady state line at the effective confining pressure of interest. Naturally the value of ESS is not constant it will keep changing for a void ratio. So, whenever we determine this the state parameter we need to keep the same confining pressure. So, for these both the points the confining pressure is same.

If I change the confining pressure then not only E, ESS value will change. So, this need to be determined. When the state parameter is positive the soil exhibit contractive behavior, susceptible to flow liquefaction. When it is negative dilative occurs soil is not susceptible to flow liquefaction we already discussed has been related to friction angle, dilation angle, CPT resistance, PMT results and D. So, what has been done this state parameter psi has been linked with many of the properties which is psi is linked with the function phi or it could be linked with the SPT resistance the n values or CPT resistance or other results.

Ishihara in 1993 showed that there is ability of the state parameter to characterize soil of very loose and under low effective confining may be limited and proposed an analogous parameter the state index based on the relative distance between the initial state and the quasi steady. So, I mean there is as we discussed in the beginning of this soil liquefaction soils that there are on this topic there are some controversy. So, this state parameter was established, but some of the exceptions has been found where this state parameter is not able to predict that equation. So, this corresponds to the stress and density condition at the phase transformation points observed in cases of limited liquefaction. The concept of the state parameter is very useful and the possibility would remain its value from the institute test is appealing that you once you will have the institute test data SPT data or angle of internal friction of phi then you can find the value of psi the state parameter and then you can determine whether liquefaction will occur known.

The accuracy which is this can be determined is influenced by the accuracy which is the position of the SSL can be determined. So, with this I conclude this lecture that is the 33 lectures and this was the second lecture on the liquefaction susceptibility. So, with this we conclude the liquefaction susceptibility. Thank you very much for your kind attention. Thank you.