Earthquake Geotechnical Engineering

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Lecture 39

Initiation of Liquefaction (Conti.)

I welcome you all for this NPTEL online course on earthquake geotechnical engineering. And in this course, we are at lecture number 39, which is on liquefaction of soils. In fact, you may be aware that we are under the module 4 of this course, which is on liquefaction of soils. And in this liquefaction topic, we are under the third chapter that is initiation of liquefaction, in the initiation of liquefaction, almost we are at the end. So, we already discussed under this chapter number 3rd for this module flow liquefaction surface FLS, influence of excess pore pressure has been already covered.

Then we talk about how to evaluate initiation of liquefaction and there were two approaches, one is cyclic stress approach, another is cyclic strain approach. We have already done with the cyclic stress approach and today we are going to talk about cyclic strain approach, which is quite similar. And in fact, on the same lines as well for the cyclic stress approach, where we characterized earthquake loading and this earthquake loading can be characterized based on simple method and ground response analysis. But in this case for cyclic strain approach, it will be done only on based of the simple method.

Then second, we are going to characterize the liquefaction resistance in terms of cyclic strains. So, let us talk about that. So, cyclic strain approach, this which will be called in the short CSNA and this have three topics characterization of loading conditions, earthquake loading condition, then second liquefaction resistance and third one is evaluate liquefaction potential. So, before going ahead let me acknowledge that most of the contents of the lectures are from the Kramer's book, but I will explain each and everything related to that. Now, coming to this the large number of factors, why the cyclic strain approach is required? Here because as we have seen during the cyclic stress approach, there are a number of parameters, number of factors which influence the liquefaction potential and liquefaction resistance of the soil.

So, the number of factors that influence the cyclic stress required to produce liquefaction make the laboratory evaluation of liquefaction resistance in the cyclic stress approach difficult. Because there are so many parameters which influence whatever you find the liquefaction resistance using cyclic stress approach. So, as a result in 1976, he pointed out that liquefaction characteristics from institutional deposits are determined by number of complex factors and out of these complex factor one of the most important which is normally used is called relative density of the soil, but that is only one factor that is not only the solo factor. And you know the relative density of the soil as well as void ratio more or less same thing, they are on another side sometime represented using the n values, SPT n values. So, ultimately, we need to understand this is only a one factor and there are other factors which also influence the liquefaction resistance. As a result, a more robust approach later on Dobry and Ladd in 1980 and Dobry et al in 1982 described an approach that use cyclic strains rather than cyclic stresses to characterize earthquake induced loading and liquefaction resistance. So, this later studies have used cyclic strains rather than cyclic stresses. Since the tendency for a sand to densify when dry is directly related to its tendency to develop excess pore pressure when saturated. In fact, there is an analogy between the saturated sample and dry sample. In case of dry sample when you do the loading then what will happen there will be development of volumetric strains.

When you have the saturated sample then there will be development of pore pressure. So, both are related. So, they are fundamentally related to and both this tendency to develop for the volumetric strain or excess pore pressure they are related to cyclic strains rather than cyclic stresses. In the cyclic strain approach earthquake induced loading is expressed in terms of cyclic strains. In case of cyclic stress, it was expressed in terms of cyclic stresses.

So, the time history of cyclic strain in actual earthquake is transient and irregular it is not like you know that regular time history and this was the case for the cyclic stress too. To compare the loading with laboratory measured liquefied resistance it must be represented by an equivalent series of uniform strain cycles. So, this is also on the same lines as we discussed in the cyclic stress approach that to find the liquefied resistance using the laboratory data you need to find the uniform loading that means uniform amplitude and that is we apply a factor of 65% of where toe cycle was if you recall was 0.65 toe max. Similarly, in this case computation of cyclic shear strain is also done from like simplified method because using computation of cyclic strain using ground response analysis is difficult.

So, using simplified method which is proposed by Dobry et al in 1982, an equation has been proposed to calculate the cyclic shear strain which is given from this relation 0.65 a max by g sigma v rd where sigma v is a overburden pressure total overburden pressure it is not effective overburden pressure and rd is the reduction factor which varies with the depth and if you recall this value of rd decreases when depth increases at the ground surface when z equal to 0 rd equal to 1 and then when z increases this value decreases and g shear modulus is a function of gamma cycle. What is gamma cycle is the shear modulus of the soil and that shear modulus is gamma equal to gamma cycle whereas what is gamma cycle gamma cycle is basically cyclic shear strain. Since gamma cycles influences both sides of this equation if you see because shear modulus g is not constant rather that is the value that is gamma cycle and in fact you do gamma cycle is unknown which is coming on the right hand side also as a result you need to use the iterative process and to solve this equation and iterative process you use and the iterations will be done when there is a convergence that means g max is for starting what you can do to start the iteration g can be taken as a g max and then you update you find the tow cycle and then update again g value depending on that modulus reduction curve using a modulus reduction curve and then until there is a matching both sides then we finish this iteration that is the usual process.

$$\gamma_{cyc} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v r_d}{G(\gamma_{cyc})}$$

Now coming to the characterization of loading conditions the equivalent number of strain cycles which is called in the short n equivalent will depend on the earthquake magnitude this is similar to that we have seen in case of cyclic approach, and we see that for a 7 magnitude earthquake neq was 10 while for 7.5 it was 14 number of cycles. So, once gamma cycle is determined it can be compared with the threshold shear strain gamma t what is gamma t gamma t is the value where the liquefaction will start if your gamma cycle is naturally less than gamma t then no pore pressure will be generated, and liquefaction cannot be initiated in this case we stop our liquefaction hazard evaluation at this stage. So, if cyclic strain generated inside the soil is less than the threshold value cut off value gamma t which is required strain for then we can say that there is no chance of liquefaction and we can stop. However, if your gamma cycle is greater than gamma t in this case liquefaction will be possible and the liquefaction of the soil must be evaluated in that case. So, first thing you what you do you determine based on your loading condition if your loading condition which is coming due to earthquake is enough so that your cyclic strains created is more than the required if it is not then you can stop there is no need to find the liquefaction resistance in that case.

Assuming that gamma t cycle is greater than gamma t, in that case we need to go further and then we need to next step will be characterization of liquefaction resistance. The cyclic strain approach simplify the interpretation of liquefaction resistance from laboratory tests. Experimentals evidence indicated that the factor those factors which increases the cyclic stresses required to initiate liquefaction for example density, soil fabric, strain history, over consolidation ratio, length of time under sustained pressure they also increase the shear modulus of the soil. So, those factors which increase the cyclic shear stress also increases shear modulus of the soil. So, as a result if we find because these factors both influence and they are influenced similar way similar way in the sense if due to one factor cyclic stress increases shear modulus also increases if it decreases that is also decreases.

As a result, the ratio of these two which is said tau cycle divided by g and this ratio is nothing but gamma cycle. So, the effect of these factors on this ratio of tau cycle is much smaller. So, that means the cyclic shear strain is not much influenced by the factors. So, these factors have little influence on the pore pressure generation interpreted in terms of cyclic strain and the Dobre and Ladd have provided evidence for that. So, the insensitivity of the generated pore pressure to factors other than the cyclic strain amplitude is a hallmark of the cyclic strain approach. That goes in the favor of cyclic strain approach because there are many factors which influence both g and tau cycle in a similar way. So, that means their effect on cyclic shear strain gamma cycle is very less very little. So, for example, in this case what is in this case that this has been given by Dobre and Ladd and in this figure what you have on x axis shear strain cyclic shear strain which is varying which is varying from 10 to power minus 3 to 1 percent on y axis you have the value of R u which is varying from 0 to 1. Now, you have different data points on this and what is changing in these data points three parameters especially when preparation is changing, effective confining pressure is changing, samples which has been used, but what you see that all are lying on this like on one curve that means these factors are not like you know because they are almost you know on these curves. So, the data matching is very good.

So, that means these factors are not influencing much the result because they are still lying along this line which is a graph. So, this is major pore pressure ratio after strain cycles of loading in strain controlled cyclic fractional test and this has been done by Dobre. So, this is an evidence that it is telling that many factors do not influence the cyclic strains. Coming to this, suppose we have for cyclic strains approach between the value of R u number of cycles there was a relation if you recall between R u and n by nr number of cycles and similarly there is a relation between cyclic shear strain developed for a given for a particular value of pore water pressure R ut. R ut is basically nothing but pore water excess pore water pressure ratio at any time t and n aq is the number of equivalent cycles of loading that will depend on your magnitude of earthquake which we have already discussed in detail.

So, in this case those cycles is given by this relation and in this relation what you have like you know this there are two more parameter alpha and beta are coming and these are the parameters alpha and beta are experimentally determined functions of principal effective stress ratio k which is k c which is used to characterize the cyclic strain approach. So, the k c what is k c? k c is nothing but principal effective stress ratio. So, this value of alpha and beta will depends on the value of k c and for given k c alpha and beta can be determined and then they can be used. So, this was about characterization of liquefaction resistance. Once you have to characterize the loading earthquake loading you have characterized the liquefaction resistance.

Now, the next step is to evaluate the liquefaction potential and this liquefaction potential may be evaluated in the cyclic stress approach in a similar fashion as we did for cyclic stress approach. What is done? The cyclic loading which is imposed by the earthquake is characterized by the amplitude of series of n e q uniform strain cycles is compared with the liquefaction resistance which is expressed in terms of the cyclic strain amplitude required to initiate liquefaction in the same number of cycles. So, what is done? Like you have two things one side earthquake loading on another side you have liquefaction resistance. So, both are represented in terms of one parameter single parameter that is cyclic strain and once you do that liquefaction can be expected at the depths where the cyclic loading exceeds the liquefaction resistance, and both are represented in terms of cyclic strains. Since loading and resistance are both are in terms of strains rather than stresses a factor of safety against liquefaction is not yielded.

So, it is the case here what you have here you have in this zone shear strain amplitude on x axis with the depth in the cyclic stress approach you have shear stress instead of here instead of strain you have the that is only the difference. Now, what do you do? You have equivalent cyclic shear strain which is induced by an earthquake magnitude this is the case here. And on another side you have the cyclic shear strain which is required to cause liquefaction. So, the loading strain caused in this depth from here to here due to earthquake is more than the other liquefaction resistance of the soil and this liquefaction resistance of the soil is in terms of cyclic strains. So, basically this will be your zone of liquefaction.

The liquefaction will occur along this depth, and this can be determined by comparing two cyclic strains. One cyclic strain is due to earthquake loading another is liquefaction resistance of the soil in terms of cyclic strains. So, the primary advantage of the cyclic strain approach derives from the strong relationship between excess pore pressure generation and cyclic strain amplitude. For a given soil EPP can be predicted more accurately from cyclic strains than from cyclic stresses. So, that is a that goes in the favor of cyclic strain approach that cyclic strain can be predicted more accurately. However, cyclic strains are considerably more difficult to predict accurately than cyclic stresses. This is true that many factors do not influence the cyclic strains, but they influence the cyclic stresses. However, determination of cyclic strains in the laboratory is not easy that is difficult. As a result, the cyclic strain approach is not much used still not commonly used in this as the cyclic approach in earthquake geotechnical engineering practice. Now, coming to this, like this completes our chapter number 3 on liquefaction of soils that is on initiation of liquefaction.

We are now we will work on the last chapter of this module on liquefaction of soils that is effects of liquefaction. Let us say the liquefaction is occurring, how it affects the environment, how it changes. So, these things we are going to discuss. So, as far effects of liquefaction that is the chapter number 4 for this module, the topics we are going to cover in these chapters are listed here. The effects of liquefaction, first effect is alteration of ground motion. When the liquefaction occurs, it may alter the input ground motion which is coming and when the after the liquefaction, so when the ground motion passes through the liquefiable soil, so it will be quite change before it reaches to the base of the structure or like you know at the site. Then the second development of sand boils which is very interesting. Then we have third settlement of dry and saturated sands and finally, we are going to talk about instability which is caused by the liquefaction of soils and that will be in terms of shear strength of liquefied soils, flow failures and deformation failures. So, today in this lecture, we are going to talk the first two topics only, alteration of ground motion, development of sand boils. Rest of the topics will be covered in the next lecture, that is lecture number 40.

So, let us talk about effects of liquefaction in terms of alteration of ground motion and development of sand boils. So, when we talk about the effects of liquefaction, it can affect the buildings, bridges, buried pipelines and other constructed facilities in many different ways. So, that is well known that liquefaction and liquefaction can also influence the nature of ground surface motions. Flow liquefaction can produce massive flow slides and contribute to the sinking or tilting of heavy structures. The floating of light burette structures and to the failure of retaining structures.

So, this was about flow liquefaction, but as far as the cyclic mobility is concerned, it can cause slumping of slopes, settlement of buildings, lateral spreading and retaining wall failure. So, many effects are there, one for the flow liquefaction as well as for cyclic mobility. As for the like one of the effect of that as we discussed is alteration of ground motion that the liquefaction of soils will alter the ground motion and how this alters. So, this is the development of massive excess pore pressure causes soil stiffness to decrease during an earthquake. When earthquake comes, then what happens? The positive excess pore pressure will develop and when positive excess pore pressure develops, what happens? Effective stresses decreases. Once effective stresses decreases, then the shear strength reduces. So, this causes the soil stiffness to decrease number one. Then a deposit of liquefaction that is relatively stiff at the beginning of the earthquake may be much softer by the end of the motion. So, due to the seismic excitation, there will be softening of the soil and that means stiffness will be reduced, stiffness is it is going to be reduced. As a result, the amplitude and frequency content of the surface motion may change considerably throughout the earthquake.

You know that natural frequency of the system depends on the stiffness of the system. Now what happens due to the liquefaction, when the soil gets softened, it will lose its stiffness and once its stiffness decreases, the natural frequency of the system will decrease and amplitude and frequency content which you obtained will be different than the earlier. So, when the liquefaction starts, the behavior will be changing and this behavior will keep continuously changing as the level of liquefaction increases. In the most of extreme case, the development of very high pore pressure can cause the stiffness and strength of even a thin layer to be so low that the high frequency component of a bedrock motion cannot be transmitted to the ground surface. So, for example, this effect is shown in one of the slide where it is not difficult to identify the point at which liquefied induced reduction of the stiffness of the underlying soil took place where the acceleration of frequency content dramatically changed.

For example, this is the case here. In this case, this is an accelerogram from the near apartment building testing on liquefied soil in 1964 Niigata earthquake. So, if you see clearly in this acceleration time history, what you could see before 7 seconds, this is the line here 5, 6, 7. So, if I draw a line here 6, 7 second will be here. So, in this case, on the left hand side, frequency content is quite high, you have very like you know that crowded. So, frequency is high, and amplitude is also different, but when you go on the right hand side acceleration time history, frequency decreases and the frequency content that means it is like it was elongated. So, it is become sparse. At the same time, there is a change and the increase of the amplitude also. This amplitude is here, while this amplitude is here. So, there is a large difference between the amplitude. So, as a result, so one of the issue of the liquefaction is that it altered the ground motion. So, the ground motion before the liquefaction was before 7 second, after 7 second, its characteristics have changed and this change of characteristic not only in terms of amplitude, but there is a change in the frequency content also.

So, this is one of the effect of the. Continue with the alteration of ground motion. The surface acceleration amplitude decrease when excess power pressure becomes large does not mean that damage potential is necessarily reduced because low acceleration amplitude at low frequency can still produce large displacement. So, even sometime it may happen that there may be decrease in the amplitude and it becomes acceleration amplitude is low and this becomes low particularly when excess power pressure becomes large. But still that does not mean that that damage potential have decreased rather low amplitude at low frequencies will produce high displacements as a result and then high displacement that will be damaging. These displacement may be of particular concern for buried structures, utilities and structures which are supported in pile foundations.

For example, this is the case here. If you have a pile which is passing through the liquefied ground and right now what you could see there are three layers on the top layer is non-liquefiable, middle is liquefiable and third one is also non-liquefiable. So, the middle layer is liquefiable. However, potential effect of surface liquefaction here, so this layer will here it will go bending. So, there will be the large strength that may develop in can induce bending moments, high bending moments will be developed, where high bending moments will be developed in this layer particularly the junction of the non-liquefied and liquefiable. So, your pile get bent and this is basically also the effect of what we say the liquefaction which is causing the alteration of ground motion. Now, the second part on the like you know the second step which is related to the effects of liquefaction, development of sand boils. Liquefaction is often accompanied by the development of sand boils which has been observed during many past earthquakes. During and following earthquake shaking, seismically induced excess pore water pressures are dissipated predominantly by the upward flow of pore water. Because in this what happens in this case of development of sand boils, the excess pore pressure flow upward and then it come out. So, as a result it will be like you know that and this is called the development of sand boils.

This flow produces upward acting forces on soil particles and as those forces, upward producing forces can loosen the upper portion of the deposit and live in a state of susceptible to liquefaction in future earthquakes. So, if it is subjected to future earthquakes again, if the hydraulic gradient driving the low flow reaches a critical value, the vertical effective stress will drop to 0 and the soil will be in a quick condition. In such cases, the water velocities may be sufficient to carry soil particles to the surface. What happens when there is a flow of water from downward to upward in case of liquefaction that then if flow is large then it will carry with this some soil particles will be taking along with this one.

In the field, the soil conditions are rarely uniform. So, the escaping pore water tends to flow at high velocities through localized cracks or channels. Sand particles can be carried through these channels and ejected at the ground surface to form what is called the sand boils. For example, some of the sand boils which has been seen, observed during the 2001 Bhuj earthquake. So, in the figure A, moderate size sand blowing in Great Run has been shown and in the figure B, sand blow craters in dry river bend has been shown. When there was water coming out of during liquefaction, then it was like a fountain, water fountain coming out of these sand boils. And in fact, during Bhuj earthquake, what happened? Many of the media claim when the water came due to the liquefaction out that there is a revival of river Saraswati, but that was not true, it was the liquefaction phenomena. The development of sand boils is a complicated and somewhat random process. It depends on the magnitude of the excess pore water pressure, thickness, density and depth of the zone of excess pore pressure and the thickness, permeability and intactness of any soil layers that over lie on the zone of the high EPP. What happens? Sometime water is developing, but on the top of it you have a steep layer, and that steep layer may not allow water to eject out of it.

In that case, the sand boils may not develop. However, low permeability of silty sand may prevent pore water from flowing quickly enough to produce sand boils even if high excess pore pressure exists. Sand boils are of little engineering significance by themselves, but they are useful indicators of high excess pore pressure generation. If there is a development of sand boils in the field during earthquake, then it is an indicator that the development of excess pore pressure is high below the layers. These are developed when the water inter layers break through to the ground surface. Some redistribution of the soil grains is also likely to accompany the formation of water inter layers. Specifically, the sand which is immediately beneath the water, and it may be loosened by the part flow of water in the next cycle. So, this was all about like effects of liquefaction. We will continue in the lecture number 40, the remaining topics from the chapter 4 of this module, which is on effects of liquefaction. So, I thank you very much for your kind attention. Thank you.