Earthquake Geotechnical Engineering

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Lecture 40

Effects of Liquefaction

I welcome you again on this NPTEL online course on earthquake geotechnical engineering. This is lecture number 40. This is the last lecture on this topic of liquefaction of soils. So, this is the last lecture of the module 4 and we are under chapter 4 of the module of this module effects of liquefaction which we have started in the last lecture number 39. During this chapter, already we have covered effects of liquefaction in terms of alteration of ground motion and development of the sand boils. Today in this last lecture on this topic, we are going to talk settlement which we are going to discuss for both dry and saturated sands.

Then next, we are also going to talk about instability and when we talk about instability, we will talk instability in terms of shear strength of liquefied soils. Then we are also going to talk flow failures and finally, deformation failures. So, now let us start the first one that is settlement dry and but before that let me acknowledge that most of the stuff is from the Kramer's book but there are some slides which is from other sources also. So, when we talk about effects of liquefaction settlements, subsurface densification is manifested at the ground surface and this is manifested in terms of the settlement.

So, what happens, whatever the condition you have the know if your subsurface condition is not good, then what will happen? When the building or the base or facility, they will settle. So, settlement is indicating what is the type of the subsurface condition. If subsurface condition is good, then settlement will be less. If subsurface condition is not good, very weak, then there will be high settlement. So, this is manifested and particularly earthquake induced settlement frequently cause distress to structures which are supported on shallow foundations, damage to utilities that serve pile supported structures and damage to lifelines that are commonly buried at shallow depths.

Why it is at shallow depth? Because as we discussed from the beginning that the chances are that the liquefaction occurs in the shallow depth. Dry sand deposit will densify very quickly because in the dry sand what you have the air inside the voids and when you apply the loading there will be compaction and the rate of compaction is a faster process than what you have the consolidation you may be aware. So, the settlement of dry sand deposit is really complete by the end of an earthquake. So, like if your soil sample is dry, deposit is dry, earthquake goes within few or last for about a minute or so, when the earthquake goes then there will be settlement will be completed. However, the settlement of saturated sand deposit requires more time as settlement can occur only as earthquake induced excess pore pressure dissipates.

So, what happens here there is a difference. When you have the saturated condition then it is not like air when you apply some pressure air comes out rather because there is a water and due to the shaking, there is a development of what we call excess pore pressure. So, the first thing is that there will be development of excess pore pressure and after the development of excess pore pressure this excess pore pressure get dissipated then only, we say that after dissipation of excess pore pressure the settlement will be complete. The time required for the settlement in the saturated deposit to occur will depend on the permeability and compressibility of the soil deposit and on the length of drainage path and this time which is required for the settlement can range from a few minutes to about a day. Estimation of earthquake induced settlement of sand is difficult, it is not easy task and there could be errors of 25 to 30, 50 percent for static settlement predictions even less accuracy can be expected for the dynamic case.

Now coming to that some of the effects of settlement due to liquefaction induced settlement in the past earthquake. This figure is from the 1964 Niigata earthquake where a building tilted and this is liquefaction induced settlement and this is tilting of an apartment building in 1964 Niigata earthquake. Continue with the settlement of dry sands, the densification of dry sands which is subjected to earthquake loading depends on the density of the sand. The amplitude, if you have the very loose sand then it will be faster rate. Then it also depends on the amplitude of earthquake cyclic shear stress amplitude applied.

If you are applying the higher shaking it will be the faster rate and also depend on the number of cycles of shear strain which is applied during an earthquake. It can be estimated using detailed ground response analysis with corrections for the effects of multi-direction shaking or by simplified procedure. So, using ground response analysis the settlement of dry sands can be like estimated. And there could be simplified process also. For example, in the simplified process the effective cyclic shear strain which we say gamma cycle is estimated by a processor which is similar to that is proposed by the cyclic strain approach to the initiation of liquefaction.

So, gamma cycle is given from this relation 0.65 a max by g and what you have in this relation. This relation we already discussed and this is an iterative relation because on the right-hand side value of shear modulus g is a function of cyclic shear strain which is not known. So, what we do we start our iteration assuming g equal to g max then you find the cyclic shear strain and whatever cyclic shear strain found out then update the value of g shear modulus corresponding to that cyclic shear strain and this process continues. The effective cyclic shear strain can then be used along with the relative density or SPT resistance of the sand.

To estimate the volumetric strain due to compaction and this the volumetric strain due to compaction in the short will be called epsilon c this is due to compaction and what is the effect on this volumetric strain we are going to discuss. So, this is let us say in the next slide let us first discuss this. What you have in figure a and b y axis is same volumetric strain due to compactions and on axis is also same cyclic shear strain which is so both figures are x and y axis are same. But in part a there are three lines which is correspond to different relative density dr 45%, 60%, 80% and this for 15 cycles this is also for 15 cycles. In figure b instead of relative density you have SPT value n values which is ranging from 5 to 40.

So, naturally when the relative density increases for the same cyclic strain the volumetric strain which develops will decrease or if I go other way when the volumetric this relative density decreases your cyclic for the same cyclic strain the value of volumetric strain will increase when you go from top to down. Same thing is here that when you decrease the value of n SPT n then this cyclic this volumetric strain will increase and these both the figures are for a magnitude 7.5 earthquake. In many cases with saturation strong ground motion produces excess power pressure that are not sufficient to produce initial liquefaction and these excess power pressure will dissipate. However, there could be change in the volume.

So, this is like here caution is here there could be case when there is a development of excess power pressure, but that excess development of excess power pressure is not enough to cause the liquefaction, but it does not mean that there will be no development of volumetric strain. There once excess power pressure gets dissipate then there will be change in the volume and the volumetric strain will develop. Now, this table this figure was for 7.5 magnitude earthquake if you want to calculate the corresponding strain volumetric strain for other magnitude of earthquake then the values which you obtain from this figure need to be multiplied by a factor for 7.5 magnitude earthquake the factor will be 1, but for other magnitude for example, 6, 60 percent or 6.75, 0.85 and so on. But if magnitude is higher then the factor will be more than 1. So, and in between as usual you can do the interpolation to find out the value of the volumetric strain. So, this was all about dry sands. Now, coming to the settlement of saturated sands in the dry sand there is a no development of pore pressure excess pore pressure is out of efficient what is the there is a change in the volumetric strain, but here in the saturated sand you will see for the undrained condition if you do not allow the drainage then there is no change in the volumetric strain rather than there will be development of excess pore pressure.

So, the post-earthquake densification of saturated sand is influenced by the density of the sand. The maximum shear stress induced in the sand and the amount of excess pore pressure which is generated by the earthquake. Laboratory experiment have shown that the volumetric strain after initial liquefaction varies with relative density and maximum shear strength. So, volumetric strain is not constant rather it will vary with the relative density and the maximum shear strength which is encountered. Tokimatsu and Seed in 1987 used a correlation between N 1 60 and relative density.

So, the correlation is done between N 1 60 and relative density and an estimate of the shear strength potential of liquefied soil from N 1 60 and cyclic stress ratio which is seed like that has been seed et al in 1984 has produced a chart which is given there that allows the volumetric strain after liquefaction in a magnitude 7.5 earthquake to be estimated directly from the cyclic stress ratio and SPT ratio. So, in this chart which is given by the Tokimatsu and Seed in 1987 and in this chart what you have? You have N 1 60 which is the corrected values for the SPT blow count which is varying from 0 to 40, 50 but you do not have data more than 40. Then on y axis you have cyclic stress ratio for magnitude 7.5 earthquake and in this case, you could see that there are different curves depending on volumetric strain in percentage 0.1, 0.2, 0.5, 1, 2, 3, 4 up to 10. So, if you know that if how we can read this chart, when you for let us say for the given value of CSR cyclic stress ratio, if I increase the value of N 1 60 then you could see easily that when N 1 60 is increases then the volumetric strain decreases. At low value it was higher. So, higher the value of N 1 60 then the volumetric strain decreases. Similarly, for the same value of SPT if I draw the curves here, then for the same value of SPT data if I draw a line then what happens? The volumetric strain increases as the CSR increases that is the cyclic stress ratio.

If you increase the cyclic stresses then it will increase. So, that means volumetric strain will depend not only in SPT but it will also depend on the cyclic stresses applied on the amplitude of cyclic stress which is expected. Continue with the effect settlement of saturated sand and in an alternative approach either the factor of safety against liquefaction or the maximum cyclic shear strain and the relative density SPT resistance or CPT resistance can be used to estimate the post liquefaction volumetric strain. So, like we have talked about the liquefaction resistance, similarly volumetric strain can also be estimated by this figure and let us discuss in this figure first. In this figure what you have on the x axis you have post liquefaction volumetric strain in terms of percentage while you have the factor of safety on y axis.

So, the factor of safety is ranging from 0 to 2 and this is the line where factor of safety equal to 1. Let us say when the factor of safety decreases from top to bottom that means 2 to 1 and then further then what you could see for the same value for the one curve for the single curve for example dr equal to 30 percent. What we see that when the factor of safety is decreasing then volumetric strain increases exponentially. It was here and then all of sudden it increases particularly after this point. Here volumetric strain after the when the factor of safety start decreasing less than 1 then the volumetric strain increases tremendously.

When the factor of safety is more than 1 the increment was not so much when the factor of safety becomes greater than 1 then the increment in volumetric strain is quite high.

Similarly, if I increase the relative density of the soil then there is a decrease in the volumetric strain for the same value of factor of safety. So, that if you go and as a result when dr increases from 30 to 90 percent then there is a quite decrease quite much decrease in the value of volumetric strain. Then another effect is given the maximum value of strain on this chart. So, many phenomena have been given.

Integration of these volumetric strain over the thickness of the liquefied layer produces the ground surface settlement. So, this all was about two factors one was settlement of saturated sand and before that we discussed settlement of dry sands also. Now, coming to the next part that is instability and this instability is caused by the liquefaction due to the liquefaction of the soils. Liquefaction induced instabilities are among the most damaging of an earthquake hazard. So, during earthquake if there is a liquefaction induced instability that will cause lot of damage.

Their effects have been observed in the form of flow slides, lateral spreads, retaining wall failures and foundation failures in countless earthquakes throughout the world. So, the effect has been observed it is not there that it is not known, it is well known the effects of the instability are known. Instability, failures can be produced by different liquefaction phenomena and not always clear exactly which is responsible for a given failure. So, when we talk about instability you have different earthquake phenomena, particular two phenomena we are discussing lot. One is flow liquefaction, another is cyclic mobility. Now, sometimes it is difficult to like there is instability but this instability is due to the flow liquefaction or cyclic mobility. It is difficult to distinguish between these two but that is also sometime may not required. Instability occurs when the shear stresses which is required to maintain equilibrium of soil deposit exceed the shear strength. So, the shear stresses which develop inside the soil is more than the shear strength of that deposit. In that case there will be what we call the flow liquefaction.

The soil then deforms until it reaches a configuration in which the shear stress does not exceed the shear strength. So, what will happen? If your shear stresses are exceeding shear strength there will be instability and it will continue until the point where it automatically finds some natural equilibrium condition and when it reaches to natural equilibrium condition naturally the strain level will be high. The amount of deformation which is required to reach a stable configuration is strongly influenced by the difference between the shear stresses required for equilibrium and the shear strength of the liquefied soils. So, one side you have shear stresses, another side you have the shear strength of the liquefied soils. So, whatever the difference is there that will be manifest there.

Continue with the shear strength of liquefied soils. If the shear strength of the liquefied soil is only slightly lower then the shear stress which is required for equilibrium permanent deformation are likely to be small. So, if difference is not large between shear strength and shear stresses then the development of permanent deformation will be less. If the difference between the shear strength and the shear strength very large, if it is large then one can expect that there will be very large deformation may develop. Accurate evaluation of the effects of liquefying induced instability requires accurate estimation of the shear strength of the liquefied soils.

Now continue effects of liquefaction in terms of shear strength of liquefied soils. We want to find out the shear strength of the liquefied soils. So, there are three different approaches have been developed to estimate the shear strength of liquefied soils and what these approaches. One is you test in the laboratory. So, laboratory testing approach, steady state strength is assumed in the laboratory and there could be institute testing approach which is from based on the field test and then what you call the residual strength is one of the approaches which is typically used for institute testing approach.

Then you could have normalized strength approach which is also residual strength ratio approach. We are going to discuss one by one. So, here effects of liquefaction next is flow failures. What happens in this flow failure and then you have the cyclic mobility also. So, let us discuss first the flow failure. Liquefaction induced flow failures occurs when the shear stress required to maintain the state equilibrium are greater than the shear strength of liquefied soil. So, this thing we already discussed many times that for flow failures one of the conditions is that shear stress must exceed the shear strength and this situation can arise in several different ways and the National Research Councils of USA NRC identify four different mechanisms for flow failures which we are going to discuss one by one. Estimation of the deformation which is produced by liquefaction induced flow failures is extremely difficult. Flow liquefaction failures, NRC there are different stress for this. So, NRC mechanism A is listed here what National Research Council's mechanism says.

Flow liquefaction represent an important flow failure mechanism. So, the flow liquefaction is occurring that itself is an important indicator. Flow liquefaction occurs under totally undrained conditions, no redistribution of pore water pressure or change in void ratio is involved. So, here we assume that there is no drainage not allowed and the flow this failure there is change in the excess pore pressure and that is continuously and that change in excess pore pressure is leading to the liquefaction flow. Flow liquefaction is initiated when sufficient pore pressure is generated to move the effective stress path of an element of soil from its initial position to the flow liquefaction surface.

So, if we draw a stress path that is a graph between p and p dash and q which we already discussed stress path and in that case it will look like this because normally this is p and on y axis we have q. In fact, q and q prime is same as we discussed earlier. So, normally it is a common practice to use q only rather than q prime on y axis. So, in this case stress path when you have the stress path sufficient pore pressure need to be generated and this pore pressure should be enough to move the effective stress path of an element of soil from its initial position to the flow liquefaction surface. In the stress path you will have the one flow liquefaction surface like this is this flow liquefaction surface is there.

So, once you have this flow liquefaction surface and if this reaches then effective path reaches to this point then the flow liquefaction takes place. When that occurs, the settlement becomes unstable and flow liquefaction failure will be biggest in that element of soil. In other words, if flow liquefaction failure occurs at the location where liquefaction is initiated by earthquake shaking. These characteristics can be used to distinguish between the flow liquefaction failures and other types of flow failures. Flow liquefaction failure often occur very quickly and produce large soil movements. Flow liquefaction failure can develop progressively that is the initiation of flow liquefaction in a small volume of soil may spread to large produce a large flow failure. When flow liquefaction is initiated at a particular location the shearing resistance drop to the steady state strength. So, thus there will be decrease in the shearing stress. The static shear stresses that were registered at the locations must be transformed to the surrounding soil where they may initiate further flow liquefaction.

So, there should be redistribution of the stresses. As the redistribution of stresses proceed the zone of liquefaction grows that is maybe expected earlier smaller, but when the stresses develop more then the zone of liquefaction will be high. Eventually a massive flow slide may develop. Now there could be the second mechanism which is based on the NRC only and NRC says that this mechanism of flow liquefaction flow failures may be in the second stage local loosening flow failure. Since the steady strength is very sensitive to the density of many soils a small amount of loosening can reduce the steady state strength substantially. In some cases, loosening may reduce the steady state strength to a value which is smaller than the shear stress required for equilibrium, thereby producing a flow failure. So, this loosening of the soil not only it like you knows there is change in stresses, but it also reduces the steady state strength and that once steady stress is itself is reduced then naturally even the stresses are low, but then there will be chances of liquefaction. If a sand layer is overlain by less permeable material that does not permit drainage. If you have a sand layer, but on the top of the sand layer you have low permeable layer during the earthquake, the total volume of the sand will remain constant. If a condition of initial liquefaction that is zero effective stress is reached, how the sand properties particles may rearrange under the action of gravity so that the lower part of the layer becomes denser and the upper part becomes looser. So, here you have two layers, one is the top part and another is the upper layer.

So, you can say like this is the upper layer and this is lower layer. So, what happens the lower part could be denser which is also expected that is because the density of the soil may vary from the top to bottom it increases and the upper part will be subjected to this. So, the upper part may start the liquefaction the initial flow failures. If the upper part loses sufficiently to reduce the steady state strength to a value smaller than the steady shear stress, in local loosening the flow failure occurs.

So, if it is small then here will be local loosening. Rather than the global loosening of this will occur. In extreme cases, a water interlayer may form beneath the less permeable material. Since the water interlayer would have zero shear strength, a flow failure could easily be produced. We have discussed two mechanisms related to flow failures which are based on the NRC recommendation. Now, we are going to talk about the mechanism A and B has been already discussed.

Now, we are going to discuss NRC mechanism C which is global loosening flow failure. So, in this case high excess power pressure which is generated at depth will cause poor water to flow towards drainage boundaries during and after an earthquake. Most of the flow is usually directed toward the ground surface and here why it is happening because there is a development of high excess power pressure. Sallow soils may be loosened by this flow to the extent that their steady state strength drops below the shear stress which is required to maintain equilibrium. In contrast with the local loosening case which was the case number B, this loosening is not compensated by densification at a different location.

Since the steady strength is not reduced until water flows into the shallow soil, failure may not occur until well after the earthquake. So, cracking of the surficial soils may also contribute to the failure. Now, the last stage that is interface flow failure which is the NRC recommendation mechanism recommendation D. Flow type failures can also occur when the shear strength of the interface between a liquefiable soil and a structure becomes smaller than the shear stress required for equilibrium. Plunging the failure of friction piles in an interface flow failure.

If the interface is smooth as was the case with steel or pre cast concrete piles, interface flow failures does not require volume change of the soil and therefore can occur in contractive or dilative sense. It can occur both contractive or dilative sense. So, this was about interface flow failures NRC mechanism D. Now, we are done with the flow failures. As for deformation failures are concerned, the ground surface may exhibit what you call the fissures and scarves at the head of the lateral spread shear zone along its lateral margins and compressed or buckled soil at the toe.

The surficial blocks usually more irregularly in both horizontal and vertical directions, buildings and pipelines extending across or through the head of a lateral spread may be pulled apart, pipelines which are crossing the lateral margins may be sheared and bridges or pipelines near the toe may be buckled. So, this deformation failure is very dangerous. So, many things will happen like it will be like. So, for example, there is one example which is related to the cyclic mobility that is lateral displacement usually which range from a few centimeters to a meter or two, but may be larger if shaking is strong or of long duration. So, in this slide, there is a lateral spreading which cause the bridge pier foundation to move and rotate sufficiently to for simply supported bridge spun to fail.

So, because due to lateral spreading bridge pier foundation get moves and rotate sufficiently, so then it will it is simply supported bridge spun fails. So, this was one of the example related deformation failures. With this I conclude this lecture number 40 that is the last lecture on the liquefaction of soils and this fourth chapter of module fourth that is on liquefaction of soils get completed. And with this we are done with lecture number 40 and the we will continue in the next day for the lecture number 41. Thank you very much for your kind attention. Thank you.